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Lake Shore Boulevard Design Review

Marquette, MI

September 2, 2014
12035.200

Financial assistance for this project was provided, in part, by the Michigan Coastal Management Program, Department of Environmental Quality (DEQ), through a grant from the National Oceanic and Atmospheric Administration (NOAA), U.S. Department of Commerce. The statements, findings, conclusions, and recommendations in this report are those of the Superior Watershed Partnership (Grantee) and do not necessarily reflect the views of the DEQ and the NOAA.

Innovation, Excellence & Service

Lake Shore Boulevard Design Review

Prepared for

City of Marquette

Prepared by

Baird

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Project Number: 12035.200

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EXECUTIVE SUMMARY

The City of Marquette, Michigan is planning to reconstruct approximately 3,250 feet of revetment along Lake Superior. The project includes the landward relocation of Lake Shore Boulevard and the creation of green space between the road and the revetment to prevent road closures due to significant overtopping and flooding as well as to increase public safety. The City plans to construct a rubblemound revetment as designed by Barr Engineering Co. (2001) and asked Baird to review the revetment cross-section design in support of their final design activities. Upon completing the review Baird does not recommend any specific changes to the revetment design.

Overall, the proposed cross-section design is sound and could be considered conservative, as it is based on the 1984 Shore Protection Manual (SPM). Baird's experience with shallow water structures on the Great Lakes indicates that the less conservative design method presented in the 1977 SPM is acceptable.

The cross-section was designed for the 200-year storm (combined 20-year waves and 10-year water levels) as is typical for USACE design on the Great Lakes. The water level and wave data were reviewed. Updated wave data is available on the USACE Wave Information Studies (WIS) website. However, the site is depth limited, which means that the water level at the structure will control the design significant wave height (H_s) and that larger, updated offshore wave heights will not significantly affect the armor stone size. It is important to verify that the water depth at the proposed structure is accurate. Baird recommends undertaking a nearshore bathymetric survey to verify this prior to construction. Furthermore, the shorelines north and south of the revetment are beaches, which are dynamic by nature. It is therefore important to understand the variability in the beach in response to individual storm events and seasonal water level variations. Historical nearshore beach profiles are required for comparison to determine the depth of the variation. In the absence of historical beach profiles, additional coastal process analyses should be undertaken to assess the potential variability in the nearshore profile lakeward of the revetment.

In addition, soil sampling and grain size distribution tests are recommended to determine the sediment characteristics. Based on photographs, the nearshore sediment appears to be sand. Jet probes are suggested to determine the depth of the sand layer to understand if the proposed revetment will have an adequate foundation.

The revetment design allows for wave overtopping. Calculations for wave run-up were verified and several empirical methods were used to estimate the amount of overtopping to be expected during the 200-yr design event. The volume of overtopped water for the top 2% of the waves was correlated with published damage and safety thresholds.

The draft construction drawing set was reviewed and recommendations were made concerning the demolition of the existing revetment; the interface between the proposed revetment and dune/swale complex, as well as the overlook structures. Specific stone sizes should be called out on revetment cross-sections. Additional details are suggested for the north and south terminations of the revetment; the dunes and swales terminations; the weir at the detention pond; the interface of the outfall pipe and the revetment; and for turbidity and erosion control during construction. It is also noted that geotextile specifications require updating to reflect changes resulting from updated manufacturing processes and implementation experience.

1.0 INTRODUCTION

The City of Marquette is planning to reconfigure and rebuild approximately 4,200 feet of Lakeshore Boulevard along Lake Superior. The project includes the relocation of Lake Shore Boulevard, the reconstruction of approximately 3,250 feet of revetment, and the creation of a public waterfront park including a bike path, parking areas, overlooks, and stormwater management features including a detention pond and a dune/swale complex (see Figure 1.1). The project's main area of focus is the stretch of Lakeshore Boulevard that runs parallel to the Lake Superior shoreline from Hawley Street to a point midway between Wright Street and Pine Street. This stretch of roadway is in close proximity to the shoreline, adjacent to an existing stone revetment.

The project need has been identified by the City based on the deteriorating condition of the existing revetment, as well as flooding and drainage problems along Lakeshore Boulevard. At present, there are several issues with the existing stone revetment and roadway alignment that are driving this redevelopment. This site can experience significant wave overtopping during storm events, especially at high water levels. This, in turn, causes roadway closures and is a general safety concern for the public. Flooding becomes an issue due to overtopping waves because various sections of existing land along Lakeshore Boulevard have limited flood storage capacity. During the winter months, ice build-up along the revetment causes structural undermining and damage as well as interruptions to roadway usage.

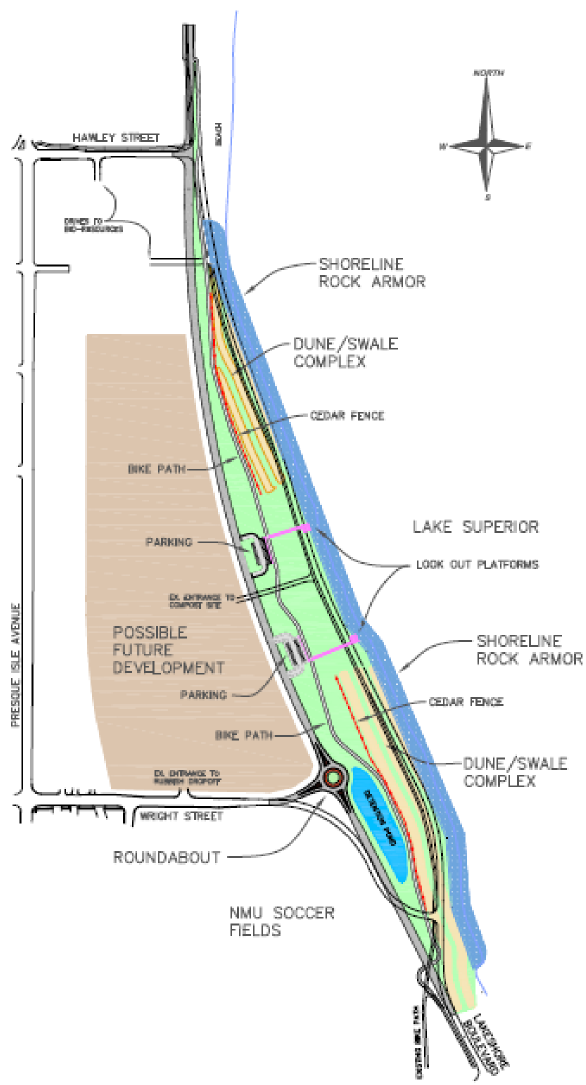

Figure 1.1 Selected Project Preferred Plan

The City plans to address these issues by utilizing a rubblemound revetment design developed by Barr (2001). It is proposed to use the Barr (2001) design to reconstruct the existing stone revetment.

The City of Marquette has retained Baird to undergo a design review of the cross-section presented by Barr (2001) for purposes of final design for the Lakeshore Boulevard Coastal Restoration project. This review will support final design activities and production of construction documents by the City of Marquette. The City plans to finalize these documents during the summer of 2014.

Baird has completed a review of the cross-sections, as well as other characteristics of the project layout. The information contained in this report includes a renewed assessment of existing design reports, site conditions, and design conditions and constraints to provide a complete review of the Barr (2001) cross-section.

2.0 REVIEW OF EXISTING INFORMATION

The following sections describe the information contained in documents that were provided to Baird for review by the City of Marquette.

2.1 Barr Report (November 2001)

The Barr (2001) report as received by Baird includes:

- A digital copy of Drawings and specifications of the proposed design
- Cost estimates associated with the presented design
- Design documentation (consisting of Barr internal memorandums) that includes:
    - Wave run-up calculations
    - Stone size and layer thickness calculations
    - Volume estimates of the existing stone revetment
    - Sand excavation volume estimates
    - A site visit report
    - Design drawings
    - Wave and water level data analysis
- Results of an independent technical review

From the internal design memorandums, it appears that both concrete and sheet pile wall structures were also considered along with the rubblemound revetment concept. However, the preliminary cost estimate favored the rubblemound revetment option. Other considerations, such as maintenance, constructability, and aesthetics were also considered.

2.2 Baird Report (July 2013)

Baird was retained to produce conceptual design alternatives for the Lake Shore Boulevard shoreline redesign and restoration with the goal of reducing wave overtopping, flooding, and over-spray on the roadway during both winter and summer months.

The existing conditions and a preliminary understanding of the coastal processes were documented including sediment characteristics, water levels, waves, sediment transport processes and shoreline evolution.

Five conceptual alternatives were produced as part of this study:

1. Remove existing revetment
2. Restore revetment
3. Landward beach development

4. Nearshore breakwaters and beach development
5. Stone groyne beach development

With the exception of option 2 above, all of the above options require that Lake Shore Boulevard be realigned. Preliminary construction cost estimates supporting these options and recommendations for further studies were also provided in this report.

The City of Marquette has since chosen to restore the revetment as well as realigning Lake Shore Boulevard so that there is ample setback between the revetment and the road.

2.3 Draft Construction Drawing Set Review

The City of Marquette requested that Baird perform a preliminary review of the draft drawing set for the Lakeshore Boulevard Coastal Restoration project. Draft drawings (PDF) were provided to Baird on July 9, 2014. Baird's cursory review of the drawings focused on overall designed constructability. It is assumed that the City of Marquette will complete an independent Quality Assurance/Quality Control and technical review of the final documents before bidding or construction. The following items have been identified as areas that may warrant further review and/or drawing revision.

2.3.1 Removal and Earthwork Plan

"Sheet 2" could be revised to better show limits of construction, salvage, clearing and grubbing, tree protection, etc. It is unclear if the existing revetment between the proposed revetment termination and Hawley Street is meant to be removed. While this area appears to include an existing beach, additional analysis may be beneficial to understand potential overtopping of the roadway in this area given its proximity to the shoreline.

2.3.2 Demo/Sorting/Staging of Existing Revetment

The demolition of the existing revetment may require a substantial land area for sorting existing armor stone, staging replacement armor stone, and reconstruction of the revetment. Access for construction machinery may be challenging or require additional temporary access.

2.3.3 Cross-Sections

The interface between the rebuilt revetment and the proposed dune/swale complex could experience erosion due to overtopping of the revetment during severe storms. Armoring of the slope from the +608.5′ terrace at the landside of the revetment or additional support of revetment "Under Layer" in this area may be beneficial.

The Barr (2001) drawings indicate an armor layer thickness but do not specify the number of layers or units associated with this measurement. It would be useful to have either or both of the stone weight range and number of layers of armor stone intended for the course shown on the drawings.

2.3.4 Revetment Termination

Additional detail of the north and south termination of the revetment into existing ground may be necessary to ensure accurate construction and stability of the revetment.

2.3.5 Swale Terminations

Additional detail or grading plans of the proposed dune and swale terminations may be necessary to ensure accurate construction. It is assumed that additional planting plans and specifications will be provided.

2.3.6 Overlooks

It is noted that the proposed overlook structures will be located over the landward side of the proposed revetment. Additional detail and specification may be required on properly sizing the earth retention plate, support piles, and construction.

2.3.7 Weir at Detention Pond

Additional detail, weirs, collars, or headwalls may be required at the interface between the concrete pipe and the proposed detention pond.

2.3.8 Outfall Pipe/Revetment Interface

Additional detail may be required at the interface of the outfall pipe and the revetment for proper placement of armor stone and protection of the pipe.

2.3.9 Permitting/Erosion Control

It is assumed that the City of Marquette has received all necessary permits for construction in and adjacent to Lake Superior. Additional detail may be required for turbidity, and erosion control measures if required by these permits.

3.0 SITE CONDITIONS

3.1 Existing Conditions

Baird has performed a preliminary review of several sources of available information to achieve a general understanding of the existing conditions at the Lakeshore Boulevard location.

