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Executive Summary

Subject: Elder/Centennial Park Shoreline Improvements
Planting Pocket Habitat, Beach Sand Nourishment Protection, & Breakwater Structure Design

The Winnetka Park District (WPD) plans to improve and conjoin Elder and Centennial Park and the adjacent beach areas into one combined park. The project will resolve access, erosion and structural damages in the beach areas that have occurred over time and will provide a significant upgrade to the community park system in Winnetka.

Other portions of the permit application for this project will outline the existing deficiencies at the project site and describe portions of the stone and steel breakwaters proposed for the project. This report and supporting calculations focus largely on the steel portions of the proposed breakwaters. It will demonstrate the included steel wave baffle structures will provide protection for the beach and planting pocket areas; will reduce the potential for beach erosion during storm events; and show that the steel portions of the breakwater system are appropriately structurally engineered and robust enough to withstand Lake Michigan wind, wave, and ice events without exceeding material limits for allowable stress or deformation.

- The project will include an expansion of beach size and character by replacing the existing Lake Michigan coastal structures. Details are provided in the permit application.
- The steel portions of the breakwater structure (“baffle structure”) will be comprised of both steel sheet piles (“piles”), steel caps (“caps”) and panelized vertical steel plate wave louvers (“louvers”) designed to manage wave attack and minimize overtopping damage.
- The louvers will increase the stability of the beach and sand retention by reducing wave transmission and turbulence. The additional amount of retained sand on the lakebed will increase habitat.
- The planting pocket zones, which are shielded by the landward 100’ of louvers, will experience a reduced amount of stress and damage because the louvers will reduce wave overtopping and runup in these areas. Sand entrained waves will strike the baffle structures and reduce impact potential by reducing wave overtopping flows and also by reflecting some of these flows back to the lake.
- The baffle structures will provide additional protection within the beach cell areas that are protected by the breakwaters. The louvers alone will reduce breakwater wave overtopping potential by approximately 9% at the north breakwater and by approximately 4% at the south breakwater, when compared to the armor stone breakwater areas that don’t include these structures. The reduced flows through the louvers will reduce the amount of wave energy that reaches shore, thus helping retain sand within the beach cells and reducing the amount of stress at the planting areas.
- These baffle structures are to be constructed as part of and on the crest of the breakwaters and provide the amount of overtopping reduction indicated above without increasing the footprint of the breakwater structures on the lake bottom. The composite design provides for wave overtopping control in a compact fashion that minimizes the lake bottom fill when compared to an armor stone breakwater constructed at higher elevations.
- The baffle structures have been designed to withstand wave attack from a 200-year recurrence interval wave storm. Several combinations of lake level and wave height recurrence intervals were considered in the design. The baffle structures are comprised of a steel sheet pile wall and vertical steel plate louvers mounted on top of the steel sheet pile cap. They are designed to withstand the

wave loads that the structure will be exposed to. The louvers will be structurally connected to the steel sheet pile and designed to withstand loads imparted by wave storms. The base of the structure will be a wide flange steel sheet pile wall driven into the ground and buttressed on both sides by the armor stone breakwater.

Project Understanding

The Winnetka Park District (WPD) plans to improve Elder and Centennial Beaches as shown on the following rendering:

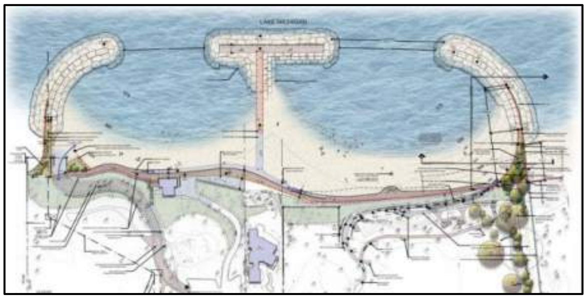


Figure 1. Proposed Beach Plan

Steel Baffle structures are proposed to be placed as a vertical extension of the armor stone breakwaters at the west end of the North and South breakwater structures. These baffle structures will provide a vertical extension of the breakwater crest to provide protection against wave overtopping that would otherwise cause beach erosion and damage to proposed planted habitat. The wave baffle concept includes the following features:

- A steel sheet pile wall base and attached vertical steel plate wave louvers that extend the vertical height of the breakwater to reduce wave overtopping potential. This concept allows for added wave protection without a large increase in the footprint of the armor stone breakwater on the lakebed.
- The reduction in wave overtopping and transmission will reduce potential for beach erosion. It will also reduce the potential for wave overtopping and transmission that would otherwise approach the proposed planting beds along the shoreline at the west end of the breakwater structures.
- The louvers are angled at 40 degrees with a 4” spacing providing views to the lake from shore while also reducing wave overtopping and transmission.

This report summarizes the following engineering studies performed for the baffle structures:

- Wave transformation analysis from deep water proceeding to the design site quantifies wave characteristics and attack stress that approach and impact the baffle structures.
- Structural analysis guides the baffle structure design.
- Breakwater system overtopping analysis for the design storm events provides information needed to design the baffle structures. The design seeks to reduce wave overtopping and runup in areas where waves would cause beach erosion and approach the planted habitat areas.

The baffle structures are designed to reduce beach erosion and to protect proposed plant habitat.

Wave Transformation Analysis

Wave transformation analysis includes an evaluation of the design wave condition at the project site. This analysis is completed in steps as follows:

- The analysis begins with a determination of offshore deep water wave storm conditions. The Corps of Engineers Wave Information Study Site No. 94027 provides deep-water wave information offshore from the project site. The deep-water location provides sufficient depth that the wave character is not significantly influenced by the lakebed. This information includes a wave height data base for the period of record from 1960 to 2014. Wave height recurrence intervals are determined for several directions of attack to the design site. Wave frequency analysis is performed for the historic wave record to evaluate wave height recurrence intervals from a variety of directions.

For this project, the directions of interest are from the North-Northeast, East, and Southeast. Waves from the NNE are likely to provide the greatest amount of stress for the North Breakwater and from the Southeast are likely to provide the greatest stress on the south breakwater. Waves from the East will travel directly towards shore approximately perpendicular to the beach. While we don’t anticipate that this third wave case will control the design, it is included in the analysis to be sure.

The analysis evaluates wave attack from three directions. Waves from the NNE (Class 3) are the largest waves. We also analyze waves approaching from the East (Class 2) and waves approaching from the SE (Class1). Class 1 and Class 3 waves will refract to shore and approach the design site at an angle. Class 2 waves approach shore with only minor refraction and generally travel in a westerly direction. The controlling case for the North breakwater will be the Class 3 wave and for the South breakwater will be the Class 1 wave. The wave transformation analysis provides the wave characteristics approaching the breakwater structures including the angle of attack, wave height, wavelength, wave period and other factors that influence the wave stress that reaches the proposed breakwater structures.

The analysis considers a 200-year wind wave storm. Wave storm recurrence intervals consider the combined probability of lake levels and wave heights. The analysis considers two different combinations of these factors: a) a storm with a 20-year lake level and a10-year wave height, and b) a storm with a 10-year lake level and a 20-year wave height. Both events produce a 200-year wave event. Both are evaluated to assess which produces the worst-case condition in terms of wave stress on the breakwater baffle structures. The wave transformation analysis considers the following wave influences:

- Wave refraction causes the waves to turn from their deep-water direction of travel towards shore. This phenomenon is due to the influence of the gradually shallower lakebed and increased bottom friction on the side of the wave that is closest to shore. This analysis provides an indication of the angle at which the wave will approach the breakwater. Waves that approach the breakwater perpendicular to the structure cause a greater stress compared to waves that hit the breakwater at an angle.
- As a wave approaches shore, the water depth is gradually reduced. This causes the wave to break and reform at smaller wave heights. This process occurs gradually as the wave approaches shore. A surf zone analysis was completed to evaluate the transformation of the wave as it progresses through the surf zone to the design site. The analysis provides an estimate of the wave height and other characteristics that influence the forces imparted to the breakwater and baffle structures. The location of baffle structures included in the breakwater crest varies for the north and south breakwater structures. the inside surfaces of the breakwater structures that are within the overall proposed beach cell receive less wave stress than the outside faces of the north and south breakwaters. This is because the central breakwater and the water gaps on each side will cause wave diffraction as waves expand into the beach area west of the gap. The baffle structure design focuses on the north side of the north breakwater, and the south side of the south breakwater. These locations are where the wave attack stresses are the greatest and where structural intervention can have the most significant influence on wave energy reduction within the beach cell.
- Results of the wave transformation analysis provide design boundary conditions for the breakwater and baffle structures. This includes the height of the wave that attacks or breaks on the breakwaters and other wave characteristics that are needed for design such as wave period, lake level and breakwater structure geometry. Structural analysis is then performed to design baffle structures that can manage the wave attack. Wave overtopping analysis for the storm events is also performed to help understand the potential for beach erosion and to design baffle structures to protect the beach and proposed plant habitat to be created adjacent to the west ends of the structures.

A wave transformation analysis that transforms the wave from deep water to the breakwater locations is provided in Appendix A. This analysis provides the wave height approaching the north and south breakwaters and the angle of attack to the structures. Results indicate that the 200-year Class 3 storm produces the critical design case. The incident wave height is 6.6 feet as it approaches this structure. The waves approach the north breakwater at an acute angle of approximately 10 degrees relative to the east-west orientation of the breakwater at the location of the baffle structures.

The South Breakwater 200-year storm Class 1 wave approaching from the south has a height of 4.9 feet as it reaches the structure. This wave approaches the breakwater at an acute angle of approximately 17 degrees relative to the orientation of the breakwater which is east west at the location of the baffle structures.

The wave information developed in this analysis provides boundary condition input for further analysis of the baffle structures.

Wave Stress Evaluation

Wave stress analysis provides estimated wave attack forces that interact with the breakwater steel baffle structures. The analysis is based on the lakebed and wave conditions that are constant for much of the baffle structure length. Two methods are used to estimate the wave loads at the baffle structures. The first method estimates the wave stress that is approaching the breakwaters. This method provides a general sense of the approach loads before they hit the structure. The second method estimates wave loads that interact with the baffle wall structures.

Wave Load Estimates Approaching the Structure

The Class 1 wave approaching from the southeast at the South Breakwater is estimated to have a design height of 4.9 feet. The Class 3 wave approaching the North Breakwater is estimated to be 6.6 feet. These waves are taken from the Wave Transformation Analysis discussed earlier in this report.

This method estimates wave loads approaching the breakwater, but assumes the wave is hitting a solid vertical wall. This method is not intended as a determination of wave forces on an armor stone breakwater; however, it provides a good reference point for the designer to compare to the wave loads estimated by the second method that we utilized. The wave loads from this method are higher than method 2 because much of the approach energy is spent on the rubble mound breakwater.

This first approach uses the Goda method ("Random Seas and Design of Maritime Structures, 3rd Edition, Yashimi Goda, 2010). It provides an estimate of the wave stress before the wave interacts with the breakwater. This analysis provides wave pressure at various elevations at the breakwater. The greatest stress occurs at the normal water line and gradually dissipates with height.

This method is not an ideal representation of the stress because it doesn't consider the influence of waves breaking on the breakwater stone and then converting to an overtopping flow on the stone crest. However, it provides a good sense of the pressures approaching the structure. This analysis is provided in Appendix B. The wave stresses are summarized on the third page of the analysis:

- P1 = the wave force at the Lake elevation
- P2 = the load at the bottom of the lake
- P3 = not applicable to this breakwater configuration.
- n* is the height at which zero wave pressure occurs. The pressure reduction from P1 to n* is linear.

Wave Load Estimates at the Baffle Structure

This method of wave load estimation is based on the Jensen and Bradbury (CEM Equation VI-5-186) method. This method is based on physical model studies in a laboratory for stone breakwaters that have a wall structure on top. The breakwater/baffle structure combination fits reasonably well with this method. The method considers the incident wave height, distance between the design water level and the breakwater crest, the significant design wave height attacking the breakwater, deep-water wavelength, the height of the baffle wall structure on top of the breakwater, computation of the wave steepness parameter, and two parameters that were developed from model study tests performed by the authors.

This method estimates a total force on the baffle wall for each longitudinal foot of wall. The computations for this analysis are provided in Appendix C. The analysis results indicate that the total force per foot of baffle wall, which includes the top of the sheet pile wall that sticks up one foot above the breakwater stone, and the height of the baffle wall; is 3,578 pounds per lineal foot of the 4.6-foot baffle wall. This load is apportioned with 778 pounds to the 1' high sheet pile wall at the base of the baffle wall.

