TATHAM ENGINEERING

645 Veterans Drive, Unit D
Barrie, Ontario L4N 9H8

T 705-733-9037
E info@tathameng.com
tathameng.com

File 424468

January 29, 2025

Ken Szekely
2213 North Service Road
Mississauga, Ontario L5K 1A3
kszekely@engplastics.com

Re: 125,127 & 129 Delphi Lane, Town of the Blue Mountains
Shoreline Hazard Assessment

Dear Ken:

Tatham Engineering Limited (Tatham) has been retained to complete a shoreline hazard assessment for the subject property located at 125, 127 & 129 Delphi Lane in the Town of the Blue Mountains (Town). The subject property, which is an amalgamation of three separate lots, is currently developed with various structures, however only one single-family dwelling is within 100 m of the shoreline and fronts onto Georgian Bay. We understand you are proposing to redevelop the property with a single-family dwelling.

The subject property is regulated by the Grey Sauble Conservation Authority (GSCA) for shoreline hazards. The GSCA have indicated a shoreline hazard assessment is required to determine the extent of shoreline hazards on the subject property and specify any measures required to meet the floodproofing, protection works and access standards for the proposed development.

This letter report summarizes the findings regarding flooding and erosion hazards at the subject property, and the recommended measures to mitigate the risk following guidelines and recommendations from the Technical Guide for Great Lakes – St. Lawrence River Shorelines (2001), Ministry of Natural Resources and Forestry (MNRF).

GUIDELINES AND BACKGROUND INFORMATION

The assessment was prepared recognizing provincial guidelines on natural hazards, including the following publications:

- Grey Sauble Shoreline Management Plan, Sandwell Inc (1994);
- Grey Sauble Conservation Authority, Policies for the Administration of the Development, Interference with Wetlands and Alterations to Shorelines and Watercourse Regulation, Ontario Regulation 151/06 (2009) (under review);

- Grey Sauble Conservation Authority, Interim Policy Guidelines for the Administration and Implementation of Ontario Regulation 41/24 Prohibited Activities, Exemptions and Permits (March 2024);
- The Ontario Ministry of Natural Resources & Forestry (MNRF) Technical Guide for the Great Lakes – St. Lawrence River System and Large Inland Lakes (2001); and
- EurOtop Manual on wave overtopping of sea defences and related structures, Second Edition (2018).

Information relating to existing topography, ground cover and shoreline conditions was obtained through a review of relevant background studies, available plans and base mapping, along with a site visit conducted on October 16, 2024.

Van Harten Survey Inc. completed a topographic survey of the property on October 18, 2024, which was provided for our use.

SITE INSPECTION

Tatham conducted a site visit on October 16, 2024, to review the existing shoreline conditions.

The subject property is located on the east side of Delphi Point on the shores of Georgian Bay and has approximately 92 m of north facing shoreline frontage. The shoreline is protected with a boulder revetment with boulders ranging in diameter between 600 to 1500 mm. There is no toe boulder embedment because the revetment is built on bedrock.

There is a large L-shaped boulder breakwater structure extending approximately 15 m perpendicular from the western property line and then runs parallel to the shore for approximately 70 m. A similar, but smaller L-shaped boulder breakwater structure protects the east end of the shoreline and extends approximately 11 m both perpendicularly and parallel to the shore. Both breakwater structures are built on bedrock and consist of boulders with diameters ranging from 300 to 1700 mm. Large boulders approximately 1200 to 2000 mm make up the main structures of the breakwaters with the smaller boulders being used to fill the voids.

On the west side of the shoreline there is a grass lawn and sand beach area which transitions into a bedrock bottomed basin. The sand beach is bordered by boulders and there is no visual evidence of erosion or sediment transport around the beach area. During the time of inspection there was no water in the basin area leaving the bedrock exposed. The entrance to the basin is at the east end of the shoreline between the two breakwater structures and faces east.

There is an existing flagstone patio and pergola covered sitting area along the shoreline in front of the mouth of the basin and the immediate backshore area is a grass lawn. Further inland is a row of mature trees built on an earth berm and the existing house, detached garage and shed.

Select photos from the site visit are provided in Appendix A.

FLOOD HAZARD

The flood hazard limit is estimated based on the 100-year high water level for the site and the wave uprush elevation. The 100-year flood level for the subject property has been determined by the MNRF as 177.9 m (Geodetic Survey of Canada Datum)[1]. A 15 m horizontal allowance to account for wave uprush is then applied to define the horizontal offset for the default flood hazard limit. This limit applies unless a site-specific study is completed to refine the flood hazard limit. The site-specific flood hazard limit we have calculated herein will, therefore, be used to define the flood hazard for the subject property.

To determine the governing wave uprush elevation, Tatham carried out an extreme value analysis of offshore wave conditions with historical wind and wave data from the Environment Canada wave buoy (ID No.: C45143) seasonally located in Georgian Bay. The MNRF recommends a 10- to 20-year return period wave height coincident with the 100-year high water level to determine wave uprush, and for the purposes of delineating the flood hazard limit. For this assessment, we have used the estimated 1:20-year wind velocity and wave height.

The breakwater structure, located approximately 17 m from the natural shoreline, has a slope of 2.5H:1V. Waves will break on the front face of the structure and then overtop it. While the natural shoreline, which has a slope of 1.4H:1V, will still be impacted by wave action, the wave energy will have dissipated after breaking on the offshore structure, providing protection to the natural shoreline. For this assessment, the wave conditions were first analyzed as they hit the breakwater, and then the subsequent overtopping wave was used for the analysis on the natural shoreline to establish the wave uprush limit attenuated by the breakwater. We have also prepared an “unprotected shoreline” condition calculation to assess the affect of the breakwater structure and establish the flood hazard limit for the property.

A Delft3D wave hindcasting model was used to estimate the nearshore wave conditions for the northernly (350 degrees) wave conditions. The analysis computed a significant wave height of 2.05 m with a period of 7.4 s. These values were used to calculate the transformed wave conditions at the breakwater, accounting for wave transformation processes, such as shoaling, refraction and wave breaking effects. The nearshore depth-limited wave height was estimated to be 1.05 m. The resultant flood hazard elevation is estimated to be 180.2 m, however this is a theoretical value as water will overtop the breakwater and propagate inland towards the revetment along the natural shoreline.

Following the methods outlined in EurOtop manual, the transformed wave height was used to calculate the transmission coefficient, which in turn yielded the transmitted wave height. With a shoreline crest width of approximately 3 m, the nearshore depth-limited wave height was estimated to be 0.23 m on the

[1] MNRF. (2001), Figure 3.9, page 3-15, Part 3, Great Lakes – St. Lawrence River System and Large Inland Lakes Technical Guide.

leeside of the breakwater. The methods recommended by the MNRF were then used to compute wave uprush and wave bore distances. The flood hazard elevation is estimated to be 178.6 m and the estimated wave bore propagation length past the crest of the shoreline revetment has been estimated as 4.3 m. Estimates based on this method are approximate and used as a guide to estimate the level of flood risk for a property and not an absolute limit. The backshore area of the property is extensive and low-lying so waves will dissipate as they propagate inland before meeting the flood hazard elevation, therefore, bore distances were used to delineate the flood hazard limit for this assessment.

While the bore distance calculated for the natural shoreline was used for the majority of the property, the bore distance from the crest of the breakwater structure at the west end of the shoreline is based on the methods proposed by Cox and Machemehl (1986) and FEMA (1989), as outlined in the MNRF Great Lakes technical guide. This was done as the breakwater connects to the shoreline and therefore there is no natural shoreline in this section and therefore this method was selected to determine the bore distance from the breakwater across the grass lawn. The FEMA method suggests the design waves may propagated inland 17.2 m from the crest of the breakwater structure, where there are no obstructions.