The northern section of the stone revetment was constructed before 1939, while the southern section was constructed between 1939 and 1972. Based on a review of aerial photographs and images supplied by the Lake Superior Watershed Partnership (LSWP), it appears that the revetment shows signs of aging, was not well engineered, and the material was dumped in place by trucks. The material ranges in size from cobble to armor stone (Figure 3.1).

Figure 3.1 Lakeshore Boulevard Stone Revetment

Physical changes to the shoreline south of the upper harbor have been recorded since the completion of the multi-phase harbor of refuge in 1939, which included the construction of a 2,600 foot Federal breakwater. The shoreline erosion processes south of the harbor determined the need to provide a stone revetment along Lakeshore Boulevard, as shown in Figure 3.2.

Figure 3.2 Project Location Map

The National Oceanic and Atmospheric Administration (NOAA) have published LiDAR (Light Detection and Ranging) surveys of the Great Lakes from several sources including the U.S. Army Corps of Engineers (USACE) through the Joint Airborne LiDAR Bathymetry Technical Center of Expertise. Data compiled by NOAA in 2011 was utilized in the basemap created for the Baird (2013) study as shown in Figure 3.3.

Based on the 2011 LiDAR data, the following elevations are noted. The LiDAR data refers to the International Great Lakes Datum of 1985 (IGLD 85). The difference between IGLD 85 and the National Geodetic Vertical Datum of 1929 (NAV 29) is discussed in Section 4.1.

- Elevations along the centerline of Lakeshore Boulevard vary between 606.5 ft (Hawley Street) to 606.8 ft (Wright Street) IGLD 85.
- Elevations along the crest of the existing stone revetment vary between 608.0 and 612.0 ft (higher elevations at Wright Street) IGLD 85.
- The average elevation along the toe of the revetment is 577.0 ft IGLD 85.

It is suggested that a nearshore bathymetric survey be completed prior to construction to verify the elevation of the lakebed and the toe of the proposed revetment along its full length.

Figure 3.3 Existing Conditions (Baird, 2013)

3.2 Proposed Design Alternative

It is understood that the selected design alternative includes relocating the roadway alignment to increase the area between the revetment and roadway. The revetment will also be reconstructed between Hawley and Wright Streets. The area between the roadway and revetment is to be developed into waterfront park space and shoreline bike paths. Figure 3.4 illustrates the proposed plan.

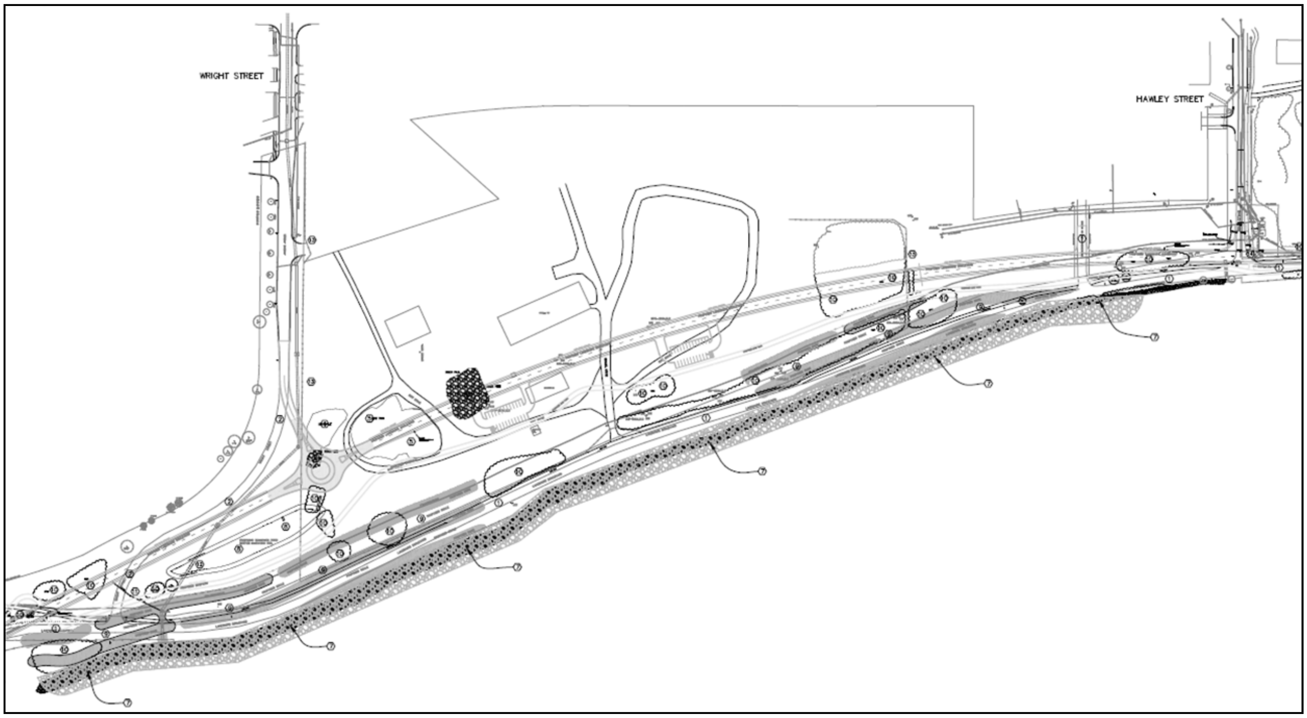

Figure 3.4 Proposed Layout

3.3 Coastal Conditions Overview

3.3.1 Sediment Characteristics

No data is available regarding the properties of the nearshore sediments or sediment depths (thicknesses) in the project area. The photograph in Figure 3.1 shows sand in the nearshore area in front of the existing revetment. Generally, detailed soil sampling and analysis is required to understand if the proposed revetment will have an adequate foundation. For example, if the sand layer is too deep, the revetment might sink into the sand layer. Areas where sediment is encountered could be excavated to suitably hard substrate, or alternatively a stone bedding layer could be installed. This will require review during construction of the revetment. There is no information documenting any subsiding of the current revetment. However, if subsidence occurred, it would likely have happened quickly (following construction). Jet probes are suggested to confirm sediment depths.

3.3.2 Shoreline Evolution

The evolution of the shoreline from 1951 to 2008 was documented in the Baird (2013) report. The proposed revetment is replacing an existing one and is therefore not likely to impede or significantly alter any sediment processes in the region.

3.3.3 Water Levels

Water levels may have a significant influence on the performance of coastal structures in shallow water where the wave conditions at the structure may be depth-limited. In this case, the wave heights will increase at higher water levels, leading to a requirement for larger armor stone to provide a stable armor layer. In addition, the freeboard of the structure (i.e. height of crest above water level) is reduced at higher water levels, resulting in more overtopping and an increased risk of damage. As such, the effectiveness of any proposed shoreline protection structure will be reduced at higher water elevations. Low water levels may be a consideration in the design of an appropriate toe detail for the structure.

Water levels on Lake Superior vary on several different time scales in response to climatic fluctuations. Changes in precipitation and evaporation patterns over the Great Lakes drainage basin result in long-term (i.e. over a period of years) lake level fluctuations in the order of several feet on Lake Superior. In addition, there are seasonal changes in water level due to annual precipitation patterns and spring runoff. Lake Superior levels tend to be highest during July and lowest during March, with the typical annual variation in lake levels being approximately 1 foot. In addition, local water levels may vary significantly on a short-term basis (i.e. over a period of hours to days) due to storm surge resulting from meteorological forcing (i.e. wind stress and barometric pressure). Storm surges on Lake Superior may reach several feet (positive/setup or negative/setdown), depending on site location, and occur independent of the long-term and seasonal lake level fluctuations. However, there is a tendency for the most severe surges to occur during the stormy winter period when lake levels tend to be lower.

The Baird (2013) report provides water level data as analyzed by the USACE in a 1993 publication (Design Water Level Determination on The Great Lakes) in which design water levels are estimated for various return periods. The design water level is defined as the lake level and storm surge combined. These values are further discussed in Section 4.1.

3.3.4 Wave Climate

Wave data from the Baird (2013) and the Barr (2001) reports are discussed in Section 4.2.

4.0 REVIEW OF DESIGN CONDITIONS

4.1 Design Water Levels

The design water level including the effects of long term and seasonal lake level fluctuations, as well as storm surge for various recurrence intervals is shown in Table 4.1. These values were determined by the USACE (1993) and are reported in the Baird (2013) report as well as the Barr (2001) design report. The elevations are based on a gage analysis of 130 years of data at Marquette, MI.

Table 4.1 Summary of Design Water Levels Near Presque Isle, MI (USACE, 1993)

Recurrence interval	Design water levels (IGLD 85)
10 year	604.2 ft
20 year	604.3 ft
30 year	604.5 ft
50 year	604.7 ft
100 year	604.9 ft

It should be noted that there is a conversion factor between the NAV 29 (or NGVD 29) and IGLD 85 datums which is location dependent. There is no difference between IGLD 85 and NAVD 88. Figure 4.1 shows that the difference between NAV 88 (or IGLD 85) and NAV 29 is between 0 and 8 inches (20 cm). The exact difference needs to be interpolated from sites where the conversion was calculated.

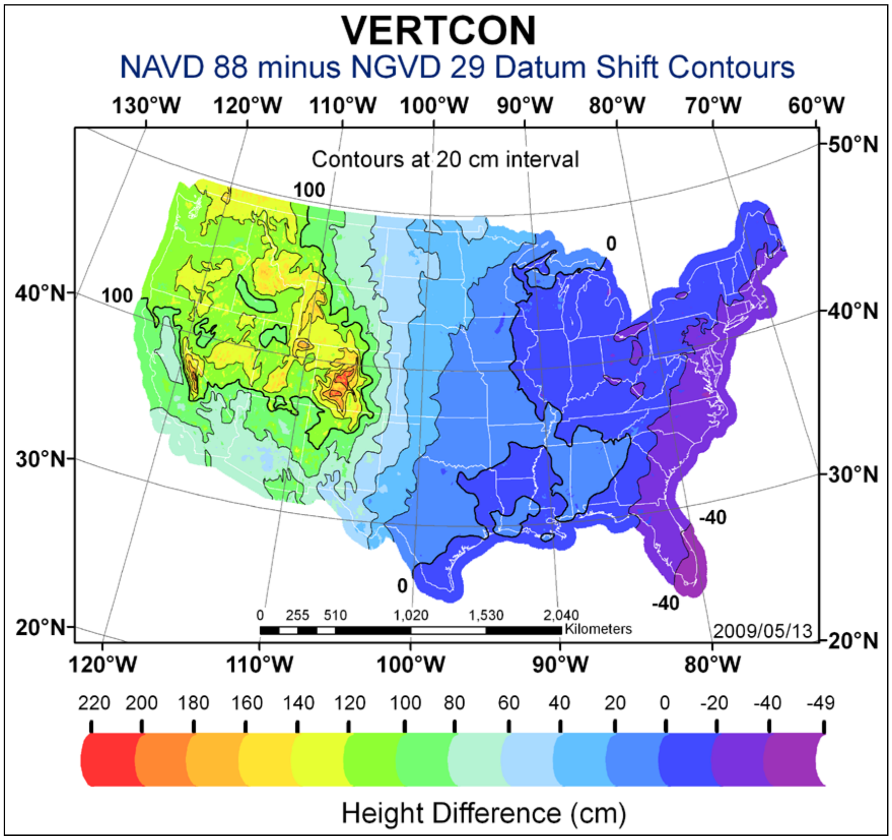

Figure 4.1 Difference between NAV29 and IGLD85 datums (NOAA, 2013)

The drawings use NAV 29 and the following equations convert the design water level shown on the drawings (604.13 ft) to NAV 88 (or IGLD 85) to relate to the water level analysis.

$$\text{NAV 88} - \text{NAV 29} = 8'' \text{ (or 0)}$$

$$\text{NAV 88} = 604.13 \text{ ft} + 8'' \text{ (or 0)}$$

$$\text{NAV 88} = 604.8 \text{ ft (or 604.13 ft)}$$

If a difference of 8" is assumed, the design water level listed on the drawings relates to between the 50- and 100-year recurrence intervals, which are more conservative than the assumed 10-year recurrence used in design calculations. Therefore, it was assumed that there is a difference of 0 between IGLD 85 and NAV 29 at the project site.