The 2,800-pound remainder of the load is taken on by the baffle wall that is connected to the steel sheet pile wall at its base. This load is reduced to an extent since the baffle plates are angled, and the load is a glancing blow. In addition, water that passes through the angled baffle wall louvers reduces pressure somewhat.

The breakwater louvers are an extension of the armor stone breakwater height designed to reduce wave overtopping and transmission. These structures provide a reduction in wave overtopping and transmission. This provides erosion protection for the beach sand due to the elimination of a significant portion of the wave energy that would otherwise enter the harbor. The baffle wall structure also absorbs some of the wave stress that would otherwise have access to the planted habitat zones on the west ends of the breakwater structures.

The baffle structure louvers are spaced at 4" and angled at 40 degrees. The waves that strike the louvers are broken up and the wave load is imparted to the louver surfaces. This has the effect of spreading the load out over a surface area that is larger than the the linear length of the baffle wall alignment.

Wave Overtopping and Transmission Analysis

An analysis of wave overtopping and transmission at the baffle wall locations, performed with and without the baffle structure in place, provides an estimate of the amount of wave overtopping stress that is avoided. The steel sheet pile, which comprises the lowest 1 foot of the baffle structure intercepts and reflects a portion of the overtopping flow back out to the lake. The baffle structure louvers above the level of the steel sheet pile further reduce the overtopping. The baffles restrict and deflect the flow through the louvers and deflect the flows onto the armor stone crest of the breakwater.

The method used to estimate wave overtopping at the baffle structure is the Eurotop *Manual on Wave Overtopping of Sea Defenses and Related Structures* (2018). We first estimate the breakwater overtopping flows without the wave baffle structure. We then estimate the overtopping flow reduction factor for the steel baffle structure based on a literature review of wave energy reduction through perforated structures, and analysis of the impacts of flow influences of angled plates.

Table 1 provides overtopping flow reductions for the north and south breakwater structures with the baffle structures in place. The baffle breakwater structures are located adjacent and to the east of the habitat planting zones and will will help protect these areas from erosion. The baffled breakwater structures also reduce the erosion potential for the beach nourishment sand and associated habitat. The baffle structures add height to the breakwater without any increase in the footprint of the armor stone breakwater that supports this structure. This feature reduces the lakebed impact. The breakwater armor stones also provide fish habitat.

Table 1. Wave Overtopping Flow Reductions at Baffle Structures

Breakwater	Event Recurrence	2% Wave Runup (ft)	SSP Wall Flow Reduction	Baffle Wall Flow
South	200 Year	5.8	96%	4%
North	200 Year	8.0	90.9%	9.1%

The total flow reduction at the North Breakwater is estimated to be 82%, and at the South Breakwater 92%. Several field studies and wave tank studies have documented wave transmission reduction due to wave screens, perforated breakwaters, or curtainwall pile breakwaters. Rageh et. al showed that reductions of 40% to 50% occur in breakwaters using slotted wave screens (2013). Fugazza and Natale showed a similar trend of 45% to 75% reductions based on single chamber perforated breakwaters (1992).

These examples account for reduction of energy based on perforated devices near the water surface. The proposed design has the louvers located 5' above normal water. The lower 1 foot of the structure is a solid steel sheet pile wall which will reflect all water that approaches it back out to the lake. The baffle structure louvers will be hit by water that overtops the breakwater at an oblique angle. This will cause wave reflections within the 4-inch spaces between adjacent louver structures. The turbulent flow in the louver space will cause confused flow conditions that reduce wave transmission. The flows passing through will then spend energy on the sizeable breakwater stone crest before falling back into the water on the lee side of the structure.

Based on the impulse momentum equation and assuming continuity of flow, the angles of the louvers and the angles of waves is estimated to result in a significant reduction in flowrate that would pass through the louvers. The flows coming through the baffle structure are expected to run on the armor stone breakwater crest and slope back into the water. It will no longer be in a wave form and will flow back into the lake with much of the energy being spent on the louvers and breakwater rocks.

The breakwater baffle structure will manage wave attack and reduce wave erosion potential in the vicinity of the proposed plant habitat zones at the west ends of the north and south breakwaters.

In addition, the breakwater baffle structures that are proposed adjacent to the habitat planting areas - north of the north breakwater and south of the south breakwater will provide protection from wave action on the steel sheet pile walls and adjacent beach areas. Wave energy can converge in the corner areas formed where the breakwaters meet the beach.

Breakwater Baffle Structure Design

AECOM performed preliminary structural analyses and design of the breakwater structure baffle wall structure. The wave load calculations indicate that the controlling wave case for the North breakwater baffle wall is a Class 3, 20-year wave attack with a 10-year lake level. The South breakwater baffle wall controlling wave case would be a Class 1, 20-year wave attack with a 10-year lake level. The Class 3 wave load is greater than Class 1 wave load. Both walls are designed using the larger Class 3 wave to provide a consistent structure for both baffle walls.

The baffle walls will be prefabricated in panels. Each panel will consist of several baffle louvers and are comprised of the following: 8-inch-wide x 3.6 ft high x 5/8-inch-thick steel plates welded to a 12-inch-wide x 6-ft long (+/-) x 1-inch thick steel baseplate. The north breakwater baffle wall design is as follows:

- The largest controlling wave loads occur at the eastern 40 feet of the baffle wall. The wave loads developed for this area are also used for the baffle wall areas located closer to shore where the wave heights and loads are less due to more shallow water depths. The western portions of the baffle wall are in shallow water. The waves will converge somewhat in the corner formed by the lake edge and the planter walls. This convergence will provide some splash and spray and the baffle structure will provide protection from this. The baffle wall structural design for this area will be the same as for the eastern end of the baffle wall. Though the louver lengths are longer.

- The top of the baffle wall at the eastern 40 feet of North breakwater and 55 feet at the South breakwater is at a maximum elevation of 590.60', and gradually reduces in height going east.

- The bottom of the baseplate at the steel sheet pile cap is at Elevation 587.00' in the deep-water locations, and gradually rises approaching shore. The wave loads on the baffle walls are reduced as the wall approaches shore. The lake depth becomes gradually shallower, and the wave loads gradually reduced.

- The louver plates will be spaced at 4 inches as measured in the direction perpendicular to the plane of the louver plate. The baseplates will be bolted to a steel cap (16-in channel) at the top of the steel sheet pile (SSP). The steel cap would be welded to the SSP wall. This wall will be embedded into the ground and will be buttressed on both sides by the breakwater.

- Two rows of bolts (5/8-inch diameter, A325) would connect the louver panels to the steel cap of the SSP wall. The bolts would be spaced at 12 inches (+/-) in the long direction of the cap and at 8 inches in the transverse direction of the cap.

- Structural calculations and design schematics showing details of a typical louver panel is provided in Appendix E.

The steel louvers are oriented with a 40-degree angle rotated clockwise to the northeast. The incident design wave would approach the breakwater at a skew of 14 to 17 degrees to the orientation of the breakwater – a glancing attack. The skew angle between the incident wave and the orientation of the baffle wall louvers is therefore estimated to be an angle of 54 degrees relative to the approaching wave, and a line that is perpendicular to the plane of the louver.

The component of the design wave force acting in the direction perpendicular to the plane of the louver is estimated to be reduced by 40% to account for the skewed wave strike on the face of the louvers. The resulting pressure was used to determine the moment and shear stresses in the louver plates. We followed AISC Specification Standards to design the louver plate, welded connection to the baseplate and bolted connection to the steel cap of the SSP wall.

AECOM evaluated other load cases. The wind load case was considered insignificant. The ice load case was analyzed following recommendations and guidelines presented in the USACOE (EM-2-1612). It was assumed that a 12-inch-thick by 18-ft long ice sheet is floating by waves and hitting the baffle wall at mid-height in combination with the wave pressure. This load would be infrequent since the baffle structure base elevation is 5 feet above flood stage. Results of these analyses indicate the louver panels will experience stresses below their allowable limits during Lake Michigan wind, wave and ice force events, and are therefore structurally adequate for use as proposed.

Appendix A

Wave Transformation Model

Introduction and Purpose: Perform coastal engineering calculations to transform deepwater waves in Lake Michigan to the nearshore environment. Estimate Incident wave conditions. Estimate the required armor stone sizes to resist wave forces. The shoreline and breakwater locations are illustrated on the design drawings. Use the 10-year lake level and the 20-year deepwater wave as the preliminary design condition. This combination will produce a combined event recurrence interval in excess of a 100 year event. Then perform a check for the 20-yr lake level and a 10-yr wave.

I. Deepwater Wave and Water Level Conditions, and Basic Shoreline Characteristics at Project Site

Deepwater Waves:

Figure 1 illustrates the shoreline orientation for the site relative to the regional shoreline orientation for the nearest available wave information station. This station is designated as WIS Station No. 94027. The project site is located at 42.1 deg N, 87.72 deg W. WIS Station No. 94027 is located at 42.2 deg N, 87.72 deg W. Wave station information from the year 1960 through 2014 is downloaded in ONELINES Data Format. The largest 500 waves for each class was analyzed using the CEDAS - ACES program developed by the U.S. Army Engineer Waterways Experiment Station, Bicksburg, Mississippi to produce the following Extremal Significant Wave Heights. A Weibull distribution shape parameter k = 1.0 best represents the Class 1 WIS data; k = 1.4 best represents Classes 2 and 3 WIS data.

Deepwater Waves for the 20-year event:

$H_{oClass3} := 17.72$ $\quad H_{oClass2} := 13.62$ $\quad H_{oClass1} := 7.63$

Deepwater Waves for the 10-year event:

$H_{oClass3\_10} := 16.83$ $\quad H_{oClass2\_10} := 12.5$ $\quad H_{oClass1\_10} := 7.19$

Design Basis Lake Michigan Water Level :

A Lake Michigan Level and Wave Climate Evaluation completed by AECOM in 2010 provides updated water level frequency curves for Calumet Harbor. "Chicago Lake Michigan Level and Wave Climate Evaluation, September 15, 2009." This study includes a 93 year Lake Michigan water level record. The results for Calumet Harbor are then transferred to the Winnetka design site using a proration of the Corps 2009 Lake Level Study (1993) for levels between Calumet and Milwaukee. The Extreme High Water Level Summary in IGLD 85 is shown in the below table.

	10 year	20 year
Class 1	581.8	582.3
Class 2	582.3	582.8
Class 3	582.5	583.0

In the AECOM analysis, the mean lake level for each return period is added to a storm surge to produce a total combined water level for each return period. The evaluation also took into account the effects of wave setup directionality and lag time between time of peak wave heights and wind setup. Each high water level analysis is broken up into Sectors that roughly correspond to the various classes included in this analysis.

Lake levels for this design are based on an interpretation of the Lake Michigan Level and Wave Climate Evaluation (AECOM, 2010) for Calumet Harbor. The levels at the project site will be lower than those are Calumet Harbor for Class 3 wave attack due to the reduced lake fetch to the site that produces a lesser wave lake level setup. Therefore, the AECOM (2010) analysis needs to be adjusted. This analysis corrects for this by prorating the relative difference in lake levels from the Calumet Harbor and Milwaukee gages. The annual design water levels for each gage site in IGLD 85 is reproduced in the below table from the Design Water Level Determination on the Great Lakes (US Army Corps of Engineers, 1993).

	10 year	20 year	30 year
Calumet Harbor	582.9	583.3	583.6
Milwaukee	582.2	582.5	582.8
Difference	0.7	0.8	0.8

Since the project site in Winnetka is between Milwaukee and Calumet Harbor, the AECOM (2010) lake levels will be adjusted accordingly. The distance between Calumet Harbor and Milwaukee is 90 miles and the distance between Calumet Harbor and Winnetka is approximately 31 miles. The design lake level will be adjusted by 34% of the difference between the design water levels from the above table, or 0.24 and 0.27' will be subtracted from each of the 10 year and 20 year AECOM (2010) lake level frequency values to adjust them for the project site. This adjustment is shown in the below table.