A summary of the wave uprush calculations is presented in Table 1 and supporting calculations are included in Appendix B. The estimated wave uprush limit attenuated by the breakwater is shown on Drawing SH.1.

The MNRF Technical Guidelines indicate the regulated limits are to be mapped on the assumption no shoreline protection works are in place. The breakwater is considered shoreline protection and therefore the “unprotected shoreline” condition was assessed to determine the flood hazard limit for the subject property in the absence of the breakwater structure. Where necessary, shoreline protection is used to address the hazard when considering development regulations. The same methods above were used, however, the initial Delft3D values were used against the natural shoreline. The resultant flood hazard limit is mapped on Drawing SH.1.

Development should be directed outside of the areas exposed to wave action, however, where this is not practicable and the risk to development is low and may be readily mitigated, limited development is typically permitted, however, it is generally required the proposed development maintain the minimum setbacks of the existing dwelling as far as practicable such the risk does not increase relative to the existing dwelling. The recommended mitigation and floodproofing measures are outlined in the following sections.

We note GSCA’s Shoreline Management Plan also identifies flood hazard elevations for the Georgian Bay shoreline within its jurisdiction. The subject property is located in Reach G-4 of this plan and the wave uprush elevation is given as 179.3 m. This is higher than the value calculated as part of this assessment and the difference is attributed to the variability in the shoreline arrangement within the reach.

Table 1: Wave Parameters

PARAMETER	BREAKWATER STRUCTURE	REVETMENT STRUCTURE
100-year High Water Level, SWL	177.9 m	
Significant Wave Height, H_{0s}	2.05 m	0.23 m
Wave Period, T_p	7.4 s	
Transformed Wave Height, H_{ss}	1.05 m	0.23 m
Shoreline Structure Slope	2.5H:1V	1.4H:1V
Shoreline Structure Crest Elevation	178.5 m ±	177.9 m ±
Approximate Wave Uprush Elevation	180.2 m	178.6 m
Wave Bore Length from Crest of Structure	17.2 m	4.3 m
Unprotected Shoreline Condition Wave Bore Length	-	15.7 m
Floodproofing Elevation	178.9 m	

Floodproofing for the Proposed Development

As noted, development adjacent to the Georgian Bay shoreline at the subject property is within the GSCA's regulated area and requires approval from the Conservation Authority. GSCA has established policies and guidelines outlining the requirements for approval under Section 28 of the Conservation Authorities Act.

The minimum floodproofing elevation is typically taken as 0.3 m above the wave uprush elevation. We note the existing dwelling has a surveyed finished floor elevation of 178.5 m. The wave uprush elevation was calculated to be 178.6 m, which means the recommended finished floor elevation for any proposed development is 178.9 m. There are no conceptual site plan drawings to review, however, we understand the existing dwelling is to be demolished and a new single-family dwelling is to be constructed. The new dwelling shall be floodproofed to a minimum opening elevation of 178.9 m and shall not encroach further lakeward than the existing dwelling.

EROSION HAZARD

The erosion hazard limit includes the cumulative impact of the stable slope allowance plus 100-year recession allowance.

Station H-36 in the GSCA's Shoreline Management Plan is located east of the subject property and has an average annual recession rate of -0.05 m/yr, which indicates accretion of sediments. As with the wave

uprush limit value provided in the Shoreline Management Plan, there is variability in the shoreline arrangement with the erosion monitoring station located in a minor embayment between Delphi Point and Craigleith Provincial Park. The subject property is located on the headland of Delphi Point and from available aerial imagery from Grey County GIS does not show any significant shoreline accretion or recession over the period of 2006 – 2020.

Additionally, from the visual site inspection, there was no evidence of erosion damage or slope failure across the subject shoreline. Bedrock is the limiting substrate of the shoreline and only the overburden veneer is subject to erosion, which is protected by the breakwater and revetment structures. Therefore, the flood hazard limit is the governing hazard for the subject site.

DYNAMIC BEACH HAZARD

The subject shoreline has a bedrock substrate and therefore a dynamic beach hazard does not apply and was not assessed.

SHORELINE DEVELOPMENT POLICIES

As noted, development along the Georgian Bay shoreline in this region requires approval from GSCA. GSCA has established policies and guidelines outlining the requirements for approval under the Conservation Authorities Act.

Development adjacent to the Georgian Bay shoreline flood or erosion hazard may be permitted provided, among other policy requirements, the control of flooding, erosion, dynamic beaches, pollution or the conservation of land is not adversely affected, a minimum 6 m maintenance access allowance is retained along existing or proposed shoreline structures. The 6 m maintenance access allowance is mapped on Drawing SH.1.

Development within the flood hazard may be supported in the area of least and acceptable risk with the appropriate mitigation measures implemented to achieve the Provincial floodproofing, protection works and access standards. It is our opinion floodproofing and protection works standards are achieved considering the existing breakwater structure and the recommendations included in this report are implemented, however, it is expected maintaining the shoreline setbacks of the existing dwelling will be required by the GSCA.

SUMMARY AND RECOMMENDATIONS

Tatham used the methods recommended the MNRF and methods outlined in EurOtop Manual to establish the shoreline hazards present across the subject property. The wave uprush and flood hazard limits were calculated based on wave bore distances as the backshore area is flat and low lying for an unprotected shoreline and considering the affects of the breakwater structure. Drawing SH.1 indicates the flood hazard

limit for the subject property and the estimated limit of wave uprush considering the protection offered by the breakwater structure.

Based on the assessment the recommendations for the proposed development are as follows:

1. The proposed development should not extend lakeward of the existing house; best efforts should be made to locate the new dwelling outside of the flood hazard limit delineated on Drawing SH.1.
2. A minimum 6 m clear maintenance access allowance should be provided behind the existing shoreline revetment structure to allow for future maintenance and/or improvements of the shoreline structure.
3. The minimum finished floor and minimum opening elevations should be at or above 178.9 m.

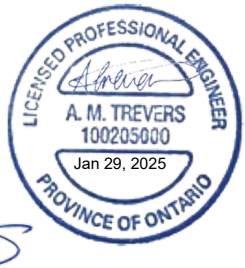
This report should be reviewed by GSCA to confirm their agreement with the findings of our report.

Tatham should be forwarded the site and grading plans to confirm the recommendations of this report have been appropriately incorporated and the conclusions remain valid.

CLOSING

We trust this letter meets your needs. Should you have any questions or comments regarding this assessment, please do not hesitate to contact the undersigned.

Yours truly,
Tatham Engineering Limited



Maggie Ryan, B.A.Sc.
Engineering Intern

Andrew Trevers, M.Eng., P.Eng.
Senior Engineer, Project Manager

MMR/AMT:ha

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Appendix A:
Photo Appendix

Figure 1: Front of existing house to demolish.

Figure 2: Patio area at shoreline.

Figure 3: Smaller groyne boulder structure at east end of shoreline.

Figure 4: Bedrock basin bordered by shoreline revetment and large groyne boulder structure; sand beach at end of basin.

Figure 5: Grass lawn landward of the boulder revetment.

Figure 6: View of boulder revetment and groyne structure from sand beach at west end of shoreline.

Figure 7: Shoreline revetment built on bedrock.

Figure 8: Boulder border of sand beach at end of basin.

Figure 9: Grass lawn west of sand beach at west end of shoreline; protected by boulder groyne on west and north sides.

Figure 10: Wetland and bedrock west of property.

Figure 11: View from top of boulder groyne structure.

Figure 12: Entrance to basin between ends of boulder groyne structures.