It should be noted that the 20-year water level was used in the Baird (2013) conceptual design report rather than the 10-year water level used in the Barr (2001) calculations. However, the difference between the two is very small (2 inches).

4.2 Waves

Barr (2001) reports using a 1978 USACE hindcast, which yielded 20-year deep water significant wave heights (H_s) of 12.5, 18.4 and 14.8 ft from angle classes 1, 2 and 3 respectively. Because the site is largely sheltered from angle class 3, waves from this angle are not used for design. The design waves from the Barr (2001) report are summarized in Table 4.2.

Table 4.2 Summary of Waves

Incident Wave Height	Significant Deep Water Wave	
	at Hawley St	at Fair Ave
Angle class 2 with diffraction	2.8 ft	14.7 ft
Angle class 1 with no diffraction	12.5 ft	12.5 ft

The toe of the proposed revetment is in relatively shallow water. Therefore, the waves listed in Table 4.1 will break offshore and the design wave height for the revetment should be determined by a breaking wave analysis under depth limited conditions. Barr (2001) estimated the breaking wave at the structure (H_b, or H_{10}) to be 7.0 ft, while the significant wave height (H_s) is 5.5 ft. According to the Barr (2001) wave analysis, this corresponds to the 20-year deep water significant wave.

Baird used Goda's method (1970) to estimate shoaling and breaking effects as the deepwater waves propagate into shore. Lakebed slopes were determined from three representative cross-sections extracted from the available LiDAR data. One of the Barr internal memos states that a T = 7 s wave was assumed, which is considered typical for storm events in the area. Table 4.3 provides the inputs and results from the Goda wave transformations.

Table 4.3 Deepwater Wave Transformation to Structure (Goda, 1970)

<table>
<tr><th colspan="2">Deepwater wave</th><th>Slope</th><th>depth at the structure</th><th>H_s</th></tr>
<tr><td rowspan="2">Angle class 1 with no diffraction at Hawley St</td><td rowspan="2">12.5 ft</td><td>1:40</td><td rowspan="3">7.1 ft</td><td>5.3 ft</td></tr>
<tr><td rowspan="2">1:30</td><td>5.8 ft</td></tr>
<tr><td>*Angle class 2 with diffraction at Wright street end of proposed revetment</td><td>9.4 ft</td><td>5.3 ft</td></tr>
</table>

*This wave height was interpolated as the study area does not extend all the way to Fair Ave.

The resulting H_s values in Table 4.3 compare well to the design significant wave height (H_b) used by Barr (2001). More detailed calculations can be found in Appendix A.

The revetment is situated in depth limited conditions. Therefore, the significant wave height (H_s) at the structure is governed by the revetment toe depth and the design water level. Because the adjacent shorelines to the north and south of the breakwater are beaches which are dynamic by nature, it is important to understand the variability in the beach in response to individual storm events and seasonal water level variations. Historical nearshore beach profiles are required for comparison to determine the depth of the variation. The toe of the revetment should reside below the level of variability in the lakebed.

The USACE Wave Information Studies (WIS) hindcast database has recently been updated (USACE, 2014) to include data from 1979 to 2012. Stations 95078 and 95077 (Figure 4.2) are closest to the site. The 20-year deep water wave is reported as 18.4 and 18.7 ft respectively. The 2-, 5-, 10- and 20-year deep water waves for WIS station 95077 are shown in Table 4.4. This updated hindcast includes the use of high resolution Climate Forecast System Reanalysis (CSFR) winds. However, there is no validation documentation available at this time. Note that the 95077 WIS waves for all return periods are larger than recorded in the Barr (2001) report. However, because the waves are depth limited, the transformed wave at the shore does not significantly increase; from 5.5 ft (Barr, 2001) to 5.8 ft for the 20-yr event. A revised deep water wave height may be larger, but would not significantly change the size of armor stone required as this is dependent on H_s at the structure.

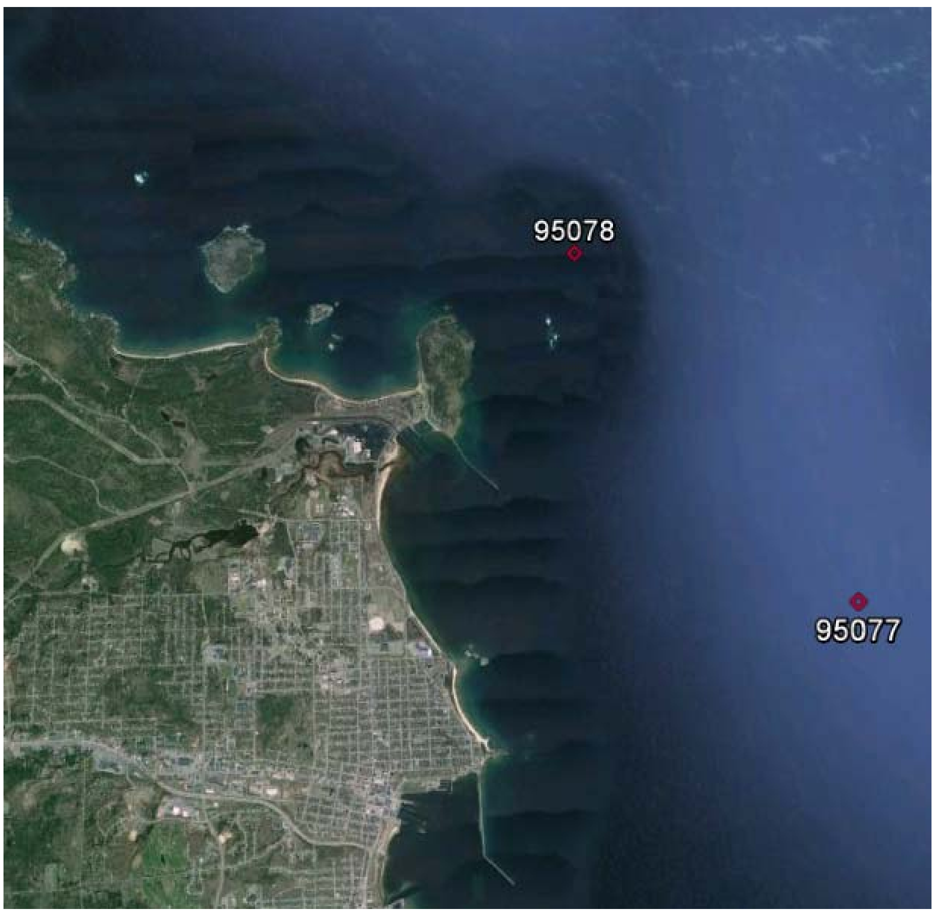

Figure 4.2 Location of the WIS deep water wave hindcast data

Table 4.4 Deep Water Waves for various Return Periods

Recurrence interval	Deep water wave height (USACE 2014)	Transformed to shore (Goda, 1970)
2-year	14.4 ft	5.6 ft
5-year	15.7 ft	5.7 ft
10-year	17.4 ft	5.8 ft
20-year	18.7 ft	5.8 ft

4.3 Basis of Design

Coastal structures are designed to resist a harsh marine environment governed largely by the passage of episodic storm events. The greatest storm to occur over the life of the structure can only be *estimated* at the time of design. Therefore, there is always some inherent risk associated with the design conditions; it is important that the Owner has a clear understanding of these risks.

4.3.1 Risk

There are no firm rules for selecting the level of acceptable risk for coastal structures; however, industry practice guidelines (e.g., PIANC, 2003; Lamberti, 1992; ISO/DIS 21650:2007) provide some insight on the level of acceptable risk for design purposes based on various safety classes. Ultimate Limit States (ULS) is applicable for catastrophic failure of the structure, and Serviceability Limit States (SLS) is applicable for structures which can sustain some damage and remain functional (such as an armor stone shore protection). Table 4.5 summarizes the maximum acceptable risk based on these safety class levels (PIANC, 2003), along with examples provided in ISO/DIN 21650:2007.

It is suggested that a *very low* to *low* risk level be considered for the Lake Shore Boulevard revetment as there is very little risk to public as they should not be using the green space during the design event. Some environmental consequences would result in the failure of the breakwater because of loss of shoreline and/or green space. There would be some economic consequences and, depending on the extent of public profile for this project, failure may also result in some political consequences. However, it is up to the Owner to choose the level of risk, that they are comfortable with.

Table 4.5 Acceptable Level of Risk for Various Safety Classes (PIANC, 2003)

Safety Class	Indicators	SLS	ULS	Examples (ISO/DIS 21650:2007)
Very Low	• No risk to human injury • Small environmental consequences • Small economic consequences	40%	20%	Small coastal structures
Low	• No risk to human injury • Some environmental consequences • Some economic consequences	20%	10%	Larger coastal structures such as breakwaters in deep water and exposed seawalls protecting infrastructure
Normal	• Risk to human injury • Some environmental consequences • High economic or political consequences	10%	5%	Breakwaters protecting a LNG-terminal or power station
High	• Risk to human injury • Significant environmental consequences • Very high economic or political consequences	5%	1%	Sea dyke protecting a populated low land

Serviceability Limit State (SLS) – e.g., overtopping, settlement
Ultimate Limit State (ULS) – e.g., foundation failure, failure of significant portion of structure

4.3.2 Design Life

The design life of a structure is the period of time during which it is expected to function properly without significant maintenance, repair works, or replacement. The design life of a structure is typically selected by the Owner, along with the level of accepted risk. Once these parameters have been specified, the design conditions may be established.

The minimum design life for various structure types and security levels are defined in the Spanish Maritime Works Recommendations (ROM, 2002) and PIANC (2003), and are summarized in Table 4.6. It is important to recognize that the design life is not equivalent to the return period of the design conditions.

Table 4.6 Design (Service) Life for Coastal Structures (ROM 2002, PIANC 2003)

Type of Work or Installation	Required Security Level		
	Level 1	Level 2	Level 3
General Use Infrastructure	25	50	100
Specific Industrial Infrastructure	15	25	50

Specific Industrial Infrastructure refers to works in the service of a particular industrial installation or associated with mineral deposits. General Use Infrastructure refers to works not associated with the use of an industrial installation or mineral deposit. Specific industrial infrastructure works typically have a lower specified design life because they are often associated with structures such as mines that will not be operational after the supply of minerals has been depleted. (PIANC, 2003)

Security Level 1 refers to works and installations of local auxiliary interest with a small risk of loss of human life or environmental damage in the event of failure. Typical structures included in Security Level 1 are defense and coastal regeneration works, works in minor ports or marinas, local outfalls, pavements, commercial installations, buildings, etc. (PIANC, 2003).

Security level 2 refers to works and installations of general interest with a moderate risk of loss of human life or environmental damage in the event of failure. Typical structures included in security level 2 are large ports, outfalls of large cities, etc. (PIANC, 2003).

Security level 3 refers to works and installations for protection against inundations or international interest with an elevated risk of loss of human life or environmental damage in the event of failure. Typical structures included in security level 3 are utility-scale power plants and measures preventing significant damage to urban or industrial centers (PIANC, 2003).

The proposed revetment is General Use Infrastructure and could be categorized as Security Level 1 or 2 which dictates a 25 to 50 year design life.

