COASTAL DESIGN & ENGINEERING

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1. PROBLEM STATEMENT

Waves and Surge Actions are fluctuations of the water level. Due to storm surges that may last from a few hours to a few days, the need for a vertical-front and sloping rubble mound structure is necessary. The design of these coastal structures must withstand various hydrodynamic and hydrostatic forces. In addition, the structures must protect and maintain the various operations that take place on the leeway. On the coast of the Barrier Island, a facility must be constructed, two options must be considered against a combined 500-year wave and storm event. The options would involve the addition of a vertical seawall, and a sloping embankment. These structures must withstand storm load, wave overtopping rates, and must allow for the passage of vehicles on top as an access road.

1.1. Wave and Surge

Analyzing the storm surge near the site, along with its respective site layout, topography, and bathymetric data. The water depth for the design would be identified. As shown in figure 1, the ground elevation near the toe of the structure is 2 m MSL. Given that the maximum storm surge is 6.2 m MSL at point 39, it would be necessary to utilize the structure being partially sheltered by a dune, however for this exam, the protection can be removed.

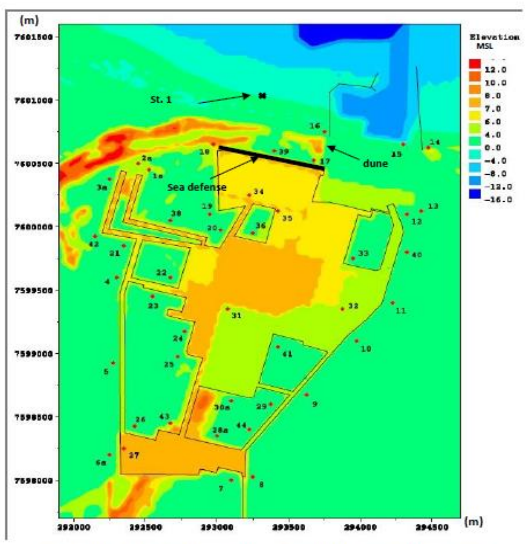

Figure 1. Facility siting, bathymetry/topography of area around structure

The elevation at Point 39 would be 2 m MSL. The Table generated below indicates the Storm Surge History at Point 39. The Storm Surges occur at Point 39 on a set of 6 hours. Identifying the various design waves and water levels for the sea defense structure would allow us to perform a wave transformation.

Table 1. Storm surge time history at Point 39

Time Step (hr)	Point 39
	S (m) MSL
1	4.3
2	5.7
3	6.1
4	6.2
5	5.6
6	4.8

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2. WAVE HEIGHT AND PERIOD ASSOCIATED WITH 500 YEAR STORM EVENT

In addition to the wave information from the buoy to the toe of the structure, The Surge effects based on zero surges or calm days must be assessed. Due to the surges ending at Point X on Figure 1, the superimposition of the surge to a no-surge depth must be performed. Various Combination of Wave and Surge levels would identify the most critical condition for loading, overtopping, rock size, etc.…

2.1. Wave Heights

The Application of the Peak Over Threshold method to a data set for Barrier Island for a period of 34 years, allowed the identification of the extreme events, assuming the storms would have an average duration of 3 days.

Referring to Table 2, the data was synthesized, and a threshold value of 3.25 m was selected which would allow the identification of the number of events, wave periods and heights relating to those events being determined.

Table 2 – Number of Extreme Events from 1980 – 2014, with a Peak Over Threshold of 3.25

Minimum Wave Height (m)	Maximum Wave Height (m)	Number of Events	Minimum Wave Height (m)	Maximum Wave Height (m)	Number of Events
0	0.250	0	5.001	5.250	10
0.251	0.500	0	5.251	5.500	4
0.501	0.750	0	5.501	5.750	6
0.751	1.000	0	5.751	6.000	1
1.001	1.250	0	6.001	6.250	1
1.251	1.500	0	6.251	6.500	3
1.501	1.750	0	6.501	6.750	3
1.751	2.000	0	6.751	7.000	0
2.001	2.250	0	7.001	7.250	0
2.251	2.500	0	7.251	7.500	0
2.501	2.750	0	7.501	7.750	0
2.751	3.000	0	7.751	8.000	1
3.001	3.250	0	8.001	8.250	0
3.251	3.500	4	8.251	8.500	0
3.501	3.750	58	8.501	8.750	0
3.751	4.000	39	8.751	9.000	0
4.001	4.250	34	9.001	9.250	0
4.251	4.500	19	9.251	9.500	1
4.501	4.750	17	9.501	9.750	2
4.751	5.000	15	Total Number of Events = 218		

Long Term Wave Analysis was conducted using Gumbel and Weibull Distributions. The identification of each respective P, and Q values were determined, allowing the creation of a Gumbel Distribution plot, and the creation of a Weibull Distribution Plot with variates of 0.8 and 1.3.