Figure 13: Entrance to basin between ends of boulder groyne structures.

Figure 14: Smaller groyne boulder structure at east end of shoreline.

Figure 15: Cobble beach east of property.

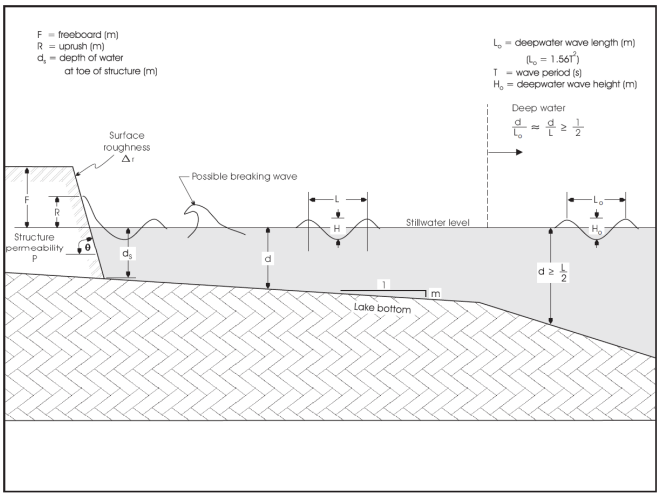
Figure 16: Land berm parallel to shoreline, landward of top of revetment structure, west of existing house.

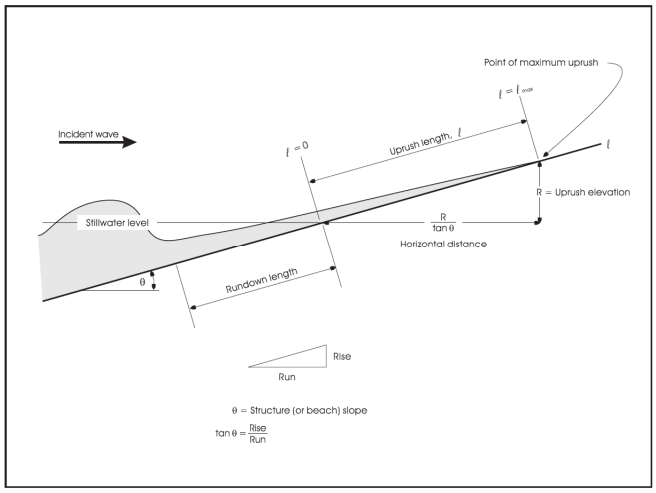
Figure 17: Land berm parallel to shoreline, landward of top of revetment structure, west of existing house.

Figure 18: Grass lawn shore access along west side of property.

Appendix B:
Supporting Calculations

Wave Uprush Calculations:





Deep water wave conditions:

$H_{0S} := 2.05\ m$ $\quad T_p := \frac{7.4\ s}{0.9}$ $\quad d_{Deep} := 3.22\ m$ (Results from DELFT3D[W=20.1m/s@350°->20YR])

$SWL := 177.9\ m$ (100yr return period SWL - MNR)

$L_0 := \frac{g \cdot T_p^{\,2}}{2 \cdot \pi} = 105.5162\ m$ $\quad \frac{d_{Deep}}{L_0} = 0.0305$ "NO Deep Water" (Exact L.Deep = 44.7233 m)

Refracted conditions:

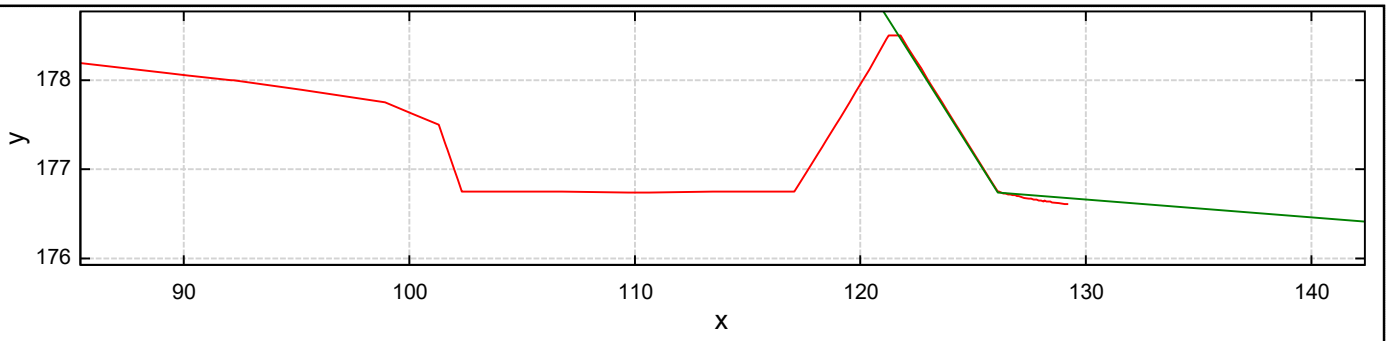
$Kr := 1.0$

$H'_{0S} := Kr \cdot H_{0S} = 2.05\ m$

$H'_{0m} := 0.626 \cdot H'_{0S} = 1.2833\ m$ $\quad T_m := 0.9 \cdot T_p = 7.4\ s$

$L_{0m} := \frac{g \cdot T_m^{\,2}}{2 \cdot \pi} = 85.4681\ m$

Slopes: $m := \frac{1}{50}$ $\quad m_s := \frac{1}{2.5} = 0.4$ $\quad Y_{toe} := 176.74\ m$



$X_{toe} = 126.1\ m$ $Y_{toe} = 176.74\ m$ $m_s = 0.4$ $d_s := SWL - Y_{toe} = 1.16\ m$

→1 / 2.5

Wave shoaling considering wave breaking:

Deep water:

$H'_{0S} = 2.05\ m$ $T_p = 8.2222\ s$ $d_{Deep} = 3.22\ m$ $L_0 = 44.7233\ m$ $k_0 := \frac{2 \cdot \pi}{L_0} = 0.1405 \cdot \frac{1}{m}$

$$n_0 := \frac{1}{2} \cdot \left(1 + \frac{2 \cdot k_0 \cdot d_{Deep}}{\sinh\left(2 \cdot k_0 \cdot d_{Deep}\right)}\right) = 0.9378$$

Toe:

$d_s = 1.16\ m$

$$L_s := \text{solve}\left(L\ m = \frac{g \cdot T_p^{\,2}}{2 \cdot \pi} \cdot \tanh\left(\frac{2 \cdot \pi \cdot d_s}{L\ m}\right), L, 1, 200\right) m = 27.4122\ m \qquad k_s := \frac{2 \cdot \pi}{L_s} = 0.2292 \cdot \frac{1}{m}$$

$$n_s := \frac{1}{2} \cdot \left(1 + \frac{2 \cdot k_s \cdot d_s}{\sinh\left(2 \cdot k_s \cdot d_s\right)}\right) = 0.9772$$

Shoaling Coefficient:

$$Ks := \sqrt{\frac{n_0 \cdot L_0}{n_s \cdot L_s}} = 1.2513$$

Depth-limited wave height - wave breaking (Weggel, 1972):

$a := 43.8 \cdot \left(1 - \exp\left(-19 \cdot m\right)\right) = 13.8469$

$$b := \frac{1.56}{\left(1 + \exp\left(-19.5 \cdot m\right)\right)} = 0.9302$$

$$H_b := \text{solve}\left(\frac{H_b\ m}{d_s} = b - a \cdot \frac{H_b\ m}{g \cdot T_p^{\,2}}, H_b, 0.5 \cdot d_s\ m^{-1}, 1.6 \cdot d_s\ m^{-1}\right) m = 1.0535\ m$$