	10 year	20 year
Class 1	581.56	582.03
Class 2	582.06	582.53
Class 3	582.26	582.73

Design Water Levels for the 20-year event:

$dwl_3 := \mathbf{582.73}$ $dwl_2 := \mathbf{582.53}$ $dwl_1 := \mathbf{582.03}$

Design Water Levels for the 10-year event:

$dwl_{3\_10} := \mathbf{582.26}$ $dwl_{2\_10} := \mathbf{582.06}$ $dwl_{1\_10} := \mathbf{581.56}$

Wave Period for Deep Water Waves:

Wave periods for class 1, class 2, and class 3 waves were obtained from the 1976 "Design Wave Information for the Great Lakes, Report 3, Lake Michigan" by Resio and Vincent. Grid Point number 33 at the Winnetka, Illinois grid location 42.26, 87.73 is closest to the project site. Table E3 provides significant wave periods orgnized by both wave height and angle class. The angle classes roughly correspond to the classes used for the significant wave heights.

In 2009, DHI completed a report titled "Long Term Wind-Wave Hindcase and Wave Transformation Modeling" updating the significant wave period analysis for Calument Harbor. The DHI Report analyzes the wave period for different wave heights and the results for the 10 and 20 year frequencies are very similar to the wave periods available in Table E3 (Resio and Vincent, 1976). Because the deepwater waves for this project location are significantly different from the wave heights reported at Calumet Harbor, the wave periods are obtained from Table E3 (Resio and Vincent, 1976) and not from the DHI report.

As a final check on the reliability of using Table E3 (Resio and Vincent, 1976), the wave periods were compared to the 1979 to 2014 Percent Occurance of Height and Period by Direction tables for the WIS station closest to this project. The WIS data confirms a 9-9.9 second wave period for the class 3 10 and 20 year wave heights, 8-8.9 second wave period for class 2 10 and 20 year wave heights, and 6-6.9 second wave period for class 1 10 and 20 year wave heights.

Wave Periods for the 20-year event:

$t_3 := 9.9$ $\quad t_2 := 8.4$ $\quad t_1 := 7.0$

Wave Periods for the 10-year event:

$t_{3\_10} := 9.6$ $\quad t_{2\_10} := 8.1$ $\quad t_{1\_10} := 6.7$

II. Proposed Structure Incident Wave Condition and Forces Analysis

A. Main Breakwater Incident Wave Conditions

Main Breakwater Incident Wave Condition Analysis:

1. Lake Bottom Condition

The lake bottom elevation is taken from the lake side toe of the proposed structure at a distance approximately equal to five times the wave height at the point of attack. Assuming the wave height is approximately 8 feet, the lake bottom elevation at 40 feet from the north breakwater baffle alignment is 575.5 NAVD 88 or approximately 575.0 IGLD 85 based on a bathymetric survey completed by Terra in December of 2020. At the end of the wave condition analysis, a check will be made to confirm that the wave height does not require a second iteration with a refined lake bottom elevation.

The AECOM survey limits extend to approximately 250 feet offshore. The lake bed slope is approximately 1:30. Beyond 130 feet offshore, the lake bed slope flattens to approximately 1:70.

Estimate lake bed depth from design lake level for the 20-year event:

$dwl_3 = \mathbf{582.73}$ $\quad bed_{3\_20} := \mathbf{575.0}$ $\quad h_{3\_20} := dwl_3 - bed_{3\_20}$ $\quad h_{3\_20} = \mathbf{7.73}$

$dwl_2 = \mathbf{582.53}$ $\quad bed_{2\_20} := \mathbf{575.0}$ $\quad h_{2\_20} := dwl_2 - bed_{2\_20}$ $\quad h_{2\_20} = \mathbf{7.53}$

$dwl_1 = \mathbf{582.03}$ $\quad bed_{1\_20} := \mathbf{575.0}$ $\quad h_{1\_20} := dwl_1 - bed_{1\_20}$ $\quad h_{1\_20} = \mathbf{7.03}$

Estimate lake bed depth from design lake level for the 10-year event:

$dwl_{3\_10} = \mathbf{582.26}$ $\quad bed_{3\_10} := \mathbf{575.0}$ $\quad h_{3\_10} := dwl_{3\_10} - bed_{3\_10}$ $\quad h_{3\_10} = \mathbf{7.26}$

$dwl_{2\_10} = \mathbf{582.06}$ $\quad bed_{2\_10} := \mathbf{575.0}$ $\quad h_{2\_10} := dwl_{2\_10} - bed_{2\_10}$ $\quad h_{2\_10} = \mathbf{7.06}$

$dwl_{1\_10} = \mathbf{581.56}$ $\quad bed_{1\_10} := \mathbf{575.0}$ $\quad h_{1\_10} := dwl_{1\_10} - bed_{1\_10}$ $\quad h_{1\_10} = \mathbf{6.56}$

2. Estimate the near-breakwater Wave Height after Refraction **10-Year Lake & 20-Year Wave**

Use Class 3, 20-Year Deepwater Wave Height = H_o

$H_{oClass3} = \mathbf{17.72}$ $\quad t_3 = \mathbf{9.9}$ sec

$L_o := \mathbf{1.56} \cdot t_3^{\mathbf{2}} \cdot \mathbf{3.281}$ $\quad L_o = \mathbf{501.65}$ $\quad \frac{h_{3\_10}}{L_o} = \mathbf{0.014}$

Offshore bottom contours are nearly parallel to shore; therefore, use Goda (Fig. 3.6):

$Kr_{20} := \mathbf{.78}$ $\quad \alpha_o := \mathbf{60}$ deg

$H_{oClass3'} := Kr_{20} \cdot H_{oClass3}$

$H_{oClass3'} = \mathbf{13.822}$

Use Class 2, 20-Year Deepwater Wave Height = H_o

$H_{oClass2} = 13.62$ $\quad t_2 = 8.4$ sec

$L_o := 1.56 \cdot t_2^{2} \cdot 3.281$ $\quad L_o = 361.151$ $\quad \frac{h_{2\_10}}{L_o} = 0.02$

Offshore bottom contours are nearly parallel to shore; therefore, use Goda (Fig. 3.6):

$Kr_{20} := .93$ $\quad \alpha_o := 60$ deg

$H_{oClass2'} := Kr_{20} \cdot H_{oClass2}$

$H_{oClass2'} = 12.667$

Use Class 1, 20-Year Deepwater Wave Height = H_o

$H_{oClass1} = 7.63$ $\quad t_1 = 7$ sec

$L_o := 1.56 \cdot t_1^{2} \cdot 3.281$ $\quad L_o = 250.8$ $\quad \frac{h_{1\_10}}{L_o} = 0.026$

Offshore bottom contours are nearly parallel to shore; therefore, use Goda (Fig. 3.6):

$Kr_{20} := .79$ $\quad \alpha_o := 60$ deg

$H_{oClass1'} := Kr_{20} \cdot H_{oClass1}$

$H_{oClass1'} = 6.028$

3. Estimate the near-breakwater class 3 Wave Angle after Refraction

The refracted wave approaching the structures is at an angle of α_p degrees from normal.

$\alpha_o = 60$ deg $\quad \frac{h_{3\_10}}{L_o} = 0.029$

$\alpha_p := 10\text{deg}$ Fig. 3.7 (Goda)

4. Estimate the near-breakwater class 1 Wave Angle after Refraction

The refracted wave approaching the structures is at an angle of α_p degrees from normal.

$\alpha_o = 60$ deg $\quad \frac{h_{1\_10}}{L_o} = 0.026$

$\alpha_p := 17\text{deg}$ Fig. 3.7 (Goda)

5. Estimate the Incident Wave Height in the Surf Zone East of the Structure

Use Goda to estimate the incident wave height in the surf zone for Class 3.

$$\frac{H_{oClass3'}}{L_o} = 0.055 \qquad \frac{h_{3\_10}}{H_{oClass3'}} = 0.525$$

Use Goda Figures 3.31 and 3.32 to estimate surf zone reduction factors $H_{1/3}/H_{0'}$ using a near shore bottom slope of 1:50:

For slope = 1:30, surf zone factor (SZF): $SZF_{30} := .49$

For slope = 1:100, surf zone factorc(SZF): $SZF_{100} := .45$

Interpolate for slope:

$$s := 50 \qquad SZF := SZF_{100} - (100 - s)\frac{(SZF_{100} - SZF_{30})}{100 - 30}$$

$$SZF = 0.479$$

Estimate significant wave for wave transformation:

$$H_{sig3} := (SZF)\cdot(H_{oClass3'}) \qquad H_{sig3} = 6.615$$

Estimate maximum wave height:

$$H_{max} := 0.58\cdot H_{oClass3'} \qquad H_{max} = 8.017$$

Use Goda to estimate the incident wave height in the surf zone for Class 2.

$$\frac{H_{oClass2'}}{L_o} = 0.051 \qquad \frac{h_{2\_10}}{H_{oClass2'}} = 0.557$$

Use Goda Figures 3.31 and 3.32 to estimate surf zone reduction factors $H_{1/3}/H_{0'}$ using a near shore bottom slope of 1:50:

For slope = 1:30, surf zone factor (SZF): $SZF_{30} := .5$

For slope = 1:100, surf zone factor (SZF): $SZF_{100} := .44$

Interpolate for slope:

$$s := 50 \qquad SZF := SZF_{100} - (100 - s)\frac{(SZF_{100} - SZF_{30})}{100 - 30}$$

$SZF = 0.483$

Estimate significant wave for wave transformation:

$H_{sig2} := (SZF) \cdot (H_{oClass2'})$ $\qquad H_{sig2} = 6.116$

Estimate maximum wave height:

$H_{max} := 0.63 \cdot H_{oClass2'}$ $\qquad H_{max} = 7.98$

Use Goda to estimate the incident wave height in the surf zone for Class 1.

$\frac{H_{oClass1'}}{L_o} = 0.024$ $\qquad \frac{h_{1\_10}}{H_{oClass1'}} = 1.088$

Use Goda Figures 3.31 and 3.32 to estimate surf zone reduction factors $H_{1/3}/H_{0'}$ using a near shore bottom slope of 1:50:

For slope = 1:30, surf zone factor (SZF): $SZF_{30} := .8$

For slope = 1:100, surf zone factorc(SZF): $SZF_{100} := .7$

Interpolate for slope:

$s := 50$ $\qquad SZF := SZF_{100} - (100 - s)\frac{(SZF_{100} - SZF_{30})}{100 - 30}$

$SZF = 0.771$

Estimate significant wave for wave transformation:

$H_{sig1} := (SZF) \cdot (H_{oClass1'})$ $\qquad H_{sig1} = 4.65$

Estimate maximum wave height:

$H_{max} := .91 \cdot H_{oClass1'}$ $\qquad H_{max} = 5.485$

The incident wave height in the surf zone for Class 3, Class 2 and Class 1 is 6.6 , 6.1 and 4.7 respectively. Therefore, Class 3 controls.

6. Estimate the near-breakwater Wave Height after Refraction for **20 year Lake level/ 10 yr wave**:

Use Class 3, 10-Year Deepwater Wave Height = H_o

$H_{oClass3\_10} = 16.83$ $\qquad t_{3\_10} = 9.6$

$L_{o\_10} := 1.56 \cdot t_{3\_10}{}^{2} \cdot 3.281$ $\quad L_{o\_10} = 471.708$ $\quad \dfrac{h_{3\_20}}{L_o} = 0.031$

Offshore bottom contours are nearly parallel to shore; therefore, use Goda (figure 3.6):

$Kr_{10} := .79$ $\quad \alpha_o := 60 \text{ deg}$

$H_{oClass3\_10'} := Kr_{10} \cdot H_{oClass3\_10}$

$H_{oClass3\_10'} = 13.296$

Use Class 2, 10-Year Deepwater Wave Height = H_o

$H_{oClass2\_10} = 12.5$ $\quad t_{2\_10} = 8.1$

$L_{o\_10} := 1.56 \cdot t_{2\_10}{}^{2} \cdot 3.281$ $\quad L_{o\_10} = 335.816$ $\quad \dfrac{h_{2\_20}}{L_o} = 0.03$

Offshore bottom contours are nearly parallel to shore; therefore, use Goda (figure 3.6):

$Kr_{10} := .94$ $\quad \alpha_o := 30 \text{ deg}$

$H_{oClass2\_10'} := Kr_{10} \cdot H_{oClass2\_10}$

$H_{oClass2\_10'} = 11.75$

Use Class 1, 10-Year Deepwater Wave Height = H_o

$H_{oClass1\_10} = 7.19$ $\quad t_{1\_10} = 6.7$

$L_{o\_10} := 1.56 \cdot t_{1\_10}{}^{2} \cdot 3.281$ $\quad L_{o\_10} = 229.763$ $\quad \dfrac{h_{1\_20}}{L_o} = 0.028$

Offshore bottom contours are nearly parallel to shore; therefore, use Goda (figure 3.6):

$Kr_{10} := .8$ $\quad \alpha_o := 60 \text{ deg}$

$H_{oClass1\_10'} := Kr_{10} \cdot H_{oClass1\_10}$

$H_{oClass1\_10'} = 5.752$

7. Estimate the near-breakwater class 3 Wave Angle after Refraction (use Goda - figure 3.7)

The refracted wave approaching the structures is at an angle of α_p degrees from normal.