4.3.3 Return Period of Design Event

Risk is defined as the probability that a given design event (for this project, a specified combination of monthly mean water level, storm surge, and wave height) will be reached or exceeded at least once during the project life. If the design event is reached or exceeded, there will be certain consequences that must be taken into consideration. For example, if the design conditions are exceeded, there may be damage to the coastal structure and a subsequent possibility of direct or indirect economic costs.

Once the acceptable level of risk and design life are established, the return period of the design event can be determined. The return period of the design event is calculated as a function of the specified risk and project life. Specifically, the risk (“R”) that a design event with a return period (T) will occur at least once in the project life of “n” successive years is defined by the equation:

$$R = 1 - (1 - 1/T)^n$$

Figure 4.3 illustrates the relationship between acceptable level of risk and return period for different design lives based on the equation above. For example, an acceptable risk of 0.4 and a 50 year design life results in the use of a 100 year return period, while an acceptable risk of 0.4 and a 25 year design life results in the use of a 50 year return period.

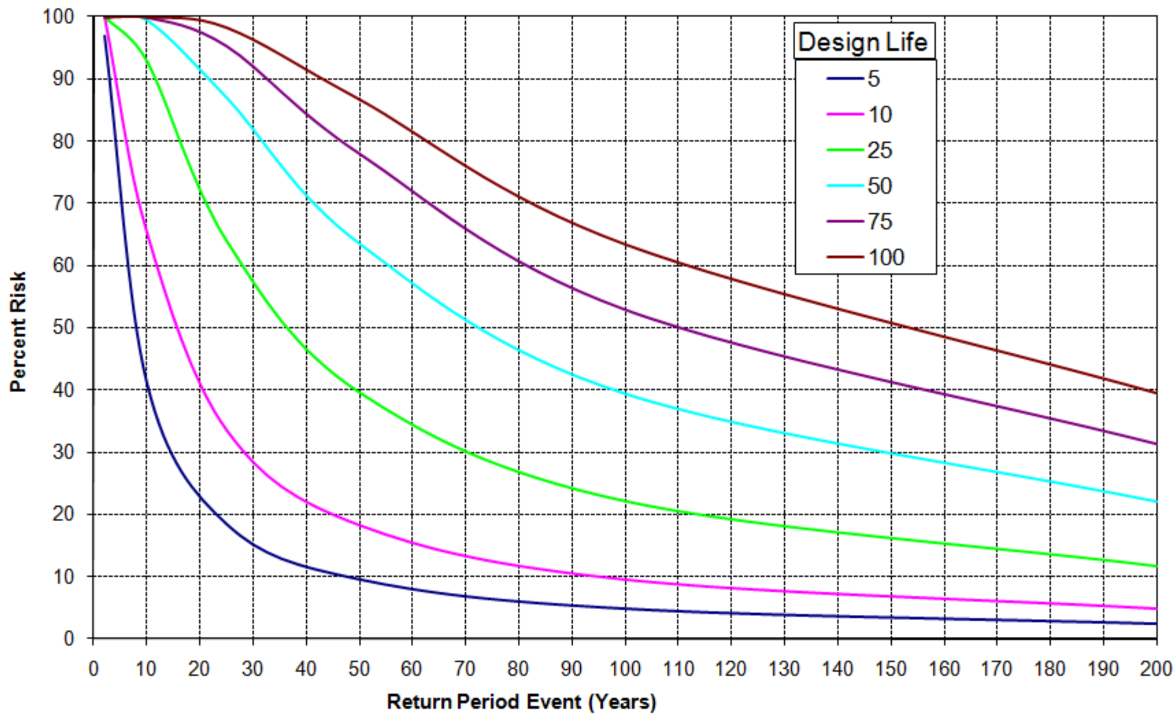

Figure 4.3 Risk of Event Occurring within the Design Life of a Structure

The selection of the acceptable level of risk and the design life is ultimately a decision that must be made by the Owner. Based on the aforementioned published literature and standard engineering practices, the return periods for the design of the proposed revetment are shown in Table 4.7.

Table 4.7 Design Return Period Summary

Safety Class	SLS	Design Life (years)	Event Return Period (years)
Very Low	40%	25-50	50-100
Low	20%	25-50	100-200

A *low* risk level results in a design return period of 100-200 years. The USACE standard for coastal design on the Great Lakes is nominally a 200 year return period, considering the combined effects of waves and water levels. Generally, different combinations of the waves and water levels would be tested to determine the worst case scenario; i.e. the 20-year wave x 10-year water level; 10-year wave x 20-year water level; or the 2-year wave × 100-year water level. Given that the waves are depth limited and the water levels do not significantly increase with increasing return period, a 200-year return period considering the combined effects of waves and water levels is considered acceptable.

In general, Rubblemound revetments are designed to accommodate some minor damage. Of note, the "no damage" criteria in standard Coastal Engineering practice typically allows for 0 to 5%

damage. In addition, some maintenance in response to long term deterioration or "wear and tear" should be expected.

In addition to the replacement costs due to deterioration of the stone, there should be an allowance for repairs to the structure which may be required as a result of storms exceeding the design event. Rubblemound structures generally fail progressively, with a failed structure generally retaining a significant amount of residual shore protection capability.

4.3.4 Typical Failure Mechanisms

CIRIA (2007) presents an overview of principal failure mechanisms for rock structures (Figure 4.4) and defines failure as a response to a defined loading which exceeds a value of performance related to the structure's functional requirements. Some failure modes can be allowed to occur repeatedly up to a certain limit during normal service life, such as overtopping or displacement of stones on a dynamically stable slope.

The degree to which the failure modes presented in Figure 4.4 are relevant varies for different structures. Some wave overtopping is permitted as discussed in Section 4.5. It is not possible to predict the mechanisms of failure, should it occur. No geotechnical information is available and the likelihood of a ship collision is unknown.

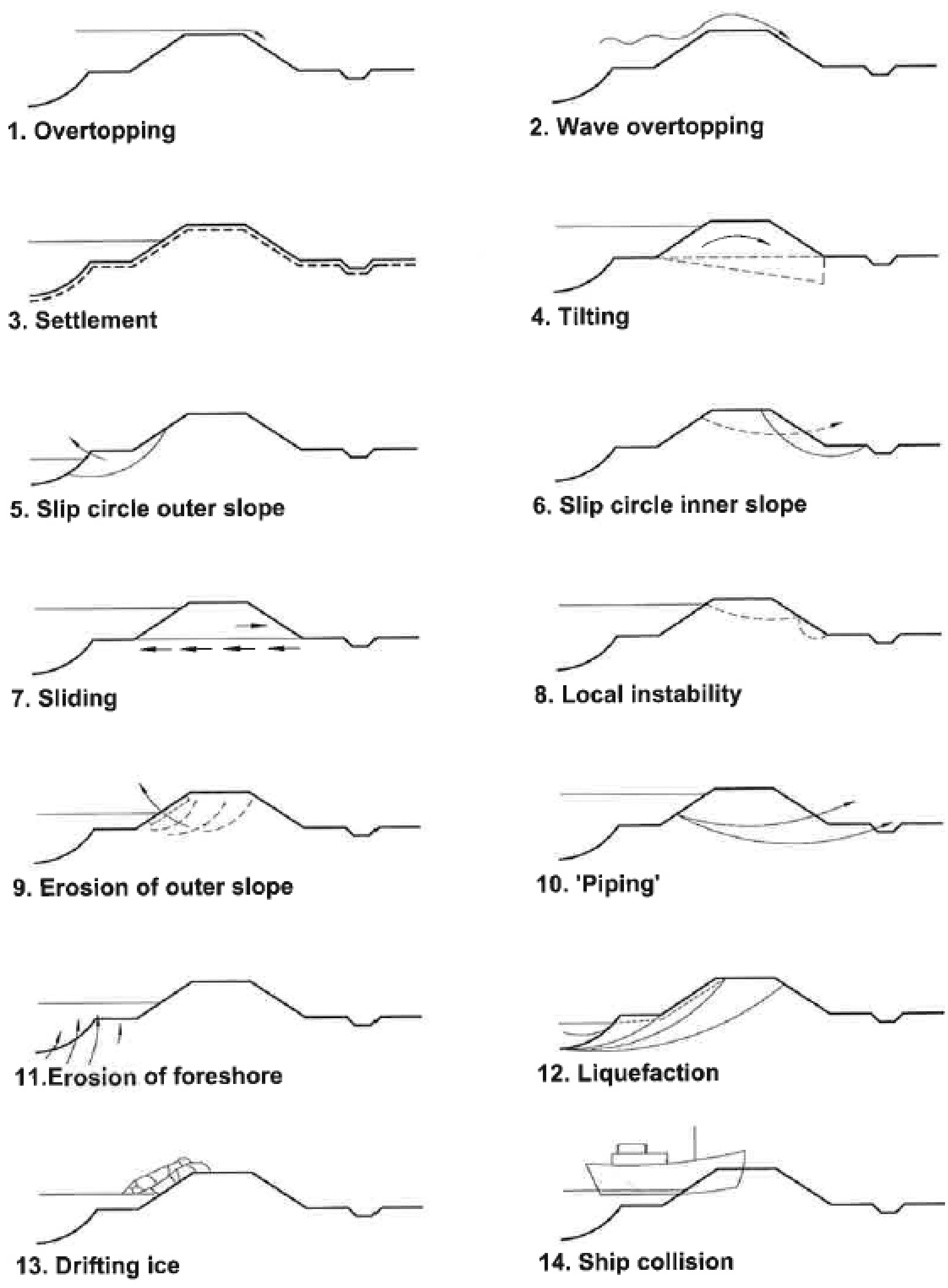

Figure 4.4 Typical failure modes for rock structures (CIRIA 2007)

4.4 Wave Run-up Considerations

Wave run-up was calculated by Barr (2001) as 9.5 ft (vertical elevation above the design SWL) based on a structure toe elevation of 597 ft (NAV 29), a design still water level of 604.13 ft (NAV 29) and a deep water wave (H_o) of 12.5 ft using methodology outlined in USACE (1984).

The same calculations were undertaken by Baird to verify these results, as shown in Table 4.8. The run-up was also calculated for the 50-year water level and for a larger design wave. A second method (CEM, 2006) was also used for comparison. More detailed calculations are available in Appendix B.

Table 4.8 Baird Run-up Calculations

Method	WL return Period	Depth at toe	H'_o	cotθ	Runup
USACE (1984)	10-year	7.1 ft	12.5 ft	2.5	10.1 ft
	50-year	7.7 ft	12.5 ft	2.5	10.5 ft
	50-year	7.7 ft	15 ft	2.5	10.5 ft
CEM (2006)	10-year	7.1 ft	12.5 ft	2.5	7.3 ft

The run-up calculated by Baird for the 10-year return period is 0.5 ft higher than the 9.5 ft reported in the Barr (2001) design memo. This is thought to be due to an error in Barr (2001) calculations where interpolation was done between Tables 7-8 and 7-9 instead of 7-9 and 7-10 (USACE, 1984). A method from CEM (2006) was also used to estimate 7.3 ft of wave run-up for comparison.

The crest of the revetment was set at 610.5 ft (IGLD85), which provides 6.4 ft of freeboard for the 10-year return period event; as the run-up exceeds the freeboard, wave overtopping will occur during the design event.

A difference of 0.5 ft of run-up elevation is not very significant in terms of revetment stability and the USACE (1984) method provides a conservative estimate compared to the CEM (2006) method which results in less wave run-up than was calculated by Barr (2001). However, the landward side of the revetment may experience higher overtopping volumes during large storm events. This area should therefore be designed to accommodate this additional overtopping, although it is anticipated the increase in water volume would only be in the order of 20%. Alternatively, the revetment crest could be raised (in the order of 0.5 ft) to mitigate the possible increase in overtopping volumes. This will obscure sight lines to the lake along Lake Shore Boulevard but it is not anticipated that this will have a substantial impact on recreational aspect of the site. Overtopping criteria and calculation results are presented in the next section of this report.