$\frac{H_b}{g \cdot T_p^{\,2}} = 0.0016$ $\frac{H_b}{d_s} = 0.9082$ $m = 0.02$

$H_{sS} := \min\left(\begin{bmatrix} H_b \\ Ks \cdot H'_{0S} \end{bmatrix}\right) = 1.0535\ \text{m}$

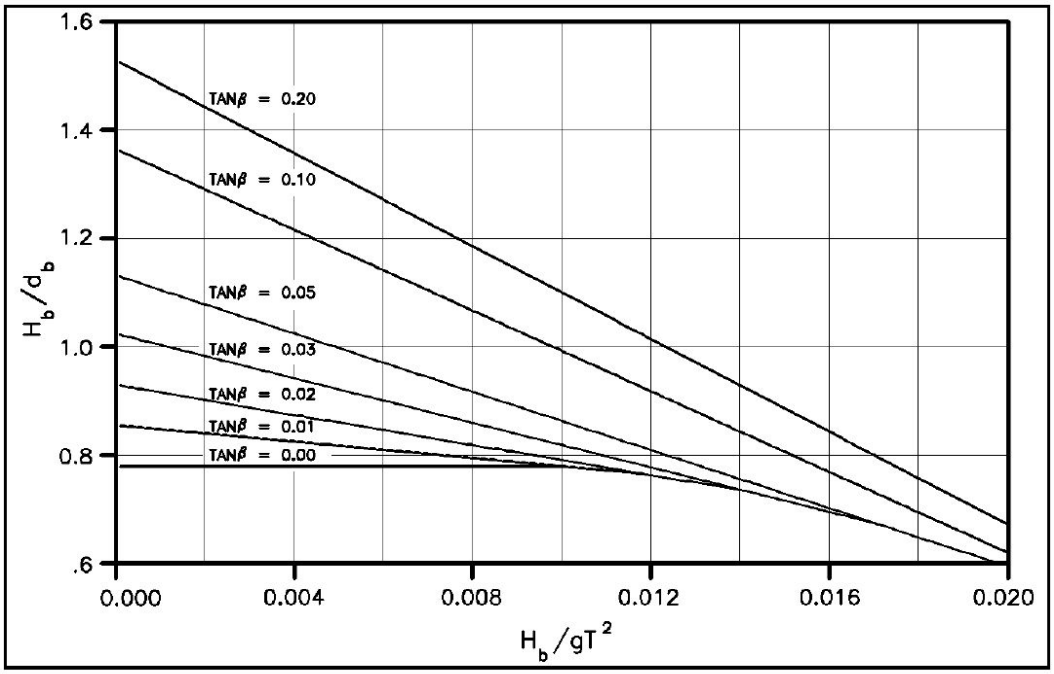


Figure II-4-2. Breaker depth index as a function of $H_b/(gT^2)$ (Weggel 1972)

Wave uprush:

Van Gent (2001):

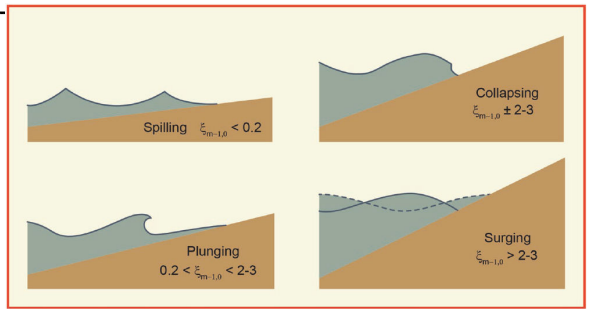
$\xi_S := \frac{m_s}{\sqrt{\frac{H_{sS}}{L_0}}} = 2.6062$ $\gamma_\beta := 1.0$ $\gamma_f := 0.7$ "Single layer revetment"

$R_{2\%} := \gamma_f \cdot \gamma_\beta \cdot H_{sS} \cdot \begin{pmatrix} \text{if } \xi_S \le 1.74 \\ \quad 1.35 \cdot \xi_S \\ \text{else} \\ \quad 4.7 - \frac{4.09}{\xi_S} \end{pmatrix} = 2.3087\ \text{m}$

Type of Wave Braking (EuroTop2018):

$\xi_S := \frac{m_s}{\sqrt{\frac{H_{sS}}{L_0}}} = 2.6062$

$$\xi_S := \frac{m_s}{\sqrt{\frac{2 \cdot \pi \cdot H_{sS}}{g \cdot T_m^{\,2}}}} = 3.6028 \qquad \text{"Non-breaking/Surging"}$$



Overland bore propagation:

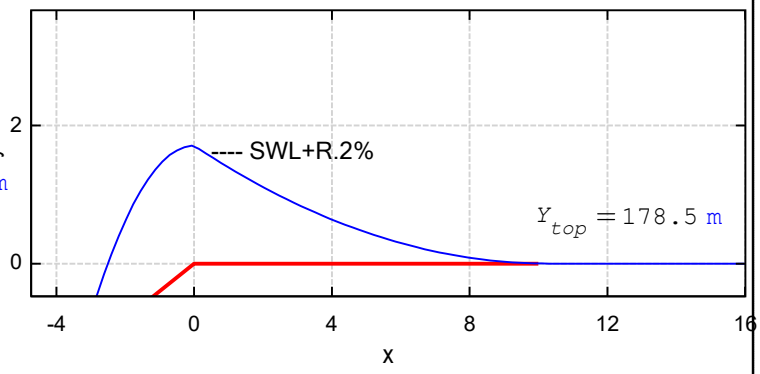
Cox and Machemehl (1986): $Y_{top} := 178.5\ \text{m}$

$F := Y_{top} - SWL = 0.6\ \text{m}$

$R_{2\%} - F = 1.7087\ \text{m}$

$\psi = 1.7$

$$L_{bore} := \frac{\psi \cdot T_m \cdot \sqrt{g}}{5} \cdot \left(R_{2\%} - F\right)^{\frac{1}{2}} = 10.2993\ \text{m}$$



Flood hazard limit:

$$SWL + R_{2\%} = 180.2087\ \text{m}$$

References

Wave uprush:

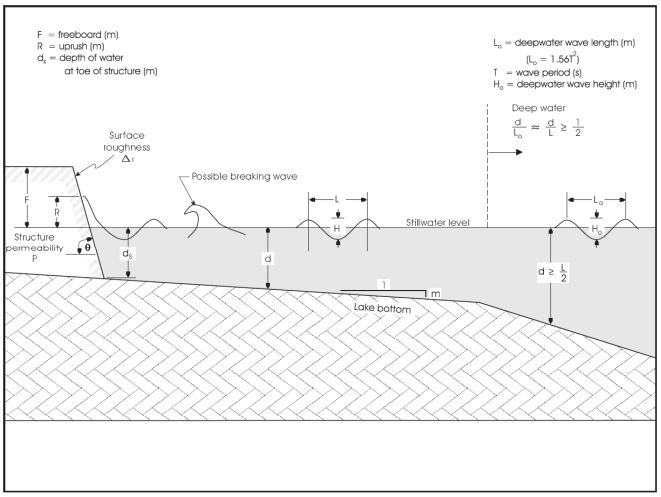
Atria Engineering (1997). *Wave Uprush and Overtopping: Methodologies and Applications - Great Lakes - St. Lawrence River System.* Prepared for the Ontario Ministry of Natural Resources (MNR)