$\alpha_o = 60 \text{ deg}$ $\quad \dfrac{h_{3\_20}}{L_o} = 0.031$

$\alpha_p := 9\text{deg}$ Fig. 3.7 (Goda)

8. Estimate the near-breakwater class 1 Wave Angle after Refraction (use Goda - figure 3.7)

The refracted wave approaching the structures is at an angle of α_p degrees from normal.

$\alpha_o = 60 \quad \text{deg}$ $\qquad \dfrac{h_{1\_20}}{L_o} = 0.028$

$\alpha_p := 9\text{deg}$ Fig. 3.7 (Goda)

9. Estimate the Incident Wave Height in the Surf Zone.

Use Goda to estimate the incident wave height in the surf zone for Class 3.

$\dfrac{H_{oClass3\_10'}}{L_{o\_10}} = 0.058 \qquad \dfrac{h_{3\_20}}{H_{oClass3\_10'}} = 0.581$

Use Goda Figures 3.31 and 3.32 to estimate surf zone reduction factors $H_{1/3}/H_{0'}$ using a near shore bottom slope of 1:50:

For slope = 1:30, surf zone factor (SZF): $SZF_{30} := .5$

For slope = 1:100, surf zone factorc(SZF): $SZF_{100} := .46$

Interpolate for slope:

$s := 50 \qquad SZF := SZF_{100} - (100 - s)\dfrac{(SZF_{100} - SZF_{30})}{100 - 30}$

$SZF = 0.489$

Estimate significant wave for wave transformation and armor stone sizing purposes:

$H_{sig3\_10} := (SZF)\cdot(H_{oClass3\_10'}) \qquad H_{sig3\_10} = 6.496$

Estimate maximum wave height:

$H_{max} := 0.59\cdot H_{oClass3\_10'} \qquad H_{max} = 7.844$

Use Goda to estimate the incident wave height in the surf zone for Class 2.

$\dfrac{H_{oClass2\_10'}}{L_{o\_10}} = 0.051 \qquad \dfrac{h_{2\_20}}{H_{oClass2\_10'}} = 0.641$

Use Goda Figures 3.31 and 3.32 to estimate surf zone reduction factors $H_{1/3}/H_{0'}$ using a near shore bottom slope of 1:50:

For slope = 1:30, surf zone factor (SZF): $SZF_{30} := .57$

For slope = 1:100, surf zone factor (SZF): $SZF_{100} := .51$

Interpolate for slope:

$$s := 45 \qquad SZF := SZF_{100} - (100 - s)\frac{(SZF_{100} - SZF_{30})}{100 - 30}$$

$$SZF = 0.557$$

Estimate significant wave for wave transformation and armor stone sizing purposes:

$$H_{sig2\_10} := (SZF)\cdot(H_{oClass2\_10'}) \qquad H_{sig2\_10} = 6.546$$

Estimate maximum wave height:

$$H_{max} := 0.70 \cdot H_{oClass2\_10'} \qquad H_{max} = 8.225$$

Use Goda to estimate the incident wave height in the surf zone for Class 1.

$$\frac{H_{oClass1\_10'}}{L_{o\_10}} = 0.025 \qquad \frac{h_{1\_20}}{H_{oClass1\_10'}} = 1.222$$

Use Goda Figures 3.31 and 3.32 to estimate surf zone reduction factors $H_{1/3}/H_{0'}$ using a near shore bottom slope of 1:50:

For slope = 1:30, surf zone factor (SZF): $SZF_{30} := .9$

For slope = 1:100, surf zone factor (SZF): $SZF_{100} := .75$

Interpolate for slope:

$$s := 50 \qquad SZF := SZF_{100} - (100 - s)\frac{(SZF_{100} - SZF_{30})}{100 - 30}$$

$$SZF = 0.857$$

Estimate significant wave for wave transformation:

$$H_{sig1\_10} := (SZF)\cdot(H_{oClass1\_10'}) \qquad H_{sig1\_10} = 4.93$$

Estimate maximum wave height:

$$H_{max} := 1.0 \cdot H_{oClass1\_10'} \qquad H_{max} = 5.752$$

The incident wave height in the surf zone for Class 3, Class 2, and Class 1 is 6.5, 6.5, and 4.9 respectively. Therefore, Class 3 controls.

Appendix B

Wave Transformation – Approach Wave Loads

Incident Wave in the Surf Zone Based on AECOM Wave Heights

Shoreline Designation	Water Level (ft - LWD)	*Average Lakebed (ft - LWD)	Depth at Structure h (ft)	**Intermed. Wave Ho	h/H_o	Wave Period (sec)	Wavelength Lo	Refraction Angle α	h/L_o	Refraction Coefficient K_r (Goda Fig 3.6)	Approach Angle after Refr (α_p)	H_o'	h/H_o'	H_o'/L_o	Goda Fig. 3.32	**Nonbrk. $H_{1/3}$ (ft)	Break Loc. 5*($H_{1/3}$) [H	Lake Bottom (Near Flat)	H.max Depth @ 5*($H_{1/3}$) hb (ft)	hb/Lo	hb/Ho`	Ho`/Lo	Hmax at Hmax/Ho' Goda Fig 3.32	Hmax @ h_b
North Breakwater																								
I. 200 year event: 20 year wave \ 10 year level																								
Class 3	582.26	575.0	7.3	17.72	0.41	9.9	501.7	60.0	0.014	0.780	**10.0**	**13.8**	0.53	0.028	0.480	**6.6**	33.2	Flat	6.6	0.01	0.48	0.028	0.580	8.0
Class 2	582.06	575.0	7.1	13.62	0.52	8.4	361.2	0.0	0.020	0.930	**0.0**	**12.7**	0.56	0.035	0.480	**6.1**	30.4	20 to 1	7.1	0.02	0.56	0.035	0.630	8.0
Class 1	581.56	575.0	6.6	7.63	0.86	7	250.8	60.0	0.026	0.790	**17.0**	**6.0**	1.09	0.024	0.780	**4.7**	23.5	50 to 1	5.2	0.02	0.86	0.024	0.910	5.5
II. 200 year event: 10 year wave \ 20 year level																								
Class 3	582.73	575.0	7.7	16.83	0.46	9.6	471.7	60.0	0.016	0.790	**13.5**	**13.3**	0.58	0.028	0.49	**6.5**	32.6	Flat	6.5	0.01	0.49	0.028	0.59	7.8
Class 2	582.53	575.0	7.5	12.5	0.60	8.1	335.8	0.0	0.022	0.940	**0.0**	**11.8**	0.64	0.035	0.55	**6.5**	32.3	20 to 1	7.5	0.02	0.64	0.035	0.70	8.2
Class 1	582.03	575.0	7.0	7.19	0.98	6.7	229.8	60.0	0.031	0.800	**18.0**	**5.8**	1.22	0.025	0.85	**4.9**	24.4	50 to 1	5.4	0.02	0.94	0.025	1.00	5.8
	***Lake Bottom Slope is approx. 30:1 approching shore																							
	**Equivalent intermediate wave beyond Surf Zone.																							
	*To Lake Bottom.																							

Note: The louvers are designed for the Significant Wave Height for a 200 year wind wave storm. The Max wave height case applies to critical infrastructure such as seawalls with pedestrian access and strucures with nearby building structures - this wave represents a 1/250 wave height and doesn't apply to the louvers which are not major structures.

I. Summary of Maximum Wave In Designated Zones.

Wave Designation	Water Level (ft - LWD)	Average Lakebed (ft - LWD)	Depth at Structure h (ft)	**Incident Wave Ho''	$H''_{1/3}$ (ft)		$5*H''_{1/3}$ (ft)	hb* (ft)	Wave Period (sec)	Wavelength Lo	hb/Lo		Incident Ho'' (ft)				Incident Ho'' (ft)	
I. 200 year event: 20 year wave \ 10 year level																		
Class 3	582.26	575	7.26	6.6	6.6		33.2	6.6	9.9	501.7	0.0132		6.6				6.6	
Class 2	582.06	575	7.06	6.1	6.1		30.4	7.1	8.4	361.2	0.0196		6.1				6.1	
Class 1	581.56	575	6.56	4.7	4.7		23.5	5.2	7	250.8	0.0207		4.7				4.7	
II. 200 year event: 10 Year wave \ 20 year level																		
Class 3	582.73	575	7.73	6.5	6.5		32.6	6.5	9.6	471.7	0.0138		6.5				6.5	
Class 2	582.53	575	7.53	6.5	6.5		32.3	7.5	8.1	335.8	0.0222		6.5				6.5	
Class 1	582.03	575	7.03	4.9	4.9		24.4	5.4	6.7	229.8	0.0235		4.9				4.9	
	* Assume harbor bottom slope of near flat **Use Highest Incident Wave Condition for Angle Class 2 for Wave Crest Pressure Computation.																	

II. Wave Crest Pressure Estimation

Wave Designation	Hmax at hb (ft)	Tp (sec)	alpha (deg)	h (ft)	h/Lo	Toe Berm Top Elev. LWD	Toe Berm thickness (ft)	d (ft)	hb (ft)	n* (ft)	alpha1 Fig 4.5	alpha2	alpha2	1/cosh Fig 4.6	alpha3	Wave Crest Pressures P1 (psf)	P2 (psf)	P3 (psf)
I. 200 year event: 20 year wave \ 10 year level																		
Class 3	6.6	9.9	10.0	7.3	0.014	0	0.0	7.3	6.6	9.9	1.048	-0.02625	2.19	0.950	0.95	420.1	399.1	399.1
Class 2	6.1	8.4	0.0	7.1	0.020	0	0.0	7.1	7.1	9.1	1.020	0.00070	2.32	0.932	0.93	387.2	360.9	360.9
Class 1	4.7	7.0	17.0	6.6	0.026	0	0.0	6.6	5.2	6.9	0.980	-0.04471	2.79	0.912	0.91	269.5	245.8	245.8
I. 200 year event: 10 year wave \ 20 year level						0												
Class 3	6.5	9.6	18.0	7.7	0.016	0	0.0	7.7	6.5	9.5	1.034	-0.04416	2.37	0.947	0.95	394.2	373.3	373.3
Class 2	6.5	8.1	0.0	7.5	0.022	0	0.0	7.5	7.5	9.7	1.010	-0.00222	2.33	0.930	0.93	1347.0	1252.7	1252.7
Class 1	4.9	6.7	9.0	7.0	0.031	0	0.0	7.0	5.4	7.3	0.980	-0.04909	2.88	0.900	0.90	1147.7	1033.0	1033.0

Station	Hmax at hb (ft)	h (ft)	P1 (psf)	P2 (psf)	P3 (psf)
I. 200 year event: 20 year wave \ 10 year level					
Class 3	6.6	7.3	420.1	399.1	399.1
Class 2	6.1	7.1	387.2	360.9	360.9
Class 1	6.5	7.7	269.5	245.8	245.8
I. 200 year event: 10 year wave \ 20 year level					
Class 3	6.5	7.3	394.2	373.3	373.3
Class 2	6.5	7.1	1347.0	1252.7	1252.7
Class 1	4.9	6.6	1147.7	1033.0	1033.0

Appendix C

Baffle Wall Wave Loads

North Breakwater

Class	Modeling Scenario	SWL (ft)	Breakwater Crest Elev (ft)	Depth at Breakwater Toe (ft)	Hb (ft)	L0 (ft)	Maximum Louvre Height (ft)	A_c (ft)	A_c (m)
CLASS 3	with Louver, 10 year	582.26	586	7.26	6.6	390	4.6	3.74	1.14
CLASS 3	With Louver,20 year	582.73	586	7.73	6.5	414	4.6	3.27	1.00

South Breakwater

Class	Modeling Scenario	SWL (ft)	Breakwater Crest Elev (ft)	Depth at Breakwater Toe (ft)	Hb (ft)	L0 (ft)	Maximum Louvre Height (ft)	A_c (ft)	A_c (m)
CLASS 1	with Louver, 10 year	581.56	586	6.56	4.7	190	4.6	4.44	1.35
CLASS 1	With Louver,20 year	582.03	586	7.03	4.9	207	4.6	3.97	1.21