Barr (2001) designed the revetment crest width to provide improved access for large equipment. This will also have the effect of providing further protection to the landward facilities from wave runup and overtopping during large storm events. The landside of the revetment consists mainly of green space, with no future plans for development. Therefore, some wave run-up and overtopping can be tolerated. However, the area should be properly signed so that the public is aware that it is unsafe to be in the area during extreme weather events. Some maintenance may also be required in this area following large storm events. The USACE (1984) Shore Protection Manual (SPM) recommends a minimum revetment crest width of three armor units. Assuming a nominal armor unit size of 3 ft, the Barr (2001) design would yield four units on the crest for a total of 12 ft which is greater than recommended in USACE (1984).

4.5 Overtopping Considerations

Overtopping rates for the proposed revetment design were estimated using empirical equations found in CEM (2006); CIRIA (2007); and Goda (2010). The numerical model PC-Overtopping was also used as a point of reference to estimate overtopping rates. The key input variables for all methods are the incident wave conditions (taking into account the nearshore bathymetry) and the structure geometry (slope, freeboard, etc.).

The above-listed methods estimated overtopping rates between 0.05 and 0.75 $ft^3/s/ft$ for the 20/10 wave/water level design event (200-year return period storm) as reported in Barr (2001). Differences in the estimated rates are attributed to differences in the experimentally derived equations and constants. Equations and calculations can be found in Appendix C.

Several guidelines (Eurotop, 2007; CEM 2006; Goda, 2010; CIRIA, 2007) were consulted to assess the potential implications of the overtopping that can be expected with the combined 20-year design wave and the 10-year design water level used by Barr (2001). During the design event the following conditions may be expected in the vicinity of the revetment:

- It will likely be very dangerous for pedestrians. The public should not be in the area during these events. It is also unsafe for trained staff that are well shod and protected. Any person in the area should expect to get very wet. Signage should be erected warning the public of the overtopping danger during severe storm events.
- Limits for safe vehicular access will likely be exceeded. It will be unsafe for driving at moderate or high speeds because of "impulsive" wave overtopping in the form of falling or high velocity jets. However, with the realignment of Lake Shore Boulevard to a location further inland, this should not be a concern, as the effect of the overtopping should be restricted to the greenspace immediately adjacent to the revetment.
- Damage to a promenade may result if it is not paved. If walkways are planned adjacent to the revetment, they should be paved to avoid damage. Otherwise some maintenance of the pathways may be required. It should also be noted that soft landscaping or grassy slopes may be damaged by overtopping during the design event.
- At the higher end of the overtopping range (above 0.5 $f^3/s/ft$), damage to the revetment armor layer may occur.

The above overtopping criteria are applicable adjacent to the shoreline and some distance inland and provides an estimation of the effects of the design conditions which have a combined return period of 200 years.

The effect of wave run-up and overtopping on any proposed viewing platforms (or other structures in the vicinity) should be properly quantified prior to design of these structures. In addition, specific studies should be conducted to determine appropriate wave impact loads.

4.5.1 Overtopping Sensitivity Analysis

Baird was also asked to quantify the effect of the 20-, 50- and 100-year storms on overtopping. For this analysis the water levels were kept at the 10-year return period. Therefore, a 2-, 5- and 10-year significant deepwater wave height was required for these calculations (see Table 4.4). It should be noted that the 20-year wave from the Barr (2001) report is smaller than the 2-year wave from the updated WIS hindcast.

This sensitivity analysis focuses on the Goda (2010) and CIRIA (2007) methods. It should be noted that the nearshore wave transformation used by the PC Overtopping method yielded non-conservative results that were not considered accurate and were therefore not considered any further in the sensitivity analysis. The CEM (2006) method resulted in overtopping values an order of magnitude smaller than the other methods and was also not considered further. Table 4.9 shows the overtopping results of these two methods for the above stated storm return periods.

Table 4.9 Overtopping volumes for significant deepwater wave heights

Overtopping method	(Barr, 2001)	WIS (USACE, 2014)			
	20-yr	20-yr	10-yr	5-yr	2-yr
Goda, 2010	0.72 cfs/ft	0.90 cfs/ft	0.90 cfs/ft	0.83 cfs/ft	0.77 cfs/ft
CIRIA 2007	0.61 cfs/ft	0.69 cfs/ft	0.69 cfs/ft	0.66 cfs/ft	0.63 cfs/ft

The overtopping volumes shown in Table 4.9 can be expected for 2% of the waves impacting the revetment and can be related to damage/safety criteria as discussed above. The overtopping values do not significantly vary between storm return periods; the variation between empirical estimation methods is greater. This is due to the small difference in H_s at the structure because of depth limited conditions. It should be noted that these methods provide only an estimate and volumes are meant to be related to established damage/safety criteria above. The calculated increased overtopping rate of 0.90 cfs/ft does not change what conditions are expected in relation to published damage or safety criteria.

4.6 Stone Sizing and Cross-Section Details

4.6.1 Stone Sizing

The Barr (2001) report documents using Hudson's equation for armor sizing as described in the SPM (1984). The inputs to Hudson's equation and the calculation results are shown in Table 4.10 Detailed calculations are shown in Appendix D.

Table 4.10 Stone Sizing Calculations

Equation input/results	Barr 2001	Baird (head)	Baird (trunk)	Barr (2001) underlayer
w_r	165 pcf	165	165	N/A
H	7 ft	5.5 ft	5.5 ft	N/A
S_r	62.4	62.4	62.4	N/A
cotθ	2.5	2.5	2.5	N/A
K_D	1.45 (USACE, 1984)	2.25 (USACE, 1977)	3.5 (USACE, 1977)	N/A
W_{50}	1.8 tons	0.5 tons	0.4 tons	350 lbs
W range	1.3 – 2.2 tons	0.4 – 0.7 tons	0.2-0.5 tons	250-450 lbs
r (USACE, 1984)	5.5 ft	3.8 ft	3.2 ft	2.6 ft
r (Baird)	6.5 ft	4.3 ft	3.7 ft	3 ft
t_d (CIRIA, 2007)	5.1 ft	3.5 ft	3.0 ft	2.4 ft

It should be noted that the Barr (2001) calculations used the K_D value for the head from Table 7-8 in USACE (1984) as well as H_b (or H_{10}). For this type of design, Baird considers the 1984 K_D values to be overly conservative. Baird's experience with shallow water structures on the Great Lakes indicates that the design method presented in the 1977 Shore Protection Manual (SPM) is acceptable (USACE, 1977).

Generally, H_s is used as the design wave height in the calculation of armor stone sizing using Hudson's equation. It is noted that Barr (2001) used H_{10} per design guidance in the 1984 SPM. As noted above, Baird's experience indicates that this approach is overly conservative. Adjusting this input value from H_{10} to H_s per the 1977 SPM results in a much smaller armor size. Accordingly, the revetment design presented by Barr (2001) is likely quite conservative.

The revetment can be classified as "trunk" because there is no termination end directly exposed to head-on wave action as with a groyne or a breakwater. However, using "head" values at the revetment termination ends is justified.

The proposed revetment slope is relatively mild at 1H:2.5V, but was selected to help reduce wave run-up. This mild slope will also help with resistance to ice action, as discussed in Section 4.8.

Stable armor sizing calculations were undertaken by Baird with less conservative slopes of 1H:2V and 1H:1.5V. If the significant wave height (H_s) is used in conjunction with the SPM 1977 K_D values, the resulting armor size is still smaller than what was calculated by Barr (2001) (see Appendix D).

The Barr (2001) report identified the north to central part of the revetment as being more exposed based on bathymetric data. Therefore, there may be construction cost savings in differential armor stone design by reducing or matching armor stone gradations. However, this would require further engineering, and the construction cost savings may not significantly outweigh engineering costs.

The size of stone for the filter layer is appropriate in relation to the armor layer based on guidance found in CIRIA (2007) and USACE (1977, 1984).

4.6.2 Layer Thickness

Two layers of armor stone is typically specified for a conventional revetment shore protection structure. Barr (2001) calculated the armor layer thickness (r) by means of Table 7-12 (USACE, 1984), whereas the SPM 1977 r values are calculated with equation 7-120. Baird generally uses $2xD_{50}$, which is similar to the r calculated from the SPM 1977. The layer thickness (t_d) was also calculated based on CIRIA (2007), as presented in Table 4.7 (equations can be found in Appendix D).

There is a difference of between 0.5-1 ft between the value of ‘r’ calculated from SPM (1977) and SPM (1984). However, the Barr (2001) calculated layer thickness is conservative compared to the CIRIA (2007) criteria, which is considered state of the art in the use of rock in hydraulic engineering. Ultimately, the in-place armor layer thickness will depend on many factors, including properties of the sourced armor stone (density, gradation and shape), placement technique and construction tolerances.

The implications of the Barr (2001) design having a thicker theoretical armor layer thickness may be reflected in the material quantification, construction bidding, and construction review. However, this will not affect the structural integrity of the revetment cross-section. Depending on the Contractor’s methodology and materials source, larger (i.e. heavier) armor stone is often more difficult to produce, transport, and place properly.

The calculated layer thickness for the underlayer appears to be acceptable. However, the stone size could be noted on the drawings in order to provide clarity.

CIRIA (2007) also offers a method for calculating the designed bulk volume of rock required as shown in the following equation. The surface area (A) is illustrated in Figure 4.5. Volume calculations were not undertaken by Baird as part of this study as they were outside of the scope of work. However, it may be useful to undertake these calculations using the various methods described above (to calculate armor layer thicknesses) to determine potential variability in quantity estimates.

$$V_{b,d} = A\, t_d$$

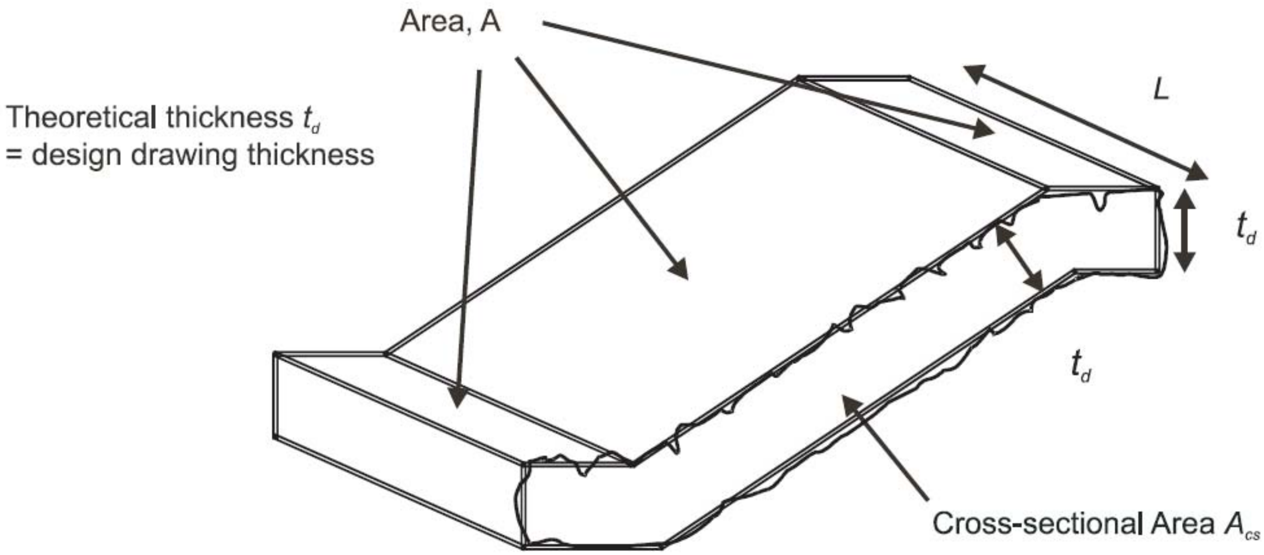

Figure 4.5 Armor layer geometry (Figure 3.25, CIRIA 2007)

Quality Assurance/Quality Control of armor stone production and placement is extremely important to the success of any rubblemound coastal structure project. QA/QC procedures should be well defined in construction specifications, and it is highly recommended that experienced professionals be involved in onsite administration and review throughout the construction process.