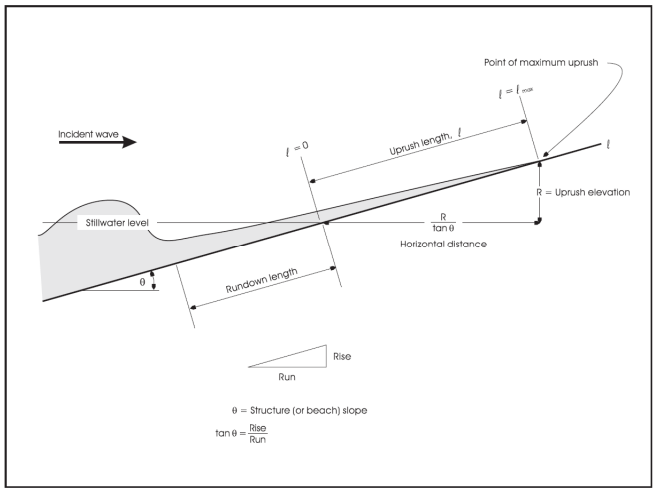
Goda (2010). *Random Seas and Design of Maritime Structures.* World Scientific

Revetmen design:

USACE (1995). *Design of Coastal Revetments, Seawalls, and Bulkheads*. US Army Corps of Engineers. (EM-1110-2-1614)

Wave Uprush Calculations:





Deep water wave conditions:

$H_{0S} := 0.23\ m$ $\quad T_p := \frac{7.4\ s}{0.9}$ $\quad d_{Deep} := 1.2\ m$ (Results from DELFT3D[W=20.1m/s@350°->20YR])

$SWL := 177.9\ m$ (100yr return period SWL - MNR)

$L_0 := \frac{g \cdot T_p^{\,2}}{2 \cdot \pi} = 105.5162\ m$ $\quad \frac{d_{Deep}}{L_0} = 0.0114$ "NO Deep Water" (Exact L.Deep = 27.8696 m)

Refracted conditions:

$Kr := 1.0$

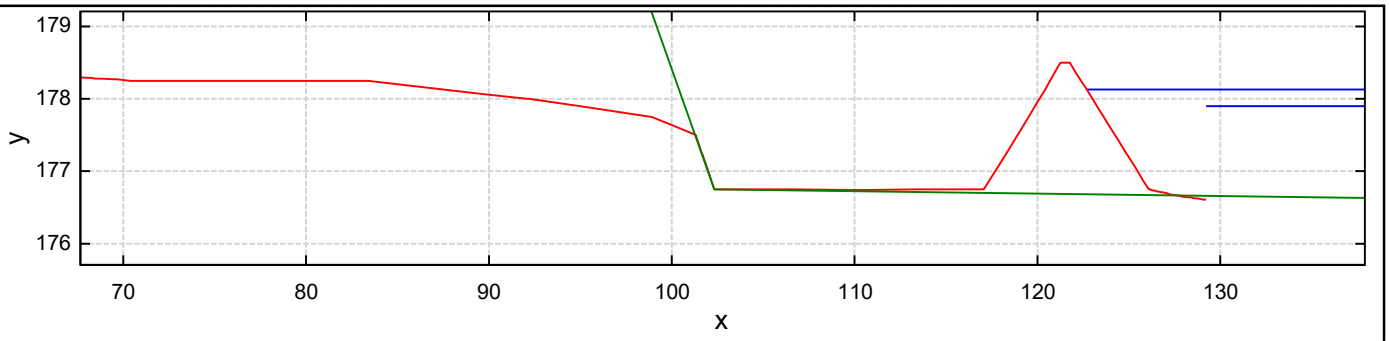
$H'_{0S} := Kr \cdot H_{0S} = 0.23\ m$

$H'_{0m} := 0.626 \cdot H'_{0S} = 0.144\ m$ $\quad T_m := 0.9 \cdot T_p = 7.4\ s$

$L_{0m} := \frac{g \cdot T_m^{\,2}}{2 \cdot \pi} = 85.4681\ m$

Slopes:

$m := \frac{1}{300}$ $\quad m_s := \frac{1}{1.4} = 0.7143$ $\quad Y_{toe} := 176.75\ m$



$X_{toe} = 102.33\ m$ $Y_{toe} = 176.75\ m$ $m_s = 0.7143$ $d_s := SWL - Y_{toe} = 1.15\ m$

→1 / 1.4

Wave shoaling considering wave breaking:

Deep water:

$H'_{0S} = 0.23\ m$ $T_p = 8.2222\ s$ $d_{Deep} = 1.2\ m$ $L_0 = 27.8696\ m$ $k_0 := \frac{2\cdot\pi}{L_0} = 0.2254\cdot\frac{1}{m}$

$$n_0 := \frac{1}{2}\cdot\left(1+\frac{2\cdot k_0\cdot d_{Deep}}{\sinh\left(2\cdot k_0\cdot d_{Deep}\right)}\right) = 0.9764$$

Toe:

$d_s = 1.15\ m$

$$L_s := \text{solve}\left(L\ m = \frac{g\cdot T_p^{\,2}}{2\cdot\pi}\cdot\tanh\left(\frac{2\cdot\pi\cdot d_s}{L\ m}\right), L, 1, 200\right) m = 27.2966\ m \qquad k_s := \frac{2\cdot\pi}{L_s} = 0.2302\cdot\frac{1}{m}$$

$$n_s := \frac{1}{2}\cdot\left(1+\frac{2\cdot k_s\cdot d_s}{\sinh\left(2\cdot k_s\cdot d_s\right)}\right) = 0.9774$$

Shoaling Coefficient:

$$Ks := \sqrt{\frac{n_0\cdot L_0}{n_s\cdot L_s}} = 1.0099$$

Depth-limited wave height - wave breaking (Weggel, 1972):

$a := 43.8\cdot\left(1-\exp\left(-19\cdot m\right)\right) = 2.688$

$$b := \frac{1.56}{\left(1+\exp\left(-19.5\cdot m\right)\right)} = 0.8053$$

$$H_b := \text{solve}\left(\frac{H_b\ m}{d_s} = b - a\cdot\frac{H_b\ m}{g\cdot T_p^{\,2}}, H_b, 0.5\cdot d_s\ m^{-1}, 1.6\cdot d_s\ m^{-1}\right) m = 0.9218\ m$$

$\frac{H_b}{g \cdot T_p^{\,2}} = 0.0014$ $\frac{H_b}{d_s} = 0.8016$ $m = 0.0033$

$H_{sS} := \min\left(\begin{bmatrix} H_b \\ Ks \cdot H'_{0S} \end{bmatrix}\right) = 0.2323\ \text{m}$