NOTES/ASSUMPTIONS

Maximum louvre height along each breakwater was used

CEM Part VI - Wave Overtopping Force on Wall -- North Breakwater

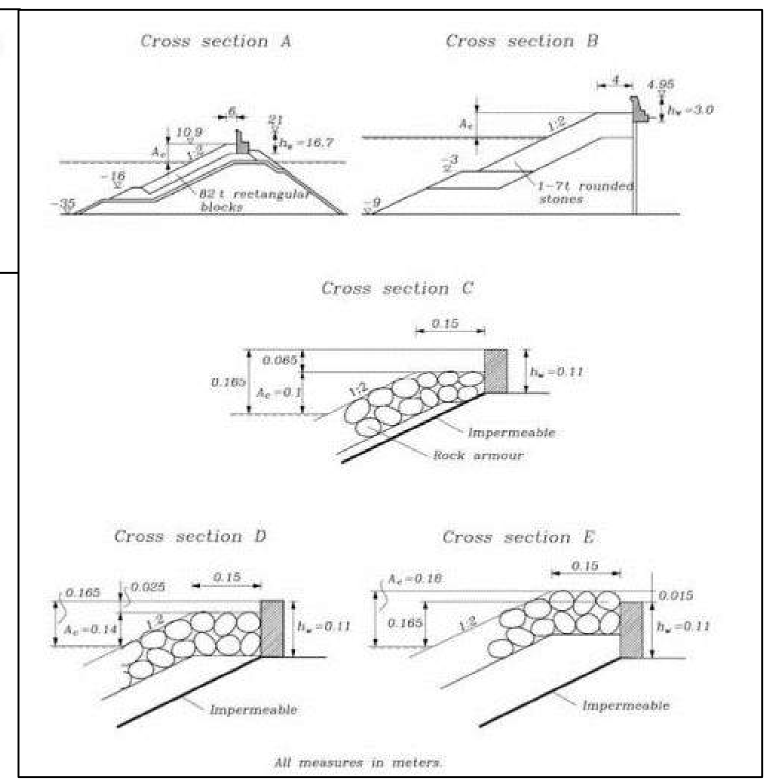
Page VI-V-176 in CEM Part VI

Force Calculations: Method 1, Jensen and Bradbury (Equation VI-5-186)

Step	Variable	Symbol					Class 3 10-Year	Class 3 20-Year	Notes
1	Mass Density of Water (slugs/ft3 or kg/m3)	r_w					*1.9*	*1.9*	Fresh water
2	Vertical Distance Between SWL and Berm Crest	A_c					*3.74*	*3.27*	
3	Gravitational Acceleration (ft/s2 or m/s2)	g					*32.174*	*32.174*	Maximum SWL along wall
4	Significant Wave Height in front of Breakwater (ft or m)	H_s					*6.6*	*6.5*	
5	Deepwater Wave Length Corresponding to Peak Wave Period (ft or m)	L_0					*390*	*414*	
6	Crown Wall Height (ft or m)	h_w					*4.60*	*4.60*	ASCE 7-22 Equation 5.4-5, Assumes Breaking Waves
7	Wave Steepness	H_s/L_0					*0.02*	*0.02*	
8	α	-					*-0.02*	*-0.02*	From CEM Table VI-5-60
9	β	-					*0.03*	*0.03*	From CEM Table VI-5-60
10	Hs/Ac	-					*1.76*	*1.99*	Calculated
11	**Force on Unit Length of Wall (lb/ft or N/m)**	$\mathbf{F_{h,0.1\%}}$					***3,148.53***	***4,005.16***	**Calculated**

$$\frac{F_{h,0.1\%}}{\rho_w\, g\, h_w\, L_{op}} = \alpha + \beta \frac{H_s}{A_C} \qquad \text{(VI-5-186)}$$

where
- $F_{h,0.1\%}$ Horizontal wave force per running meter of the wall corresponding to 0.1% exceedence probability
- ρ_w Mass density of water
- h_w Crown wall height
- L_{op} Deepwater wavelength corresponding to peak wave period
- H_s Significant wave height in front of breakwater
- A_c Vertical distance between MWL and the crest of the armor berm
- α, β Fitted coefficient, see table



Cross section	Parameter ranges in tests A_c (m)	$s_{op} = \frac{H_s}{L_{op}}$	$\frac{H_s}{A_c}$	0.1% exceedence values of coefficients in Eq (VI-5-186) α	β	Coefficient of variation	Reference
A	5.6 - 10.6	0.016 - 0.036	0.76 - 2.5	−0.026	0.051	0.21	Jensen (1984)
B	1.5 - 3.0	0.05 - 0.011	0.82 - 2.4	−0.016	0.025	0.46	
C	0.10	0.023 - 0.07	0.9 - 2.1	−0.038	0.043	0.19	Bradbury, et al. (1988)
D	0.14	0.04 - 0.05	1.43	-0.025	0.028		—
E	0.18	0.04 - 0.05	1.11	-0.088	0.011		—

CEM Part VI - Wave Overtopping Force on Wall -- South Breakwater

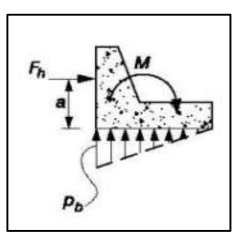
Page VI-V-176 in CEM Part VI

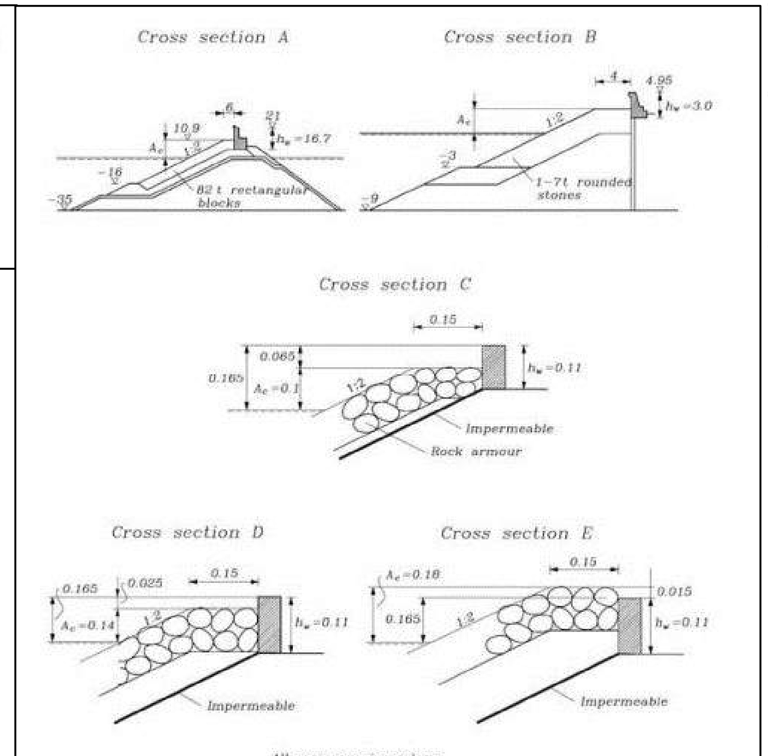
Force Calculations: Method 1, Jensen and Bradbury (Equation VI-5-186)

Step	Variable	Symbol	Class 1 10-Year	Class 1 20-Year					Notes
1	Mass Density of Water (slugs/ft3 or kg/m3)	r_w	*1.9*	*1.9*					Fresh water
2	Vertical Distance Between SWL and Berm Crest	A_c	*4.44*	*3.97*					
3	Gravitational Acceleration (ft/s2 or m/s2)	g	*32.174*	*32.174*					Maximum SWL along wall
4	Significant Wave Height in front of Breakwater (ft or m)	H_s	*4.7*	*4.9*					
5	Deepwater Wave Length Corresponding to Peak Wave Period (ft or m)	L_0	*190*	*207*					
6	Crown Wall Height (ft or m)	h_w	*4.60*	*4.60*					ASCE 7-22 Equation 5.4-5, Assumes Breaking Waves
7	Wave Steepness	H_s/L_0	*0.02*	*0.02*					
8	α	-	*-0.02*	*-0.02*					From CEM Table VI-5-60
9	β	-	*0.03*	*0.03*					From CEM Table VI-5-60
10	Hs/Ac	-	*1.06*	*1.23*					Calculated
11	**Force on Unit Length of Wall (lb/ft or N/m)**	$\mathbf{F_{h,0.1\%}}$	***570.76***	***884.53***					**Calculated**

$$\frac{F_{h,0.1\%}}{\rho_w\, g\, h_w\, L_{op}} = \alpha + \beta \frac{H_s}{A_C} \qquad \text{(VI–5–186)}$$

where
- $F_{h,0.1\%}$ Horizontal wave force per running meter of the wall corresponding to 0.1% exceedence probability
- ρ_w Mass density of water
- h_w Crown wall height
- L_{op} Deepwater wavelength corresponding to peak wave period
- H_s Significant wave height in front of breakwater
- A_c Vertical distance between MWL and the crest of the armor berm
- α, β Fitted coefficient, see table





Cross section	Parameter ranges in tests A_c (m)	$s_{op} = \frac{H_s}{L_{op}}$	$\frac{H_s}{A_c}$	0.1% exceedence values of coefficients in Eq (VI-5-186) α	β	Coefficient of variation	Reference
A	5.6 - 10.6	0.016 - 0.036	0.76 - 2.5	−0.026	0.051	0.21	Jensen (1984)
B	1.5 - 3.0	0.05 - 0.011	0.82 - 2.4	−0.016	0.025	0.46	
C	0.10	0.023 - 0.07	0.9 - 2.1	−0.038	0.043	0.19	Bradbury, et al. (1988)
D	0.14	0.04 - 0.05	1.43	-0.025	0.028		—
E	0.18	0.04 - 0.05	1.11	-0.088	0.011		—

Appendix D

Breakwater Overtopping Wave Analysis

Baffle Wall Wave Overtopping Flow Potential Summary

	Without SSP		Input Conditions												Eurotop Runup (Eqn 6.2)											Overtopping (Eqn 6.6)					
	Modeling Scenario without SSP	Event	Toe Elevation (ft)	LE (ft)	Toe Depth (ft)	Wave Period (sec)	Cot Structure Slope	Hmo (ft)	Hb	Crest Elev (ft)	Crest above toe (ft)	Gc (ft)	Rc(ft)	β (deg)	L0 (ft)	Iribarren Number, ξ_{0m}	γ_f rough	γ_f surging	γ_β	Perm	R2% (ft)	R2% max (ft)	R2% max check	Final R2% (ft)	Max Runup (ft)	γ_f mod	q (cfs/ft)	Cr (max)	Adjusted q (cfs/ft)	Length of Baffle Struct. (ft)	Armor Stone BW Overflow Rate (cfs)*
CLASS 1	South Breakwater	200 YR - 20 YR Wave, 10 YR WL	575	581.6	6.56	7	1.5	4.7	4.7	586	11	5.5	4.4	80	207.365	4.428	0.55	0.6942	0.5	No	9.936	5.692	Good	5.692	7.343	0.55	0.00357	0.52893	0.00189	50	0.094
CLASS 1	South Breakwater	200 YR - 10 YR Wave, 20 YR WL	575	582	7.03	6.7	1.5	4.9	4.9	586	11	5.5	4	80	189.972	4.151	0.55	0.6790	0.5	No	9.710	5.763	Good	5.763	7.435	0.55	0.01455	0.56821	0.00827	50	0.413
CLASS 3	North Breakwater	200 YR - 20 YR Wave, 10 YR WL	575	582.3	7.26	9.9	1.5	6.6	5.66	586	11	5.5	3.7	80	414.773	5.706	0.55	0.7643	0.5	No	15.425	7.746	Good	7.746	9.992	0.6135	0.13849	0.71287	0.09873	40	3.949
CLASS 3	North Breakwater	201 YR - 20 YR Wave, 10 YR WL	575	582.7	7.73	9.6	1.5	6.5	6.03	586	11	5.5	3.3	80	390.016	5.362	0.55	0.7455	0.5	No	15.434	7.996	Good	7.996	10.315	0.5826	0.30524	0.77889	0.23775	40	9.510