4.6.3 Revetment Toe

The width of a toe apron depends on the level of scour expected at a coastal structure. This is generally dependent on the magnitude of wave and current forcing, structure alignment and exposure, and bearing surface characteristics (erodibility).

The Barr (2001) toe design consists of an excavated trench in which the toe stone (as well as stone underlayer and geotextile bedding) are placed below existing grade. This type of toe arrangement corresponds to a configuration intended to resist "severe scour potential," per CIRIA (2007). This type of toe protection is both 'flexible' and able to resist migration of finer material through the toe due to the presence of the geotextile.

No information is available with regards to the subsurface conditions at the revetment toe. The existing revetment should be surveyed for any signs of scour. This could increase the size of the toe protection required.

The toe apron appears to be approximately five armor stones wide (14 ft) placed on top of filter and then backfilled with sand. The total depth of the toe scour apron is not shown on the typical cross-section. However, the depth is a minimum of 7.6 ft below grade, which is in excess of the design wave height (H_{10}=7 ft used in Barr, 2001) which corresponds to a "rule of thumb" for maximum scour depth at the toe of a structure. The toe width (5 x D_{50}) is also in excess of the recommended minimum of 3 x D_{50} found in most published criteria.

Considering the geometric configuration and size of stone, the Barr (2001) toe apron design appears to be conservative, despite the unknown lakebed characteristics at the toe of the structure.

4.6.4 Geotextile

Although a review of the construction specifications is beyond the scope of work, Baird cross-referenced the proposed geotextile specifications with past project experience and found them to be insufficient. The specification standards for geotextiles used in marine applications have changed in recent years to reflect knowledge gained from updated manufacturing processes and materials implementation experience. The following points should be considered when specifying the geotextile:

- The fiber should be polypropylene, not polyester (the latter can lose strength in water).
- UV resistance is typically 70%, not 50%.
- Seams may be sewn but 1-2 ft minimum overlap is more typical and seams should not be tied.
- Cushioning material is not generally required

Regarding installation of the geotextile, it is important that geotextile specifications state the following:

- The geotextile shall not extend to the outer edge of the apron and should stop approximately 3 feet from the edge to prevent undermining of the toe.
- The geotextile should be shown to terminate by hooking into the armor stone layer.
- A tie-in detail for the shore edge should also be included.

4.7 Alignment considerations

The alignment of the proposed revetment reconstruction generally matches that of the existing revetment. The revetment extends from a beach south of Wright Street to Hawley Street. The proposed revetment alignment consists of a series of straight segments to reconstruct the existing revetment rather than one single, smooth curve. While the segmented approach may be favorable from a construction standpoint, each transition between reaches will require special attention to stone placement.

The existing stone revetment appears to be reduced in elevation and width north of the "Drive to Bio-Resources." Baird notes that the north termination point of the revetment is located quite close the proposed roadway alignment. The proximity of the roadway, combined with the lack of existing shore protection and the potential of wave focusing at this location may offer little, if any, improvement over the existing conditions during storm events. Extending the revetment north may improve conditions, though would require additional engineering and design detail at the mouth of the existing stormwater outfall adjacent to Hawley Avenue. It is also noted that the bike path is located approximately 40 feet west of the revetment in this reach, and could be subject to flooding from overtopping of the revetment during large storm events (depending on drainage details).

4.8 Ice Considerations

Ice can damage a revetment in a number of ways, including:

- Thermal expansion of an ice sheet, potentially causing heave and displacement of armor stones placed on an exposed slope.
- Large moving ice sheets make contact with exposed structure slopes and 'pushing' (a few stones), or 'bulldozing' (large scale damage) armor stones, causing either localized or regional damage.
- Ice can adhere to and 'pluck' individual units from an armor slope. This can happen during periods of rapid water level increase or decrease.
- Ice may accumulate and 'ride up' a structure face, causing displacement not only to the structure slope, but also the crest.

There is limited design guidance for shoreline protection design in the presence of ice. To mitigate damage caused by ice, recommendations from CIRIA (2007), Ministry of Natural Resources (1996), and Sodhi and Donnelly (1999) were consulted. The relevant recommendations are provided below:

- For ice about 2.5 ft thick, a standard heavy grading for rock of 0.3-1.1 ton or greater should be used (CIRIA, 2007).
- Generally when there are significant water level changes and concerns over plucking out of individual stones, the median nominal stone diameter should exceed the maximum ice thickness (CIRIA, 2007).
- The surface of the armor stone needs to be relatively smooth and the armor stone layer should be well keyed.
- In the St. Lawrence River, it was inferred that 0.7 ton stone on a 1:3 slope exposed to 2 ft ice or 5 ft waves was too small and that 1.1 ton stone on a slope of 1:2 above the water line and 1:4 or flatter below the waterline was preferred (MNR, 1996).

- Rip rap failure takes place when ice thickness is equal to or thicker than the median stone size. Accepting some (15%) probability of rip rap failure, median stone size needs to be 2-3 times the ice thickness to protect a bank from ice (Sodhi and Donnelly, 1999).

There is evidence that ice has plucked stones from the current revetment. However, no site-specific information is available on the seasonal thickness of the ice or incidental ice damage events. Some ice thickness measurements are available from the NOAA Great Lakes Environmental Research Laboratory (GLERL, 2014) between December 1967 and March 1969 in the Lower Harbor (Figure 4.6). It should be noted that the ice thickness is located in a less exposed area and is not specific to the site of the proposed revetment. Other data constraints include the relatively short period of record and that the data set may not reflect the full winter severity range. The data is presented to provide an approximate ice thickness at the site. However, ice thickness varies spatially and seasonally. Over the measurement period, measured ice thickness reached a maximum of 14 inches.

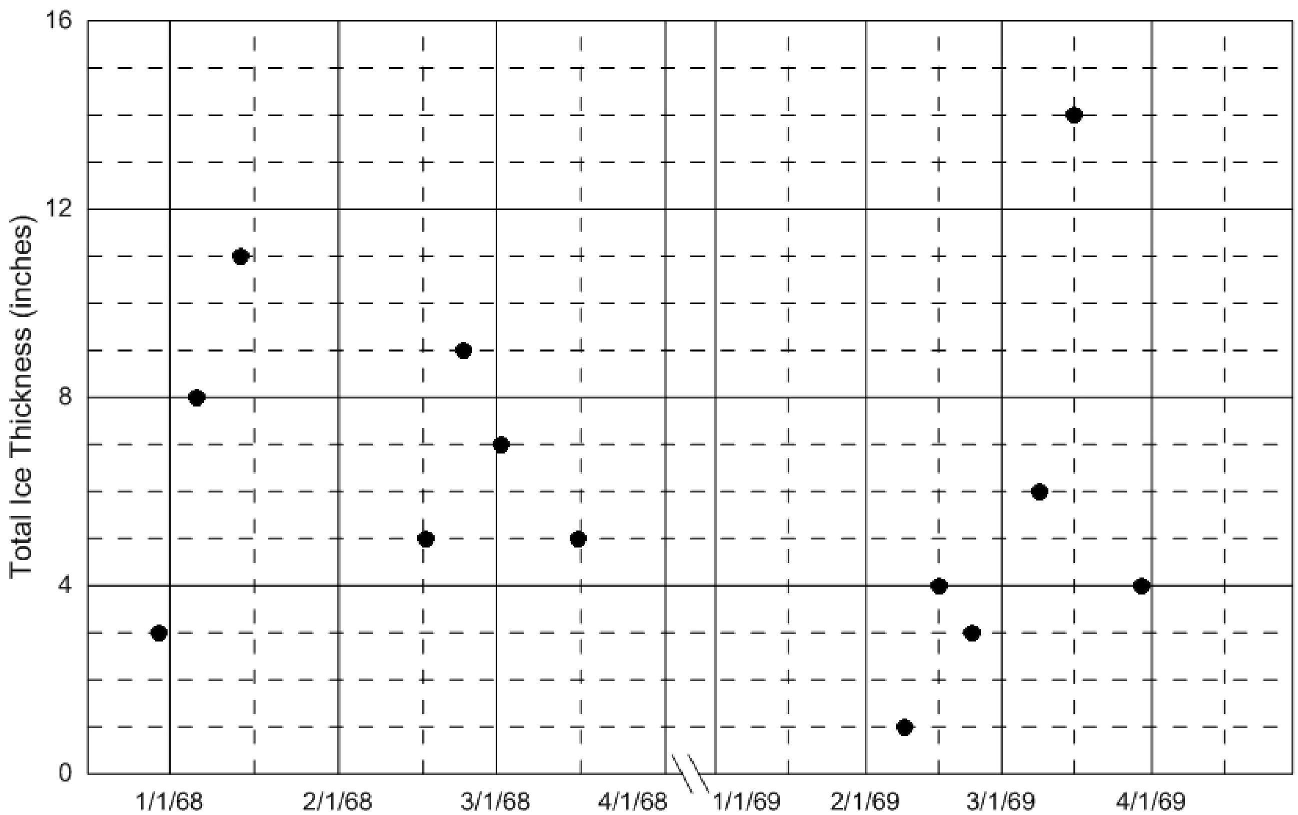

Figure 4.6 Ice thickness measurements

Based on the above recommendations, the Barr (2001) armor layer as designed should provide resistance to ice thicknesses of approximately 2 ft as W is 1.5 ton and is larger than what was found to be stable by MNR (1996) since the design wave conditions are relatively similar. The slope of the Lake Shore Boulevard revetment is steeper below the waterline but flatter above the waterline.

Placement of the armor stone to provide the least amount of roughness will improve the stability of the armor stone against ice forces. Special attention to armor stone placement during construction should be undertaken as well placed armor units have better contact with adjacent stones, reduced void spacing, and a higher level of interlock.

5.0 SUMMARY AND CONCLUSIONS

5.1 Design Conditions

5.1.1 Basis of Design

The basis of design is not explicitly described in the Barr (2001) report. However, the combined 20-year wave and 10-year water levels (which produces a nominal 200-year return period event) were used for design calculations. It should be noted that although the 20-10 (or 10-20) combined waves and water levels are the USACE standard design practice for coastal structures on the Great Lakes, other combinations of wave and water level resulting in the same return period were not discussed by Barr (2001).

Background information on the determination of the design conditions is provided. Because the site is depth limited, the significant wave height at the structure will not significantly change. The variance in water levels does not increase significantly with increasing return period.

5.1.2 Water Levels

The 10-year water level was selected for design (Barr, 2001). Water levels for various return periods are shown in the Baird (2013) and Barr (2001) reports and both originate from the same source. However, the Baird 10-year water level is reported to be 0.1 ft higher than in the Barr report. The difference is not significant and could either be attributed to rounding error or an updated version of the source data.

Two datums were used in the Barr (2001) report. The NAV 29 datum is listed on the drawings while the IGLD 85 datum is used in the water level analysis. The conversion needs to be interpolated from nearby sites where it has already been calculated. However, taking the most conservative conversion gives a difference of zero.

A full bathymetric survey prior to construction to confirm the elevation of the lakebed at the toe of the proposed revetment is important because the design wave height is depth limited. Therefore, the water depth at the toe of the revetment controls the height of the design wave.