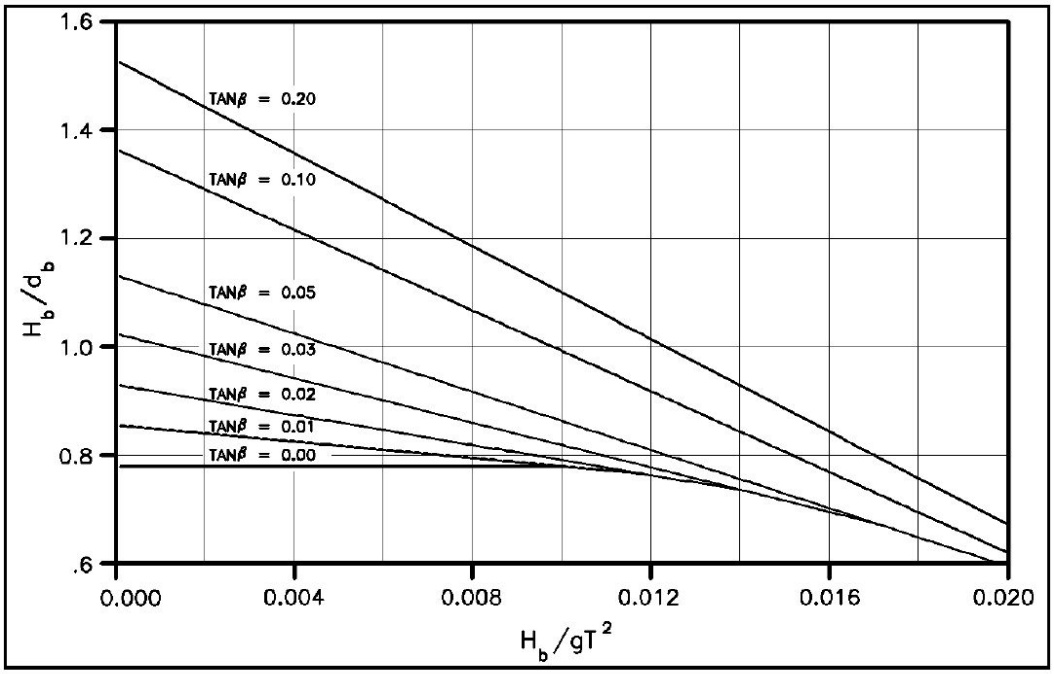


Figure II-4-2. Breaker depth index as a function of $H_b/(gT^2)$ (Weggel 1972)

Wave uprush:

Van Gent (2001):

$\xi_S := \frac{m_s}{\sqrt{\frac{H_{sS}}{L_0}}} = 7.8239$ $\gamma_\beta := 1.0$ $\gamma_f := 0.7$

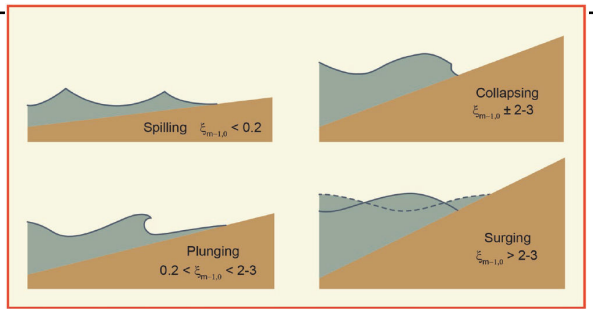
$R_{2\%} := \gamma_f \cdot \gamma_\beta \cdot H_{sS} \cdot \begin{pmatrix} \text{if } \xi_S \le 1.74 \\ \quad 1.35 \cdot \xi_S \\ \text{else} \\ \quad 4.7 - \frac{4.09}{\xi_S} \end{pmatrix} = 0.6792\ \text{m}$

Type of Wave Braking (EuroTop2018):

$\xi_S := \frac{m_s}{\sqrt{\frac{H_{sS}}{L_0}}} = 7.8239$

$$\xi_S := \frac{m_s}{\sqrt{\frac{2 \cdot \pi \cdot H_{sS}}{g \cdot T_m^{\,2}}}} = 13.7013$$

"Non-breaking/Surging"



Overland bore propagation:

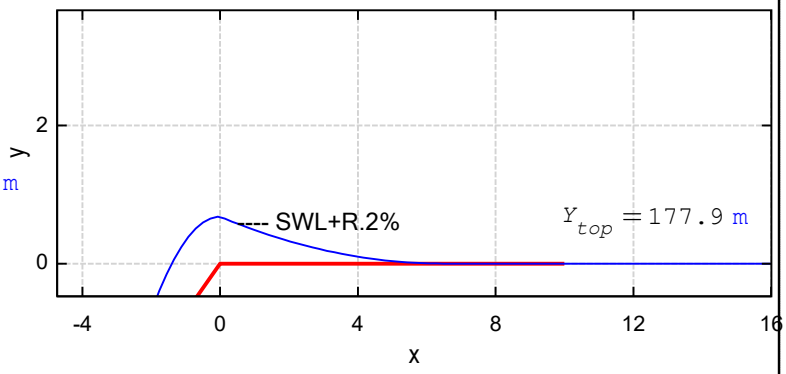
Cox and Machemehl (1986): $Y_{top} := 177.9\ m$

$F := Y_{top} - SWL = 0$

$R_{2\%} - F = 0.6792\ m$

$\psi = 1.7$

$$L_{bore} := \frac{\psi \cdot T_m \cdot \sqrt{g}}{5} \cdot \left(R_{2\%} - F\right)^{\frac{1}{2}} = 6.4935\ m$$



Flood hazard limit:

$SWL + R_{2\%} = 178.5792\ m$

References

Wave uprush:

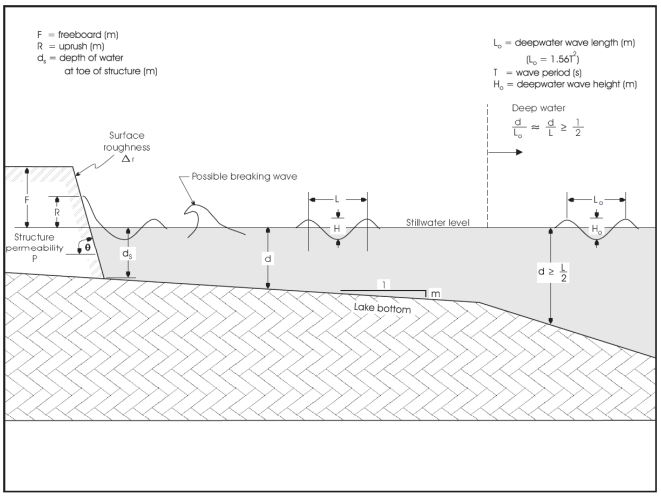
Atria Engineering (1997). *Wave Uprush and Overtopping: Methodologies and Applications - Great Lakes - St. Lawrence River System.* Prepared for the Ontario Ministry of Natural Resources (MNR)

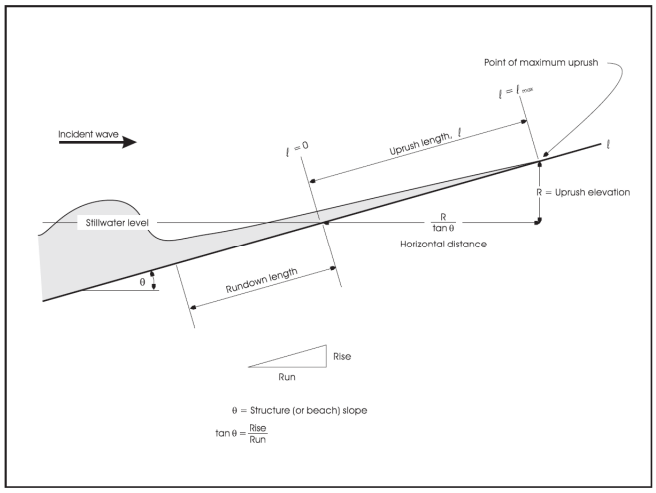
Goda (2010). *Random Seas and Design of Maritime Structures.* World Scientific

Revetmen design:

USACE (1995). *Design of Coastal Revetments, Seawalls, and Bulkheads*. US Army Corps of Engineers. (EM-1110-2-1614)

Wave Uprush Calculations:





Deep water wave conditions:

$H_{0S} := 2.05\ m$ $\quad T_p := \frac{7.4\ s}{0.9}$ $\quad d_{Deep} := 3.22\ m$ (Results from DELFT3D[W=20.1m/s@350°->20YR])

$SWL := 177.9\ m$ (100yr return period SWL - MNR)

$L_0 := \frac{g \cdot T_p^{\,2}}{2 \cdot \pi} = 105.5162\ m$ $\quad \frac{d_{Deep}}{L_0} = 0.0305$ "NO Deep Water" (Exact L.Deep = 44.7233 m)