	With SSP		Input Conditions												Eurotop Runup (Eqn 6.2)											Overtopping (Eqn 6.6)						
	Modeling Scenario with SSP	Event	Toe Elevation (ft)	LE (ft)	Toe Depth (ft)	Wave Period (sec)	Cot Structure Slope	Hmo (ft)	Hb	Crest Elev (ft)	Crest above toe (ft)	Gc (ft)	Rc (ft)	β (deg)	L0 (ft)	Iribarren Number, ξ_{0m}	γ_f rough	γ_f surging	γ_β	Perm?	R2% (ft)	R2% max (ft)	R2% max check	Final R2% (ft)	Max Runup (ft)	γ_f mod	q (cfs/ft)	Cr (max)	Adjusted q (cfs/ft)	Length of Baffle Struct. (ft)	BW Flow with 1' SSP Added to crest elev - Flowrate (cfs)*	Reduced q Due to Baffle Louver Structure (cfs)
CLASS 1	South Breakwater	200 YR - 20 YR Wave, 10 YR WL	575	581.6	6.56	7	1.5	4.7	4.7	587	12	5.5	5.4	80	207.365	4.428	0.55	0.6942	0.5	No	9.936	5.692	Good	5.692	7.343	0.55	0.0004	0.5289	0.00020	50	0.0100	0.007
CLASS 1	South Breakwater	200 YR - 10 YR Wave, 20 YR WL	575	582	7.03	6.7	1.5	4.9	4.9	587	12	5.5	5	80	189.972	4.151	0.55	0.6790	0.5	No	9.710	5.763	Good	5.763	7.435	0.55	0.0018	0.5682	0.00105	50	0.0525	0.037
CLASS 3	North Breakwater	200 YR - 20 YR Wave, 10 YR WL	575	582.3	7.26	9.9	1.5	6.6	5.66	587	12	5.5	4.7	80	414.773	5.706	0.55	0.7643	0.5	No	15.425	7.746	Good	7.746	9.992	0.6135	0.0322	0.7129	0.02294	40	0.9177	0.642
CLASS 3	North Breakwater	200 YR - 10 YR Wave, 20 YR WL	575	582.7	7.73	9.6	1.5	6.5	6.03	587	12	5.5	4.3	80	390.016	5.362	0.55	0.7455	0.5	No	15.434	7.996	Good	7.996	10.315	0.5826	0.0761	0.7789	0.05930	40	2.3720	1.660

Design and assessment approach

$$\frac{R_{u2\%}}{H_{m0}} = 1.75 \cdot \gamma_b \cdot \gamma_f \cdot \gamma_\beta \cdot \xi_{m-1,0}$$

with a maximum of $\frac{R_{u2\%}}{H_{m0}} = 1.07 \cdot \gamma_{f\,surging} \cdot \gamma_\beta \left(4.0 - \frac{1.5}{\sqrt{\gamma_b \cdot \xi_{m-1,0}}}\right)$ 6.2

From $\xi_{m-1,0} = 1.8$ the roughness factor $\gamma_{f\,surging}$ increases linearly up to 1 for $\xi_{m-1,0} = 10$:
$\gamma_{f\,surging} = \gamma_f + (\xi_{m-1,0} - 1.8)*(1 - \gamma_f)/8.2$
With a maximum $R_{u2\%}/H_{m0} = 3.21$ for structures with an impermeable core and 2.14 for a permeable core.

$$\frac{q}{\sqrt{g \cdot H_{m0}^3}} = 0.1035 \cdot \exp\left[-\left(1.35 \frac{R_c}{H_{m0} \cdot \gamma_f \cdot \gamma_\beta}\right)^{1.3}\right]$$ for steep slopes 1:2 to 1:4/3 6.6

$C_r = 3.06 \exp(-1.5 G_c/H_{m0})$ with maximum $C_r = 1$ 6.8

$\gamma_\beta = 1 - 0.0063|\beta|$ for $0° \le |\beta| \le 80°$ 6.9
for $|\beta| > 80°$ the result $\beta = 80°$ can be applied

Assumptions

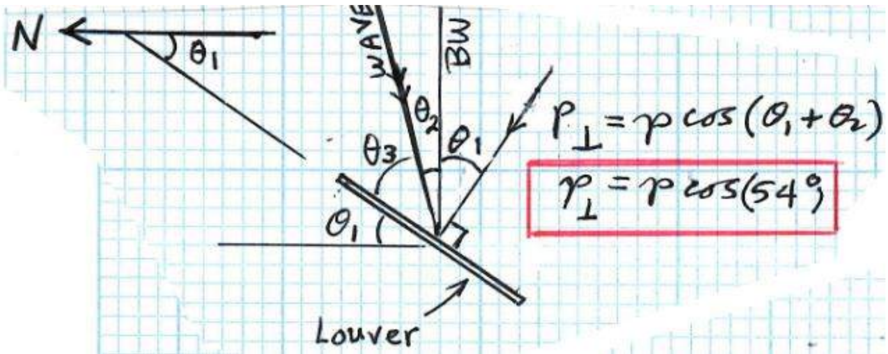
1. Waves are hitting at the toe of the breakwater at elev 575
2. Waves from north CLASS 3 (NNE)
3. Waves from south CLASS 1 (SSE)
4. Crest width Gc is half of width of breakwater assuming that is where Louvers are
5. crest height excludes louvre
6. Toe Elevation = Toe of the breakwater = 575

Note: Runup height on crest per FEMA Guidelines = 1.1' to 2.7'

Appendix E

Baffle Wall Structural Analysis

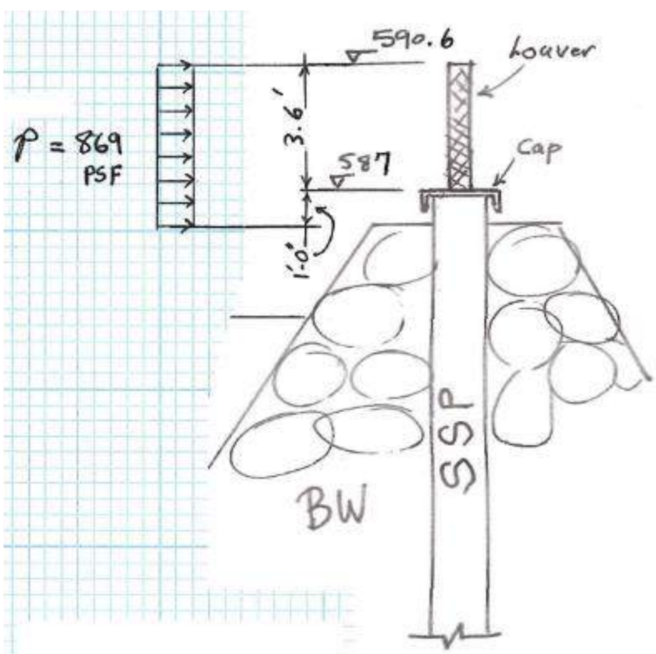
Waveload Analysis (see page 2R-7R):

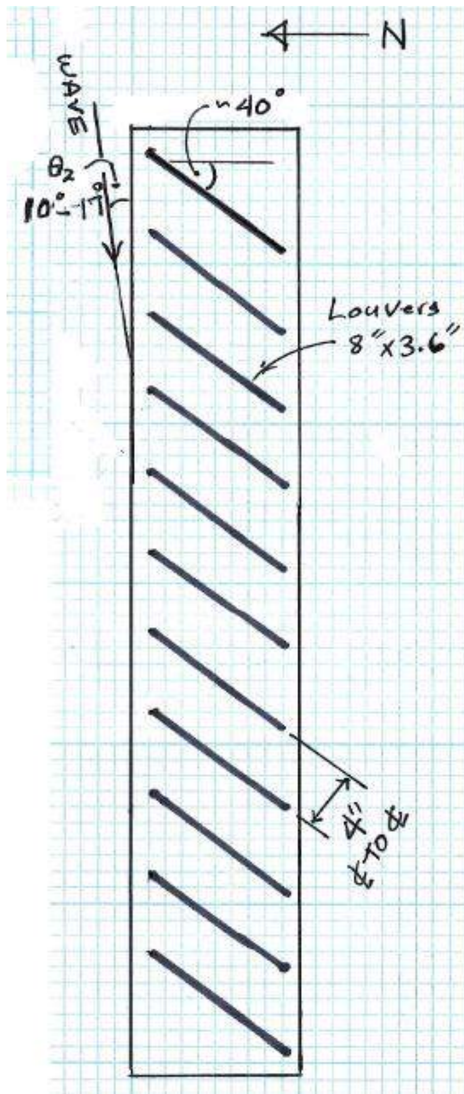


θ_2 =	14 °
θ_1 =	40 °
$\theta_3 = 90 - (\theta_2 + \theta_3)$ =	36 °

Wave pressure, p = 869 psf 20-yr, Class 3 (North B. W.)

Refer to page 8R - 12R for assumptions and louver wall design.





AECOM Calculations Cover Page, Preliminary

AECOM

Client Name				Project Number	
Project Name	Centennial Beach Breakwater and Lourvre Design				
Created by	Jeremy Mull, P.E.	Date	7-Feb-22	Page	1
Checked by	Bill Weaver, P.E.	Date	7-Feb-22	of	4

Subject

This workbook includes calcualtions of wave loading from coastal storm waves on the louvres, which will be installed along the top of the North and South Breakwater. The calculations follow the procedure outlined it the U.S. Army Corps of Engineerings (USACE) Coastal Engineering Manual (CEM) for waves that break on top of a rubble mound (like a breakwater) and the then overtopping bore impacts a seawall. Wave loading calculations are based on limited wave tank data and should be interpreted with caution.

Description

Version Updates:

Instructions:
1) Each sheet contains general inputs that are needed to complete the calculations, these cells are highlighted yellow.
2) Calculations are performed in cells that are not highlighted.

Purpose of Each Spreadsheet:

Wave Inputs
Summary of the wave inputs used in the wave loading calculations for each breakwater.

North Breakwater
Wave loading calcuations for the North Breakwater.

South Breakwater

North Breakwater

Class	Modeling Scenario	SWL (ft)	Breakwater Crest Elev (ft)	Depth at Breakwater Toe (ft)	Hb (ft)	L0 (ft)	Max Louvre Height (ft)	A_c (ft)	A_c (m)
CLASS 3	with Louver, 10 year	582.26	586	7.26	5.6628	390	4.6	3.74	1.14
CLASS 3	With Louver,20 year	582.73	586	7.73	6.0294	414	4.6	3.27	1.00

South Breakwater

Class	Modeling Scenario	SWL (ft)	Breakwater Crest Elev (ft)	Depth at Breakwater T	Hb (ft)	L0 (ft)	Maximum Lo	A_c (ft)	A_c (m)
CLASS 1	with Louver, 10 year	581.56	586	6.56	5.1168	190	4.6	4.44	1.35
CLASS 1	With Louver,20 year	582.03	586	7.03	5.4834	207	4.6	3.97	1.21

Bill's Earlier Version

North Breakwater

Class	Modeling Scenario	SWL (ft)	Breakwater Crest Elev (ft)	Depth at Breakwater T	Hb (ft)	L0 (ft)	Maximum Lo	A_c (ft)	A_c (m)
CLASS 3	with Louver, 10 year	582.26	586	7.26	6.6	390	4.6	3.74	1.14
CLASS 3	With Louver,20 year	582.73	586	7.73	6.5	414	4.6	3.27	1.00

NOTES/ASSUMPTIONS

Maximum louvre height along each breakwater was used

Although the Ac, wave steepness, and freeboard fall within range of the wave tank experiments, only a few ranges were tested

This analysis assumed depth limited breaking waves as no other information was provided

CEM Part VI - Wave Overtopping Force on Wall (North Wall)

Page VI-V-176 in CEM Part VI

Force Calculations: Method 1, Jensen and Bradbury (Equation VI-5-186)