5.1.3 Waves

The waves at the structure are depth limited. Therefore, there will be no significant change in the size of the significant design wave height at the structure if the wave return period is increased. The 20-year wave was used for design and originated from the WIS database. However, the WIS database has since been updated along with extreme event return periods. The 20-year wave is now 18.7 ft. However, when transformed into shore, H_s is not significant due to the depth limited conditions.

5.1.4 Existing Conditions

No information regarding the properties of the nearshore sediment or the thickness of the sediment layer. Jet probes are recommended to determine the type and thickness of the sediment layer.

5.2 Design Calculations

5.2.1 Wave Run-up

Wave run-up calculations were reviewed. An interpolation error was noted and the revised run-up level at the most exposed location on the revetment may increase by approximately 0.5 ft. However, the revetment is designed to accommodate some overtopping and this increase is not anticipated to be significant.

The Barr (2001) design specifies a 12 ft wide crest width which is equivalent to 4 armor stones rather than the 3 recommended in USACE (1984). This should add conservatism to the design by helping to reduce overtopping of the revetment.

5.2.2 Overtopping

No overtopping calculations are presented in the Barr (2001) report. Baird used several empirical methods to estimate the amount of overtopping to be expected during a design event and to correlate with published damage and safety thresholds. The results show that the design event could result in overtopping rates in the range of 0.05 and 0.75 $ft^3/s/ft$. The upper end of the overtopping rate is close to published criteria for the initiation of damage for the type of stone revetment being reviewed. Monitoring of the revetment front slope, crest and areas immediately landside of the revetment should be performed following large storm events. Some maintenance may be required after severe events. All nearby (exposed) promenades and pathways should be paved. Signage should be erected notifying the public that the area is unsafe during severe events.

In addition, any planned viewing platforms near or overlooking the revetment may be subject to wave loading. Further investigations would be required to determine the uplift and horizontal wave loading on these features for design purposes.

5.2.3 Armor Stone Sizing

The armor stone size was calculated (Barr, 2001) using Hudson's equation and coefficient values from USACE (1984) as well as a breaking wave height (H_b) equivalent to H_{10}. Baird reviewed the calculations and provided the following observations:

- The revised K_D values defined in USACE (1984) are considered by Baird to be overly conservative. Baird recommends using the K_D values found in USACE (1977).

- The Barr (2001) calculations used $H_b = H_{10} = 7.0$ ft (USACE 1984), while USACE (1977) calls the use of $H_s = 5.5$ ft. The H and K_D values used in the Barr (2001) design are both conservative and may result in an overly conservative cross-section design.
- The magnitude of the design wave varies over the length of the revetment. Therefore, there may be opportunities for construction cost savings through the application of smaller or more economically available stone over the length of the structure. This would require further engineering and the construction savings may not significantly outweigh engineering costs.
- The armor layer thickness was verified with USACE (1977, 1984) as well as with CIRIA (2007) criteria and was found to be conservative. This could possibly lead to over-estimation in quantity calculations, possibly leading to higher bids during the tender process. CIRIA (2007) provides a state of the art method for calculating the designed bulk volume of rock for the armor layer.
- The range of armor stone weight and the corresponding filter stone weight proposed by Barr (2001) were found to be appropriate.

The dimensions of the revetment toe were found to be adequate based on published criteria and Baird's past experience. Some concerns over the specification of geotextile properties were raised and should be addressed prior to project tendering.

5.2.4 Alignment Considerations

The proposed revetment alignment consists of a series of straight segments to reconstruct the existing revetment rather than one single, smooth curve. Straight segments may be beneficial for construction and the transition between segments should not be of concern with regards to stone placement if they are not abrupt.

The north end of the revetment is located quite close to the road. The lack of adjacent shore protection may offer little if any improvement over the existing conditions during storm events at this location.

It should also be noted that the bike path is located approximately 40 feet west of the revetment in the reach, and could be subject to flooding from overtopping of the revetment during large storm events. Proper design of drainage systems will be required over the length of the project.

5.2.5 Ice considerations

There is no site-specific ice data available. However, a small amount of historical measurements are presented for the lower harbor. Published criteria applicable to the design of stone revetments were examined in the context of the Barr (2001) revetment design. Due to the structure's mild slope and conservative armor size, the design appears to be adequate to resist ice loading. The armor stone should be placed in such a way to produce a relatively smooth slope surface in order to maximize unit interlock and minimize armor unit plucking from ice.

5.2.6 Construction Drawings

Several areas for additional detail were noted on the current drawings, including:

- Revetment and swale termination details;
- Revetment landside detail including dune/swale interface and overlooks;
- Specifying the stone size for both the under layers and primary armor layers;
- Details of the pipe, detention basin, and outfall; and
- Demolition details or specifications along with the staging for construction.

In addition, stone production and placement specifications should be developed including suitable QA/QC measures.

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APPENDIX A

Wave Shoaling Calculations

WAVE SHOALING CALCULATIONS

Interpolation of wave heights at Wright St:

from Wave Analysis			
Incident wave height	Significant Deep Water Wave (ft)		
	at Hawley St	at Fair Ave	at Wright st
Angle class 2 with diffraction	2.8	14.7	9.4
Angle class 1 with no diffraction	12.5	12.5	
approximate distance (m)	0	1800	1000

Shoaling of deepwater wave into shore:

Angle class	Deep water Hs		slope	Period	depth at structure		Hs (Goda, 1970)	
	ft	m		s	ft	m	m	ft
1	12.5	3.81	0.025	7	7.13	2.17	1.6	5.3
2	18.4	5.61	0.033	7	7.13	2.17	1.8	5.8
3	14.8	4.51	0.033	7	7.13	2.17	1.7	5.6
1	12.5	3.81	0.033	7	7.13	2.17	1.7	5.5
average slopes	12.5	3.81	0.018	7	7.13	2.17	1.6	5.2
average slopes	12.5	3.81	0.027	7	7.13	2.17	1.6	5.4
average slopes	12.5	3.81	0.037	7	7.13	2.17	1.7	5.6
2 with diffraction	9.4	2.87	0.033	7	7.13	2.17	1.6	5.3
WIS 95077	18.7	5.70	0.033	7	7.13	2.17	1.8	5.8

APPENDIX B

Wave Run-up Calculations

WAVE RUN-UP CALCULATIONS

From Chapter 7, Section II of the USACE (1984)

d_s=	7.13	7.7	7.7	ft		
H'_o=	12.5	12.5	15	ft		
	3.81	3.81	4.572	m		
g=	9.81	9.81	9.81	m/s^2		
T=	7	7	7	s		
cotθ=	2.5	2.5	2.5			
d_s/H'_o=	0.5704	0.616	0.513333			
$H'_o/(g*T^2)$=	0.007926	0.007926	0.009511			
R/H'_o=	1.13	1.13	1.015115	from Figure 7-9	d_s/H'_o=	0.45
R/H'_o=	1.51	1.51	1.407957	from Figure 7-10	d_s/H'_o=	0.8
$R/H'_{o(smooth)}$=	1.26072	1.310229	1.086201	interpolated		
r=	0.55	0.55	0.55	from Table 7-2		
$R/H'_{o(rough)}$=	0.693396	0.720626	0.597411			
$R_{(uncorrected)}$=	8.66745	9.007821	8.961159	ft		
k=	1.169	1.169	1.169	scaled correction factor from Figure 7-13		
R=	10.13225	10.53014	10.47559	ft		

Also calculated from CEM (2006) – calculation are in metric as published. Results are converted to imperial.

$$\xi_o = \frac{\tan \alpha}{\sqrt{s_o}}$$

where

α = slope angle

s_o = deepwater wave steepness (= H_o/L_o)

H_o = deepwater wave height

L_o = deepwater wavelength (= $gT^2/2\pi$)

T = wave period

g = acceleration due to gravity

tan α =	0.4	
s_o =	0.049801	
H_o =	3.81	m
L_o =	76.50419	m
T =	7	s
g =	9.81	m/s2
ξ_o=	1.792421	plunging

$$\frac{R_{ui\%}}{H_s} = \left(A\xi + C\right)\gamma_r \gamma_b \gamma_h \gamma_\beta \qquad \text{(VI-5-3)}$$

where

$R_{ui\%}$ = runup level exceeded by i percent of the incident waves

ξ = surf-similarity parameter, ξ_{om} or ξ_{op}

A, C = coefficients dependent on ξ and i but related to the reference case of a smooth, straight impermeable slope, long-crested head-on waves and Rayleigh-distributed wave heights

γ_r = reduction factor for influence of surface roughness ($\gamma_r = 1$ for smooth slopes)

γ_b = reduction factor for influence of a berm ($\gamma_b = 1$ for non-bermed profiles)

γ_h = reduction factor for influence of shallow-water conditions where the wave height distribution deviates from the Rayleigh distribution ($\gamma_h = 1$ for Rayleigh distributed waves)

γ_β = factor for influence of angle of incidence β of the waves ($\gamma_\beta = 1$ for head-on long-crested waves, i.e., $\beta = 0°$). The influence of directional spreading in short-crested waves is included in γ_β as well

Table VI-5-3
Surface Roughness Reduction Factor γ_r in Equation VI-5-3, Valid for 1 < ξ_{op} < 3-4

Type of Slope Surface			γ_r
Smooth, concrete, asphalt			1.0
Smooth block revetment			1.0
Grass (3 cm length)			0.90 - 1.0
1 layer of rock, diameter D, (H_s/D = 1.5 - 3.0)			0.55 - 0.6
2 or more layers of rock, (H_s/D = 1.5 - 6.0)			0.50 - 0.55
Roughness elements on smooth surface (length parallel to waterline = ℓ, width = b, height = h)			
Quadratic blocks, $\ell = b$			
h/b	b/H_s	area coverage	
0.88	0.12 - 0.19	1/9	0.70 - 0.75
0.88	0.12 - 0.24	1/25	0.75 - 0.85
0.44	0.12 - 0.24	1/25	0.85 - 0.95
0.88	0.12 - 0.18	1/25 (above SWL)	0.85 - 0.95
0.18	0.55 - 1.10	1/4	0.75 - 0.85
Ribs			
1.00	0.12 - 0.19	1/7.5	0.60 - 0.70

Ahrens (1981) for slopes between 1:1 and 1:4 for $\xi_{op} > 1.2$

$R_{u2\%}$

A = 1.6

C = 0

σ_{Ru}/R_u = 0.15

R_{us}		
A =	1.35	
C =	0	
σ_{Ru}/R_u =	0.1	
H_s/D=	1.98	
Υ_r=	0.55	
$R_{u2\%}$=	2.6	m
$R_{u2\%}$=	8.7	ft
with		
σ_{Ru}=	10.0	ft
R_{us}=	2.2	m
R_{us}=	7.3	ft

APPENDIX C

Overtopping Calculations

OVERTOPPING CALCULATIONS

Several methods were used.