Refracted conditions:

$Kr := 1.0$

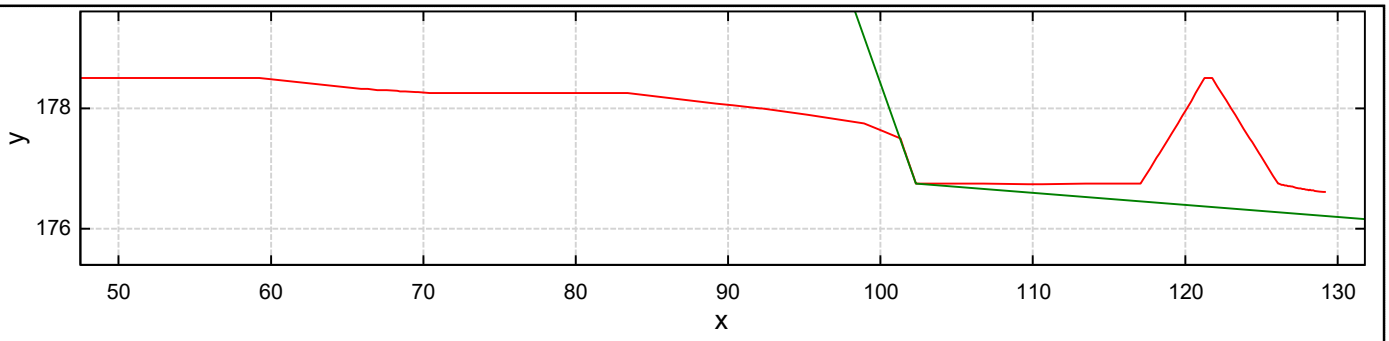
$H'_{0S} := Kr \cdot H_{0S} = 2.05\ m$

$H'_{0m} := 0.626 \cdot H'_{0S} = 1.2833\ m$ $\quad T_m := 0.9 \cdot T_p = 7.4\ s$

$L_{0m} := \frac{g \cdot T_m^{\,2}}{2 \cdot \pi} = 85.4681\ m$

Slopes:

$m := \frac{1}{50}$ $\quad m_s := \frac{1}{1.4} = 0.7143$ $\quad Y_{toe} := 176.75\ m$



$X_{toe} = 102.33\ m$ $Y_{toe} = 176.75\ m$ $m_s = 0.7143$ $d_s := SWL - Y_{toe} = 1.15\ m$

→1 / 1.4

Wave shoaling considering wave breaking:

Deep water:

$H'_{0S} = 2.05\ m$ $T_p = 8.2222\ s$ $d_{Deep} = 3.22\ m$ $L_0 = 44.7233\ m$ $k_0 := \frac{2\cdot\pi}{L_0} = 0.1405\cdot\frac{1}{m}$

$$n_0 := \frac{1}{2}\cdot\left(1+\frac{2\cdot k_0\cdot d_{Deep}}{\sinh\left(2\cdot k_0\cdot d_{Deep}\right)}\right) = 0.9378$$

Toe:

$d_s = 1.15\ m$

$$L_s := \text{solve}\left(L\ m = \frac{g\cdot T_p^{\,2}}{2\cdot\pi}\cdot\tanh\left(\frac{2\cdot\pi\cdot d_s}{L\ m}\right), L, 1, 200\right) m = 27.2966\ m \qquad k_s := \frac{2\cdot\pi}{L_s} = 0.2302\cdot\frac{1}{m}$$

$$n_s := \frac{1}{2}\cdot\left(1+\frac{2\cdot k_s\cdot d_s}{\sinh\left(2\cdot k_s\cdot d_s\right)}\right) = 0.9774$$

Shoaling Coefficient:

$$Ks := \sqrt{\frac{n_0\cdot L_0}{n_s\cdot L_s}} = 1.2538$$

Depth-limited wave height - wave breaking (Weggel, 1972):

$a := 43.8\cdot\left(1-\exp\left(-19\cdot m\right)\right) = 13.8469$

$$b := \frac{1.56}{\left(1+\exp\left(-19.5\cdot m\right)\right)} = 0.9302$$

$$H_b := \text{solve}\left(\frac{H_b\ m}{d_s} = b - a\cdot\frac{H_b\ m}{g\cdot T_p^{\,2}}, H_b, 0.5\cdot d_s\ m^{-1}, 1.6\cdot d_s\ m^{-1}\right) m = 1.0446\ m$$

$\frac{H_b}{g \cdot T_p^{\,2}} = 0.0016$ $\frac{H_b}{d_s} = 0.9084$ $m = 0.02$

$H_{sS} := \min\left(\begin{bmatrix} H_b \\ Ks \cdot H'_{0S} \end{bmatrix}\right) = 1.0446\ \text{m}$

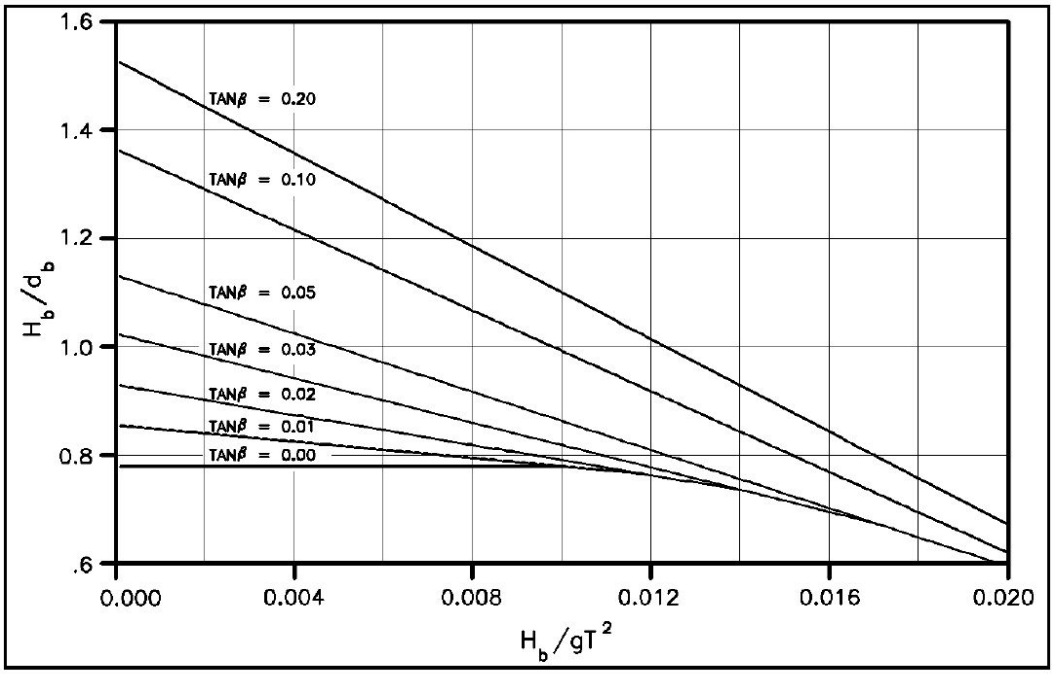


Figure II-4-2. Breaker depth index as a function of $H_b/(gT^2)$ (Weggel 1972)

Wave uprush:

Van Gent (2001):