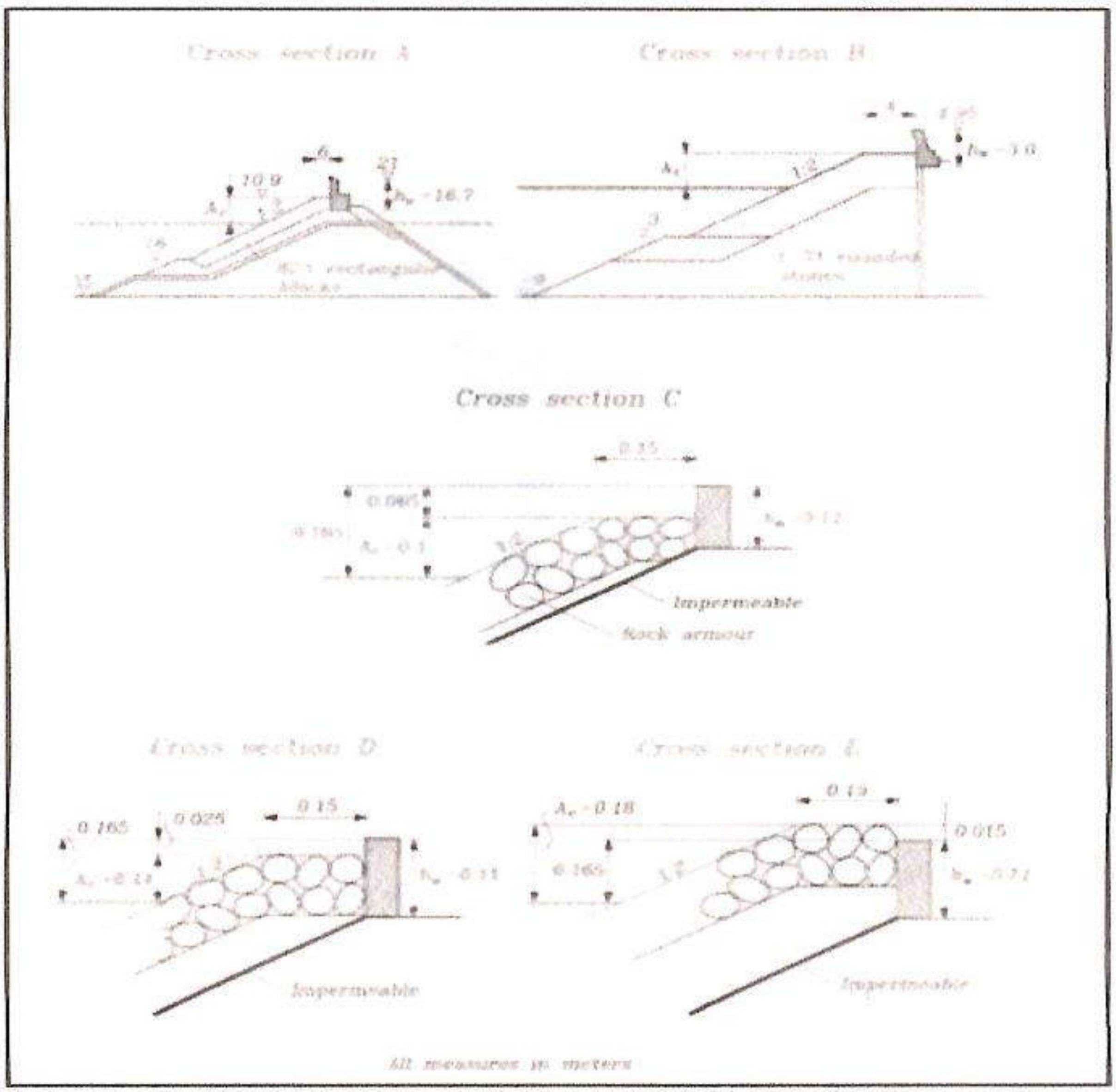
Step	Variable	Symbol	Class 3 10-Year	Class 3 20-Year
1	Mass Density of Water (slugs/ft^3 or kg/m^3)	ρ_w	1.9	1.9
2	Vertical Distance Between SWL and Berm Crest (ft or m)	A_c	3.74	3.27
3	Gravitational Acceleration (ft/s^2 or m/s^2)	g	32.174	32.174
4	Significant Wave Height at Breakwater Toe (ft or m)	H_s	5.7	6.0
5	Deepwater Wave Length at Peak Wave Period (ft or m)	L_0	390	414
6	Crown Wall Height (ft or m)	h_w	4.60	4.60 *
7	Wave Steepness	H_s/L_0	0.01	0.01
8	α	-	-0.016	-0.016
9	β	-	0.025	0.025
10	Hs/Ac	-	1.51	1.84
11	**Force on Unit Length of Wall (lb/ft or N/m)**	$F_{h,0.1\%}$	**2,447.03**	**3,577.49** *

$$\frac{F_{h,0.1\%}}{\rho_w\, g\, h_w\, L_{op}} = \alpha + \beta \frac{H_s}{A_C} \qquad \text{(VI-5-186)}$$

where
- $F_{h,0.1\%}$ — Horizontal wave force per running meter of the wall corresponding to 0.1% exceedence probability
- ρ_w — Mass density of water
- h_w — Crown wall height
- L_{op} — Deepwater wavelength corresponding to peak wave period
- H_s — Significant wave height in front of breakwater
- A_c — Vertical distance between MWL and the crest of the armor berm
- α, β — Fitted coefficient, see table

Cross section	Parameter ranges in tests A_c (m)	$s_{op} = \frac{H_s}{L_{op}}$	$\frac{H_s}{A_c}$	0.1% exceedence values of coefficients in Eq (VI-5-186) α	β	Coefficient of variation	Reference
A	5.6 - 10.6	0.016 - 0.036	0.76 - 2.5	−0.026	0.051	0.21	Jensen (1984)
B	1.5 - 3.0	0.005 - 0.011	0.82 - 2.4	−0.016	0.025	0.46	
C	0.10	0.023 - 0.07	0.9 - 2.1	−0.038	0.043	0.19	Bradbury, et al. (1988)
D	0.14	0.04 - 0.05	1.43	-0.025	0.028		
E	0.18	0.04 - 0.05	1.11	-0.088	0.011		

Notes
Fresh water
Maximum SWL along wall
ASCE 7-22 Equation 5.4-5, Assumes Breaking Waves
From CEM Table VI-5-60
From CEM Table VI-5-60
Calculated
Calculated



CEM Part VI - Wave Overtopping Force on Wall (South Wall)

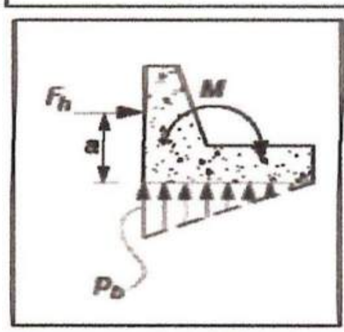
Page VI-V-176 in CEM Part VI

Force Calculations: Method 1, Jensen and Bradbury (Equation VI-5-186

Step	Variable	Symbol	Class 1 10-Year	Class 1 20-Year
1	Mass Density of Water (slugs/ft³ or kg/m³)	ρ_w	1.9	1.9
2	Vertical Distance Between SWL and Berm Crest (ft or m)	A_c	4.44	3.97
3	Gravitational Acceleration (ft/s² or m/s²)	g	32.174	32.174
4	Significant Wave Height at Breakwater Toe (ft or m)	H_s	4.7	4.9
5	Deepwater Wave Length at Peak Wave Period (ft or m)	L_0	190	207
6	Crown Wall Height (ft or m)	h_w	4.60	4.60 *
7	Wave Steepness	H_s/L_0	0.02	0.02
8	α	-	-0.016	-0.016
9	β	-	0.025	0.025
10	Hs/Ac	-	1.06	1.23
11	**Force on Unit Length of Wall (lb/ft or N/m)**	$F_{h,0.1\%}$	**570.76**	**884.53** *

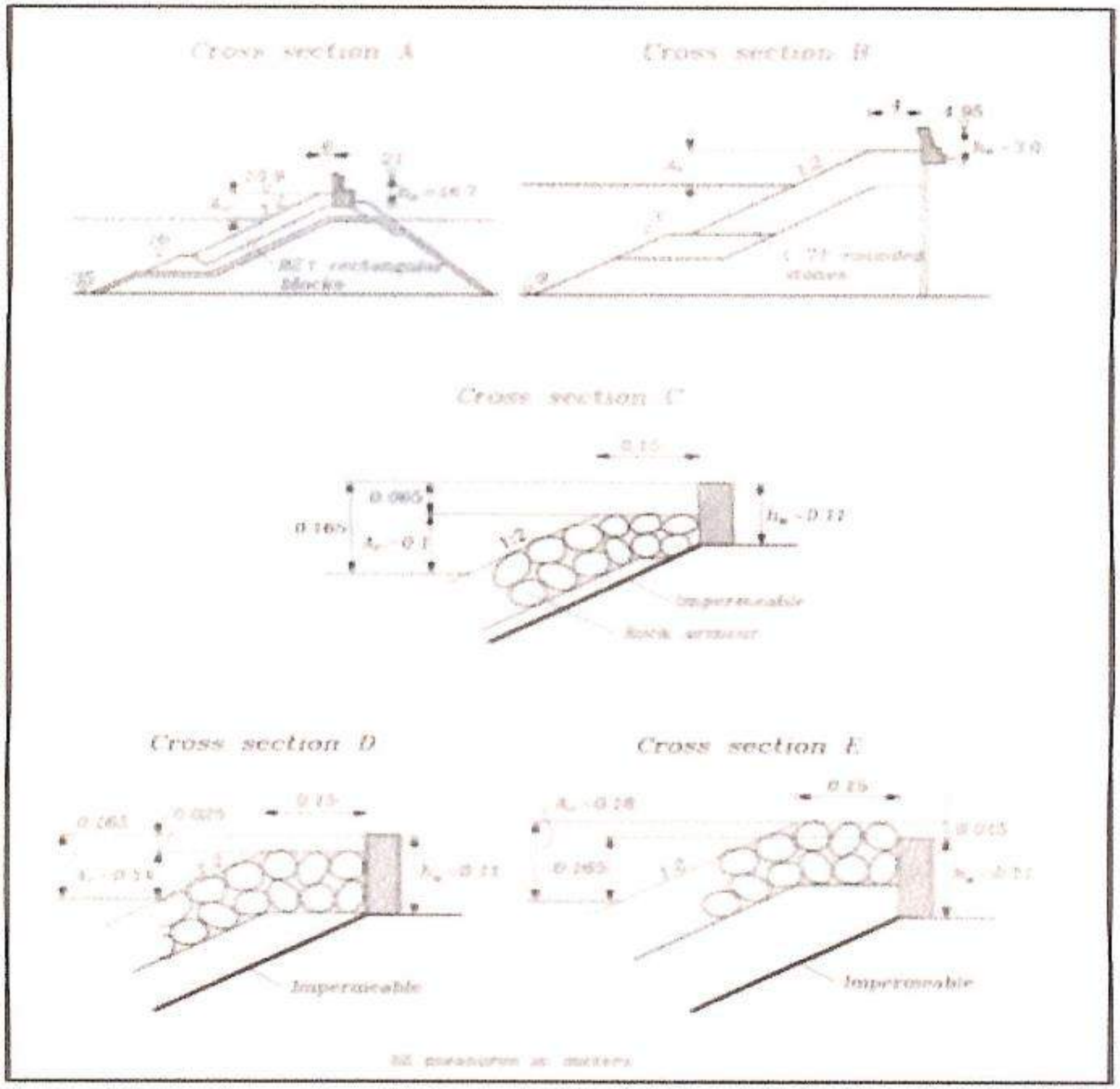
$$\frac{F_{h,0.1\%}}{\rho_w\, g\, h_w\, L_{op}} = \alpha + \beta \frac{H_s}{A_C} \qquad (VI-5-186)$$

where
- $F_{h,0.1\%}$ Horizontal wave force per running meter of the wall corresponding to 0.1% exceedence probability
- ρ_w Mass density of water
- h_w Crown wall height
- L_{op} Deepwater wavelength corresponding to peak wave period
- H_s Significant wave height in front of breakwater
- A_c Vertical distance between MWL and the crest of the armor berm
- α, β Fitted coefficient, see table



Cross section	Parameter ranges in tests			0.1% exceedence values of coefficients in Eq (VI-5-186)		Coefficient of variation	Reference
	A_c (m)	$s_{op} = \frac{H_s}{L_{op}}$	$\frac{H_s}{A_c}$	α	β		
A	5.6 - 10.6	0.016 - 0.036	0.76 - 2.5	−0.026	0.051	0.21	Jensen (1984)
B	1.5 - 3.0	0.05 - 0.011	0.82 - 2.4	0.016	0.025	0.46	
C	0.10	0.023 - 0.07	0.9 - 2.1	−0.038	0.043	0.19	Bradbury, et al. (1988)
D	0.14	0.04 - 0.05	1.43	-0.025	0.028		
E	0.18	0.04 - 0.05	1.11	-0.088	0.011		

Notes
Fresh water
Maximum SWL along wall
ASCE 7-22 Equation 5.4-5, Assumes Breaking Waves
From CEM Table VI-5-60
From CEM Table VI-5-60
Calculated
Calculated



Assumptions:

- The north breakwater is to be designed as "Class 3", for a 20-year wave attack and wves making 14 degrees with breakwater, as shown on page 1R. This governs the G347 design of louvers.
- The south breakwater is to be designed as "Class 1" for a 20-year wave and for waves making similar angle but opposite direction. This does not govern design of the louvers.
- Per analysis on page 4R, the force applied to a 4.6' wall = 3,578 lb/ft above crest of north breakwater:

  p = 3,578/4.6 = 778 psf

  This pressure has a component perpendicular to the louver surface:

  $p_{\perp}$ = 778*cos(54°) = 457 psf (see page 1R, to be used for design of louvers)
- Louver wall is 3.6' high above SSP wall, which extends 1' above crest of breakwater.
- The calculated force of 3,578 lb/ft of wall length will be resisted by the total wall height of 4.6' (1' SSP wall extension above breakwater + 3.6' louvers).
- Louver panels will consist of 8" wide x 3.6' long steel plates welded to 1" thick x 12" wdie base plates, which will be bolted to a steel cap (channel) at top of SSP wall.
- The spacing of the louver plates is 4" on center. The analysis and design will ignore the louver spacing, which allows for some wave water to flow through and result in pressure reduction (conservative).
- Ignore the bottom foot of the wall because it is a solid SSP and very capable of resisting the wave forces.
- Wind pressure is estimated to be ~ 30 psf << wave pressure 457 psf. Therefore, the wind pressure load case does not govern.