The Plots for the Gumbell and Weibull Distributions are located at Figures 2, Figures 3, and Figures 4.

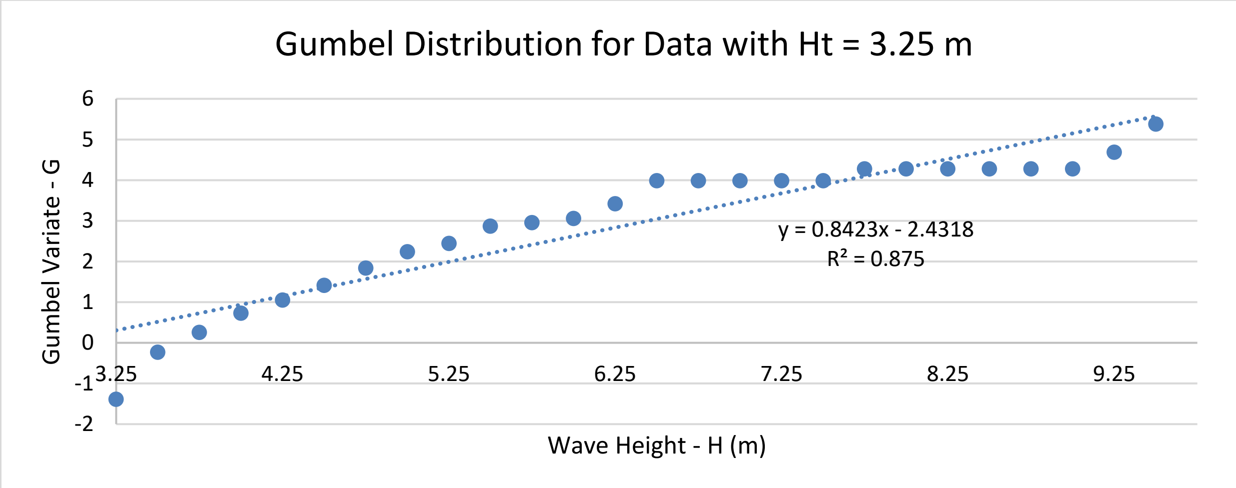

Figure 2: Gumbell Distribution with a Peak Over Threshold of 3.25 m

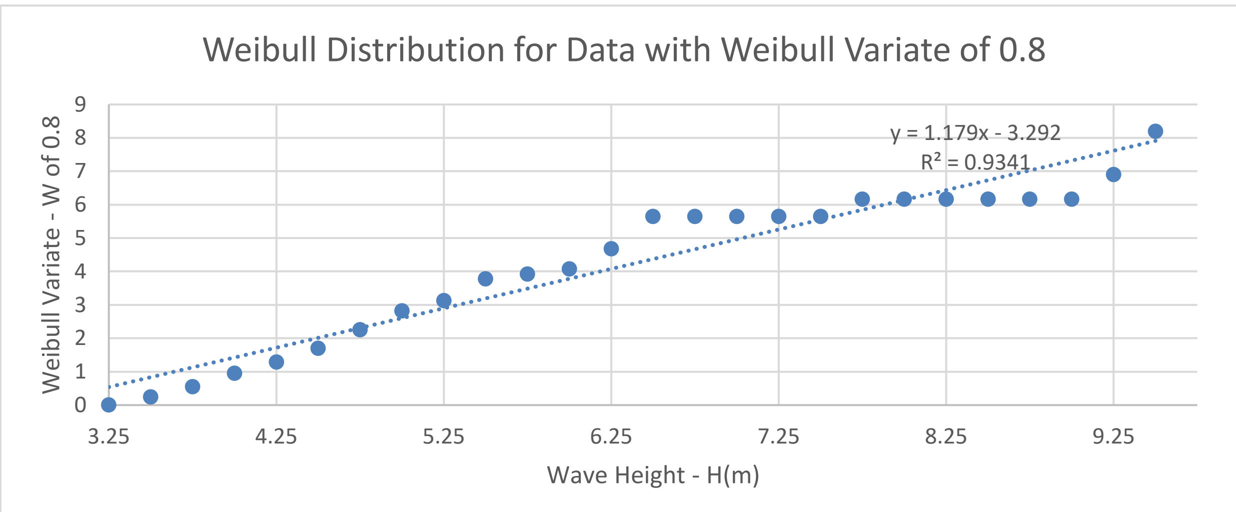

Figure 3: Weibull Distribution with a Peak Over Threshold of 3.25 m and a variate of 0.8.