Goda (2010)

$$\frac{q}{\sqrt{g\left(H_{1/3}\right)_{toe}^{3}}} = q^* = \exp\left\{-\left[A + B\frac{h_c}{\left(H_{1/3}\right)_{toe}}\right]\right\}$$

$$A = A_0 \tanh\left\{(0.956 + 4.44s) \times \left[h/\left(H_{1/3}\right)_{toe} + 1.242 - 2.032s^{0.25}\right]\right\}$$

$$B = B_0 \tanh\left\{(0.822 + 2.22s) \times \left[h/\left(H_{1/3}\right)_{toe} + 0.578 - 2.22s\right]\right\}$$

$$A_0 = 3.4 - 0.734\cot\alpha_s + 0.239\cot^2\alpha_s - 0.0162\cot^3\alpha_s$$

$$B_0 = 2.3 - 0.5\cot\alpha_s + 0.15\cot^2\alpha_s - 0.011\cot^3\alpha_s$$

	Barr (2001)	WIS station 95077 (USACE, 2014)					
	20yr	20yr	20yr	10yr	5yr	2yr	
g=	9.81	9.81	9.81	9.81	9.81	9.81	m/s^2
$H_{1/3}$=	5.5	5.5	5.8	5.8	5.7	5.6	ft
	1.68	1.68	1.77	1.77	1.74	1.71	m
h=	7.13	7.13	7.13	7.13	7.13	7.13	ft
	2.17	2.17	2.17	2.17	2.17	2.17	m
cot α_s=	2.5	2.5	2.5	2.5	2.5	2.5	
s=	0.05	0.02	0.05	0.05	0.05	0.05	
h_c=	6.37	6.37	6.37	6.37	6.37	6.37	ft
	1.94	1.94	1.94	1.94	1.94	1.94	m
A_0=	2.81	2.81	2.81	2.81	2.81	2.81	
B_0=	1.82	1.82	1.82	1.82	1.82	1.82	
A=	2.67	2.67	2.65	2.65	2.66	2.67	
B=	1.69	1.67	1.67	1.67	1.67	1.68	
q*=	0.0098	0.0100	0.0113	0.0113	0.0108	0.0103	
q=	0.0667	0.0680	0.0832	0.0832	0.0774	0.0719	$m^3/s/m$
	67	68	83	83	77	72	L/s/m
	0.72	0.73	0.90	0.90	0.83	0.77	cfs/ft

CIRIA (2007) – Owen's Method

- **Owen's method (1980)**

To calculate the time-averaged overtopping discharge for smooth slopes, the dimensionless freeboard, R^* (-), and the dimensionless specific discharge, Q^* (-), were defined by Owen (1980) with the Equations 5.28 and 5.29, using the mean wave period, T_m (s), and the significant wave height at the toe of the structure, H_s (m):

$$R^* = R_c / \left(T_m \sqrt{gH_s}\right) = R_c / H_s \sqrt{s_{om}/2\pi} \tag{5.28}$$

$$Q^* = q / \left(T_m \, gH_s\right) \tag{5.29}$$

where R_c is the elevation of the crest above SWL (m); s_{om} is the fictitious wave steepness based on T_m (see Equation 5.1), q is the **average specific** overtopping discharge (m³/s per m).

Equation 5.30 gives the relationship between the non-dimensional parameters defined in Equations 5.28 and 5.29:

$$Q^* = a \exp\left(-b R^* / \gamma_f\right) \tag{5.30}$$

where a and b are empirically derived coefficients that depend on the profile and γ_f is the correction factor for the influence of the slope roughness, similar to that used to calculate wave run-up (see Section 5.1.1.2).

	(Barr 2001)	WIS station 95077 (USACE, 2014)						
inputs	20yr	20yr	10yr	5yr	2yr		From Table 5.5	
Rc=	1.94					m	a=	0.0103
Tm=	7					s	b=	24.5
g=	9.81					m/s2	From Table 5.2	
Hs=	1.68	1.77	1.77	1.74	1.71	m	Υ_f=	0.55
Calculations								
R*=	0.0684	0.0666	0.0666	0.0672	0.0678	N/A		
Q*=	0.0005	0.0005	0.0005	0.0005	0.0005			
q=	0.0563	0.0643	0.0643	0.0616	0.0590	m3/s/m		
	56	64	64	62	59	l/s/m		
	0.61	0.69	0.69	0.66	0.63	cfs/ft		

CEM (2006)

Table VI-5-11
Overtopping Formula by van der Meer and Janssen (1995)

Straight and bermed impermeable slopes including influence of surface roughness, shallow foreshore, oblique, and short-crested waves, Figures VI-5-14a and VI-5-14b.

$\underline{\xi_{op} < 2}$

$$\frac{q}{\sqrt{g\,H_s^3}}\sqrt{\frac{s_{op}}{\tan\alpha}} = 0.06\,exp\left(-5.2\,\frac{R_c}{H_s}\,\frac{\sqrt{s_{op}}}{\tan\alpha}\,\frac{1}{\gamma_r\,\gamma_b\,\gamma_h\,\gamma_\beta}\right) \qquad (VI-5-24)$$

application range: $0.3 < \frac{R_c}{H_s}\,\frac{\sqrt{s_{op}}}{\tan\alpha}\,\frac{1}{\gamma_r\,\gamma_b\,\gamma_h\,\gamma_\beta} < 2$

Uncertainty: Standard deviation of factor 5.2 is $\sigma = 0.55$ (See Figure VI-5-15).

$\underline{\xi_{op} > 2}$

$$\frac{q}{\sqrt{g\,H_s^3}} = 0.2\,exp\left(-2.6\,\frac{R_c}{H_s}\,\frac{1}{\gamma_r\,\gamma_b\,\gamma_h\,\gamma_\beta}\right) \qquad (VI-5-25)$$

Uncertainty: Standard deviation of factor 2.6 is $\sigma = 0.35$ (See Figure VI-5-15).

The reduction factors references are

γ_r Table VI-5-3

γ_b Eq VI-5-8

γ_h Eq VI-5-10

$$\left.\begin{array}{l} \text{Short-crested waves} \\ \gamma_\beta = 1 - 0.0033\,\beta \\ \text{Long-crested waves (swell)} \\ \gamma_\beta = \begin{cases} 1.0 & \text{for } 0^\circ \le \beta \le 10^\circ \\ \cos^2(\beta - 10^\circ) & \text{for } 10^\circ < \beta \le 50^\circ \\ 0.6 & \text{for } \beta > 50^\circ \end{cases} \end{array}\right\} \qquad (VI-5-26)$$

The minimum value of any combination of the γ-factors is 0.5.

R_c=	1.94	m
H_s=	1.68	m
R_c/H_s =	1.16	
application range =	1.17	
q=	0.0026	m3/s/m
	2.6	l/s/m
	0.03	cfs/ft

PC Overtopping

Wave Overtopping

Calculation Tool

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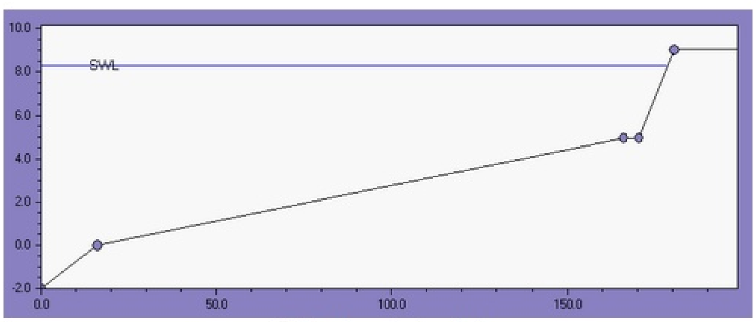
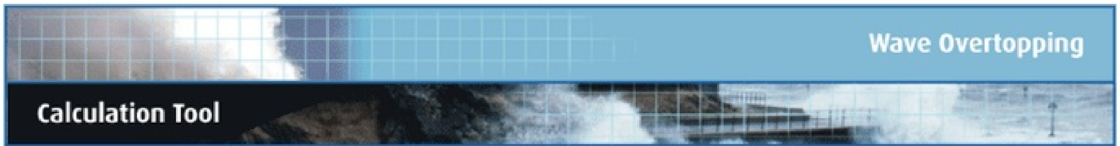
parameter	symbol	value	unit
Significant wave height	Hm0	3.810	[m]
Wave direction	β	90.000	°
Storm duration	tsm	440000.0	[s]
Water level	SWL	8.300	[m]
Average period	Tm	7.000	[s]
Spectral wave period	◉ Tm-1,0	7.000	[s]
Spectral peak period	○ Tp	7.700	[s]

calculate...



☐ Transform to standard

Section	X begin	Y begin	X end	Y end	Slope (tan)	Material	Roughness factor		
1	0.000	-2.000	16.000	0.000	0.125	Own input	1.000		
2	16.000	0.000	166.000	4.950	0.033	Fine granular materials - sand/gravel packed in geotextile	0.900		
3	166.000	4.950	170.300	4.950	0.000	Natural rubble mound of rock	0.550		
4	170.300	4.950	180.550	9.050	0.400	Natural rubble mound of rock	0.550		



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Calculated parameters	Average value
2% wave runup	2.362
2% wave runup	85.612
Percentage of overtopping	67.409

Overtopping[l/s/m]	Crest height[m]
0.100	0.000
50.000	8.300
60.000	8.300
100.000	8.300

Percentage[%]	Amount[l/m]
Number of waves	42371
Vmax	17508
1.000%	> Vmax
10.000%	> Vmax
50.000%	4895.718
100.000%	0.000

Interim results of calculation	
Calculation results	
Ru2PERC	2.362 [m]
Ru2PERC+SWL	10.662 [m]
Overtopping	85.612 [l/s/m]
V max	17507.520 [l/wave/m]
Comment	De 2%-golfoploop is hoger dan de dijk
Calculation of cross section	
Ru2PERC	2.362 [m]
Overtopping	85.612 [l/s/m]
Hm0	1.675 [m]
Tm0	5.715 [s]
Ksio	2.207 [-]
L0	50.985 [m]
GammaB	1.000 [-]
GammaF	0.572 [-]
GBeta run up	0.824 [-]
GBeta overtopping	0.736 [-]
Waterlevel	10.300 [m]
TanAlpha	0.400
Iterations	3

APPENDIX D

Armor Size Calculations

ARMOR SIZE CALCULATIONS

From Chapter 7, Section III of SPM (1977, 1984):

	units	head SMP 1984 H10	head SMP 1984 H10	head SMP 1977 H10	head SMP 1977 Hs	trunk SMP 1977 Hs	trunk SMP 1977 Hs	trunk SMP 1977 Hs	head SMP 1977 Hs	Underlayer
w_r	pcf	165	165	165	165	165	165	165	165	165
H	ft	7	7	7	5.5	5.5	5.5	5.5	5.5	
W_w	pcf	62.4	62.4	62.4	62.4	62.4	62.4	62.4	62.4	62.4
S_r		2.64	2.64	2.64	2.64	2.64	2.64	2.64	2.64	
cotθ		2.5	2.5	2.5	2.5	2.5	2	1.5	1.5	
K_D		1.45	1.45	2.25	2.25	3.5	3.5	3.5	2.9	
W	lbs	3512	3512	2263	1098	706	882	1176	1420	351
W	tons	1.8	1.8	1.1	0.5	0.4	0.4	0.6	0.7	0.2
W (range)	tons	1.32	1.32	0.85	0.41	0.26	0.33	0.44	0.53	0.132
		2.20	2.20	1.41	0.69	0.44	0.55	0.74	0.89	0.220
r (SPM 1984)	ft		5.5	4.8	3.8	3.2	3.5	3.8	4.1	2.6
r (SPM 1977)	ft		6.4	5.5	4.3	3.7	4.0	4.4	4.7	3.0
*D_{50}	ft		2.8	2.4	1.9	1.6	1.7	1.9	2.0	1.3
*D (range)	inches	**34.3	30.2	26.1	20.5	17.7	19.1	21.0	22.3	14.0
		**41.1	35.8	30.9	24.3	21.0	22.6	24.9	26.5	16.6
t_d (CIRIA, 2007)	ft		5.1	4.4	3.5	3.0	3.2	3.5	3.8	2.4

*Calculated

**From Table 7-12

Layer thickness calculations from CIRIA (2007)

$$D_{50} = \left(\frac{W}{w_r}\right)^{1/3}$$

$$t_d = n\, k_t\, D_{50}$$

Suggested k_t value for irregular rock is 0.92 (from Table 3.9, CIRIA, 2007)