Louver Wall Design:

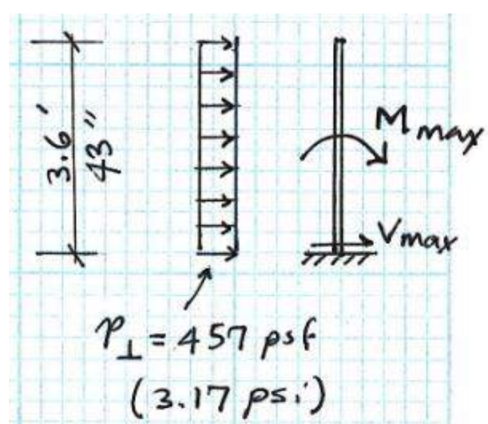
Try 3/8" thick louvers x 8" wide, fixed at bottom (welded to base plate, which will be bolted to SSP cap).

Use 1" vertical strip of louver wall to calculate flexural and shear stresses per AISC Standard Specifications (14th Ed.).

Sx = $(3/8)^2$*1/6 = 0.0234 in^3

Mmax = 3.17*$(43.2)^2$/2 = 2958 in-lb/in

Vmax = 3.17*43.2 = 137 lb/in



Try A36 steel (Fy = 36 ksi):

Fb = Mmax/Sx = 126.2 ksi

\>>

Fb = 36/Ω = 21.6 ksi NOT GOOD

Ω = 1.67

Re-analysis Louvers with Revised Assumptions:

- Assume louver plates are 5/8" thick, Grade 50 steel.
- Reduce wave pressures by 40% due to refraction of wave because of the nature of the multiple plate (louver wall) as it compares to smooth flat plate, per Bill Weaver's wave analysis.

$Sx = (5/8)^2 * 1/6 =$ 0.065 in^3

Mmax = 0.6*2,958 = 1775 in-lb/in

Fb = (2,958*0.6)/(1,000*0.065)

Fb = 27.3 ksi

<

Fb = 50/Ω = 30 ksi OKAY

Ω = 1.67

$Fv = (137*0.6)/(0.625*10^3)$

Fv = 0.13 ksi

<<

Fv = (0.6*50)/1.5

Fv = 20 ksi OKAY

Use 5/8" louver plates, Grade 50 steel, 8" wide x 3.6' high.

Design Louver Panel and Connections:

Weld:

Try 3/8" fillet weld to base plate:

e = 0.625" + 2*(0.375/2)

e = 1 in

C = T = Mmax/1" = (2,958*0.6)/(1,000*1)

C = T = 1.8 k/in

Allowable weld strength,

$R_n/\Omega = F_{n,BM}*A_{BM}/\Omega$ (based on base material)

Nominal strength of base material, $F_{n,BM}$ = 36 ksi

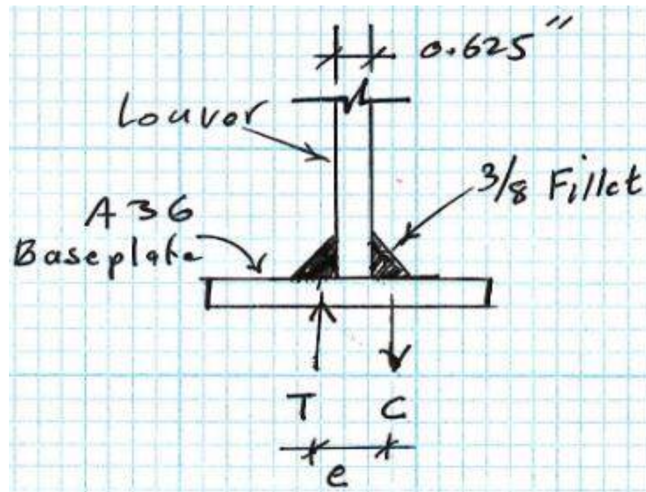
Effective area of base material, A_{BM} = 0.375"*1"

Ω = 2

R_n/Ω = 6.8 k/in

\>

C = T = 1.8 k/in OKAY



Allowable weld strength,

$R_n/\Omega = F_{n,w}*A_w/\Omega$ (based on weld strength; AISC J2-3)

Nominal strength of weld, $F_{n,w}$ = 70 ksi

Effective area of weld, A_w = 0.375"*cos(45°)/2 (fillet angle)

Ω = 2

R_n/Ω = 10.1 k/in

\>

C = T = 1.8 k/in OKAY

Note weld size is a bit overside to account for long-term fatigue and possible corrosion with time.

Ice Loading:

Reference: US Corp. of Engineers (USACE) EM 1110-2-1612, "Engineering and Design - Ice Engineering"

Per Section 2-3, ice breakthrough load (allowable P), floating ice sheet:

$P = A*h^2$

A = 1/16 for most practiced purposes

P = ice load in tons

h = thickness of ice sheet

Assume a 12" thick ice sheet is floating by waves attacking the louver wall at mid height:

$P = (1/16)*(12)^2 =$ 9 tons

P = 18 kips

Characteristic length, Lc, of a floating ice sheet can be assumed to be 15 to 20 times thickness of ice for freshwater (Section 2-3C):

Lc = 18 ft (average value)

or

Ice force, Pi = (18 kips)/18')cos(54°)*(1000/12 in/ft)

Pi = 49 k/in

$Mmax = (3.17*(16^2/2)*1.0) + (49*22)$

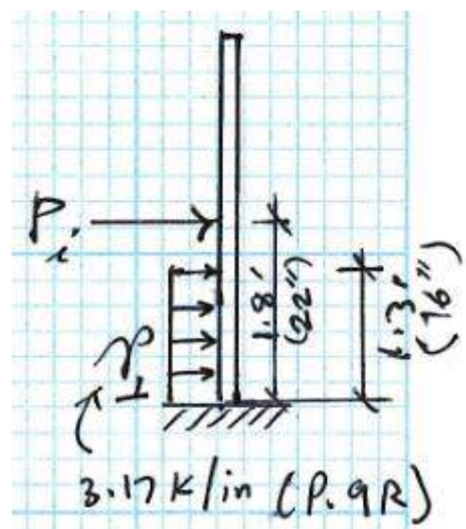
Mmax = 1483 in-lb/in

Fb = (1483/1000)/0.065

Fb = 22.8 ksi

<

Fb = 30 ksi OKAY

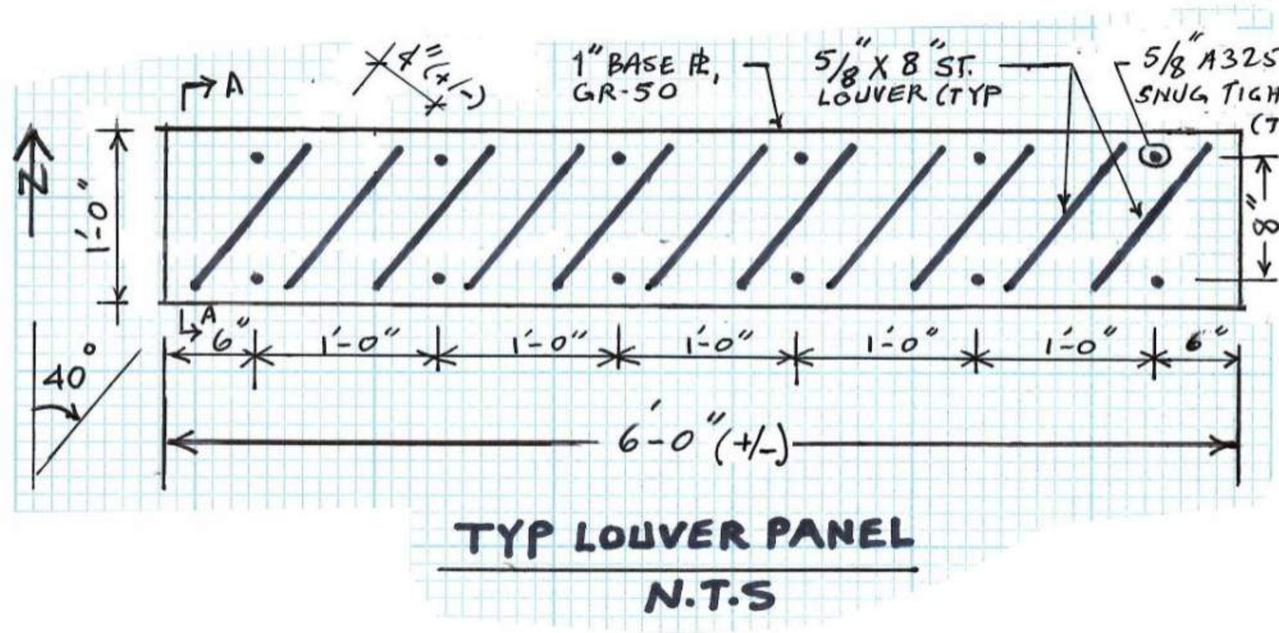


Notes:

1) Shear stress is okay by inspection.
2) No reduction was considered due to wave refraction/deflection, since water was assume dto be confined by the ice sheet, conservative.

Bolts for Base Plate to SSP Cap:

Assume 12" wide base plate bolted to SSP on both sides of louver plate.

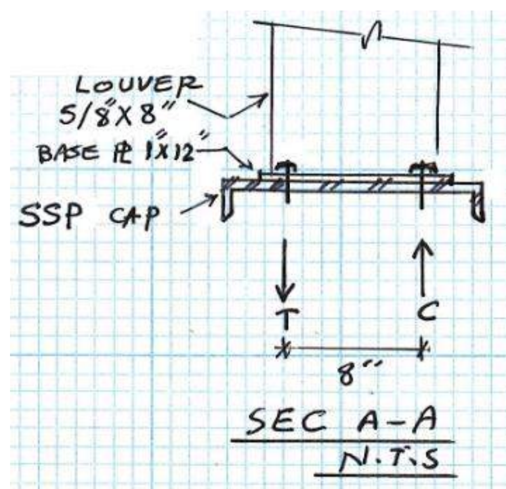


Mmax = 1775 in-lb/in (page 9R)

Bolt spacing, s = 12 in

C = T = (1775/1000)*(8"/8")*2 louvers/ft

C = T = 3.6 k/bolt



Per Table J3.1 (AISC, 14th Ed.), A325 bolts are in Group A.

Allowable tension, rn/Ω = 13.8 kips for 5/8" dia. A325

(Table 7-2, AISC, 14th Ed.)

>

C = T = 3.6 k/bolt OKAY

Use 5/8" dia. A325 snug tight @ 12" spacing, per layout shown above, H.D., galvanized.

Note bolts are a bit oversized to allow for future corrosion, cross-sectional area loss, and fatigue stresses.