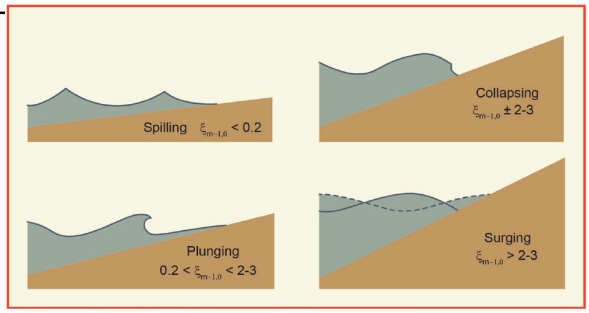
$\xi_S := \frac{m_s}{\sqrt{\frac{H_{sS}}{L_0}}} = 4.6736$ $\gamma_\beta := 1.0$ $\gamma_f := 1$

$R_{2\%} := \gamma_f \cdot \gamma_\beta \cdot H_{sS} \cdot \begin{pmatrix} \text{if } \xi_S \le 1.74 \\ 1.35 \cdot \xi_S \\ \text{else} \\ 4.7 - \frac{4.09}{\xi_S} \end{pmatrix} = 3.9956\ \text{m}$

Type of Wave Braking (EuroTop2018):

$\xi_S := \frac{m_s}{\sqrt{\frac{H_{sS}}{L_0}}} = 4.6736$

$$\xi_S := \frac{m_s}{\sqrt{\frac{2 \cdot \pi \cdot H_{sS}}{g \cdot T_m^{\,2}}}} = 6.4609$$ "Non-breaking/Surging"



Overland bore propagation:

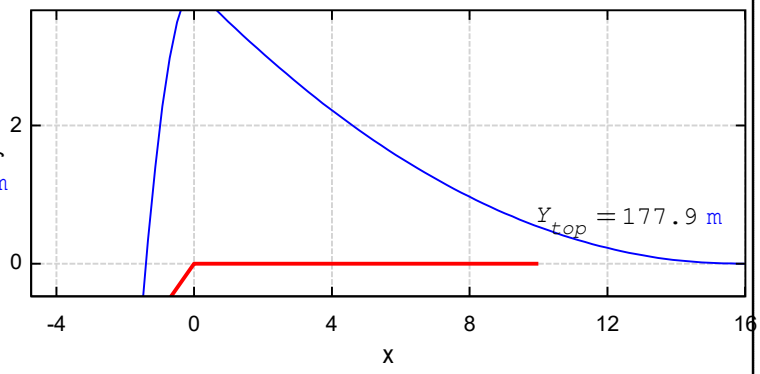
Cox and Machemehl (1986): $Y_{top} := 177.9\ m$

$F := Y_{top} - SWL = 0$

$R_{2\%} - F = 3.9956\ m$

$\psi = 1.7$

$$L_{bore} := \frac{\psi \cdot T_m \cdot \sqrt{g}}{5} \cdot \left(R_{2\%} - F\right)^{\frac{1}{2}} = 15.7494\ m$$



Flood hazard limit:

$$SWL + R_{2\%} = 181.8956\ m$$

References

Wave uprush:

Atria Engineering (1997). *Wave Uprush and Overtopping: Methodologies and Applications - Great Lakes - St. Lawrence River System.* Prepared for the Ontario Ministry of Natural Resources (MNR)

Goda (2010). *Random Seas and Design of Maritime Structures.* World Scientific

Revetmen design:

USACE (1995). *Design of Coastal Revetments, Seawalls, and Bulkheads*. US Army Corps of Engineers. (EM-1110-2-1614)

Drawing

GEORGIAN BAY

WATER LEVEL OCTOBER 18, 2024

TOP OF BANK

BOTTOM OF BANK

EX. BEACH

CULVERT 450mmø

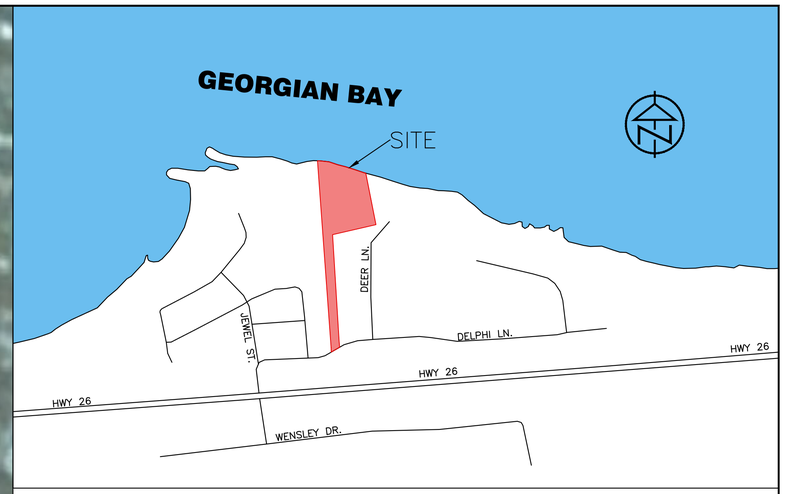
INV=176.69

INV=176.54

EX. BUILDING

DOOR SILL

SIB



KEY PLAN

LEGEND

15 m SETBACK FROM 100-YEAR HIGH-WATER LEVEL (177.9 m CONTOUR)	
WAVE UPRUSH LIMIT ATTENUATED BY BREAKWATER	
FLOOD HAZARD LIMIT	
6 m MAINTENANCE ACCESS ALLOWANCE	
100-YEAR HIGH-WATER LEVEL (177.9 m CONTOUR)	
WATER'S EDGE AT TIME OF TOPOGRAPHIC SURVEY	
PROPERTY LINE	
EXISTING BUILDING	
EXISTING RETAINING WALL	
BOULDER REVETMENT/BREAKWATER STRUCTURE	
EXISTING PATIO/DECK	
EDGE OF GRAVEL	
TOP/BOTTOM OF BANK	
CULVERT	
DITCH	
DECIDUOUS/CONIFEROUS TREE, SHRUB/BUSH	
BUSHLINE/TREELINE	
CONTOUR	179.00
SPOT ELEVATION	× 179.00
STANDARD IRON BAR	SIB
IRON BAR	IB

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BENCHMARKS

TBM1 – ELEVATION 179.11 m
CUT CROSS ON TRANSFORMER PAD

TBM2 – ELEVATION 181.53 m
CUT CROSS ON CONCRETE HEAD WALL

NOTES

TOPOGRAPHIC SURVEY COMPLETED BY VAN HARTEN SURVEYING INC., DATED NOVEMBER 14, 2024 (FIELD DATA COLLECTED OCTOBER 19, 2024). DRAWING TO BE READ IN CONJUNCTION WITH TATHAM REPORT DATED JANUARY 29, 2025.

ELEVATIONS ARE BASED ON GPS OBSERVATIONS TO PERMANENT REFERENCE STATIONS IN THE NAD83 (CSRS-2010) COORDINATE SYSTEM AND HAVE BEEN CORRECTED TO ORTHOMETRIC ELEVATIONS ON THE CGVD28 DATUM (1978 ADJUSTMENT) WITH GEOID MODEL HTv2.0, AS SUPPLIED BY NATURAL RESOURCES CANADA.

No.	REVISION DESCRIPTION	DATE
1.	DRAFT FOR CLIENT REVIEW	NOV 22/24
2.	ISSUED FOR GSCA REVIEW	JAN 29/25

ENGINEER STAMP

125, 127 & 129 DELPHI LANE
TOWN OF THE BLUE MOUNTAINS

TATHAM ENGINEERING

SHORELINE HAZARD LIMIT

DESIGN: MMR	FILE: 424468	DWG: SH.1
DRAWN: MMR	DATE: JAN. 2025	
CHECK: AMT	SCALE: 1:200	

Drawing Name: 424468-SHX01.dwg, Plotted: Jan 29, 2025