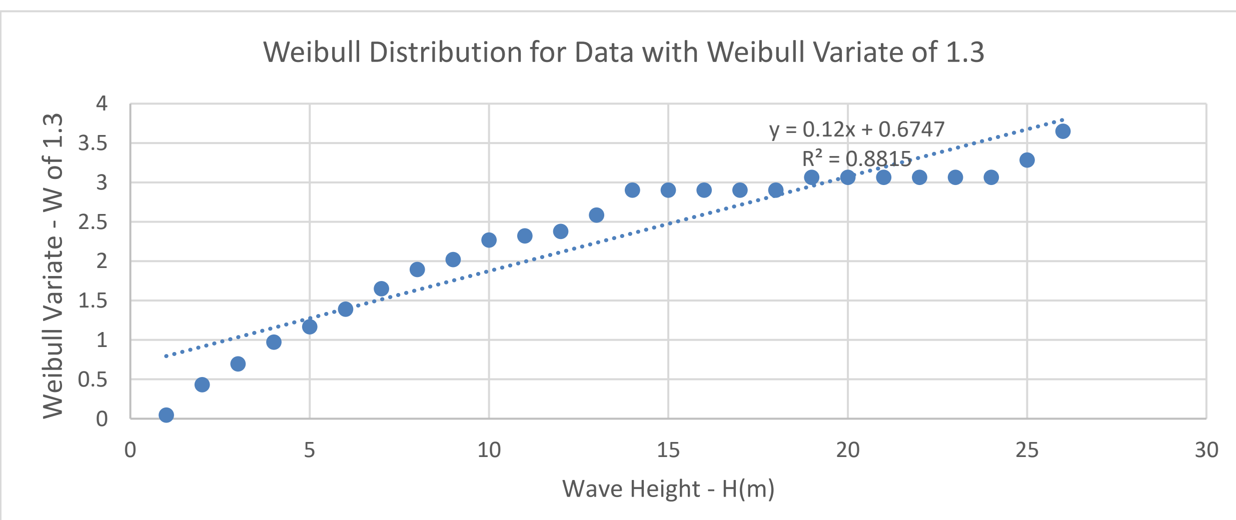

Figure 4: Weibull Distribution with a Peak Over Threshold of 3.25 m and a variate of 1.3.

Ultimately, after keen analysis with several varying thresholds (normally 2 or more threshold values would be plotted and the most linear would be chosen). The purpose of utilizing 2 or more threshold values is that it would allow us to see how changing the thresholds would impact our overall distribution. After analysis, the threshold values of 3.25 m, and an alpha variate of 0.8 was chosen to be the most linear.

The Return Periods Plots were Established according to the Gumbel and Weibull Distributions, the return periods of extreme events up to 500 Years were determined. Along with the Return periods relating to the probability of occurrence on a semilogarithmic plot. Several values necessary to plot the distributions are given below on Table 3.

Table 3 – Data Sets for the Return Periods for Weibull and Gumbel Distributions

	β	γ	λ
Gumbel	1.187	0.937	6.412
Weibull (0.8)	0.848	2.792	6.412

The Return Wave Heights for Gumbel Distributions were determined by solving the following:

$$H_{Tr} = \gamma + \beta \ln(\lambda \mathrm{Tr})^{1/\alpha}$$

The Return Wave Heights for Weibull Distributions were determined by solving the following:

$$H_{Tr} = \gamma + \beta \ln(\lambda \mathrm{Tr})^{1/\alpha}$$

The results for the Wave Height Predictions are given in Table 4:

Table 4 – Return Wave Heights for Gumbel and Weibull Distributions

Tr (Years)	1	2	5	10	20	50	100	200	300	500
Htr, Gumbel (m)	3.04	3.918	5.036	5.868	6.696	7.786	8.610	9.433	9.915	10.521
Htr, Weibull (0.8) (m)	4.632	5.526	6.805	7.832	8.903	10.377	11.533	12.712	13.428	14.334

The fitted curves for Weibull and Gumbel for Return Wave Heights with a sample representative ranging from 1-500 Years is given. This data will gauge the wave heights. Calculating the Tr of any extreme wave height selecting using the POT by using, $P = 1 - (1/\lambda Tr)$, allows us to calculate the Tr for a given H giving us actual data, we would pair this data with the Return Wave Heights for Gumbel and Weibull given in Table 4. The Plot of the Extreme Wave Height using POT to calculate Tr for a given H, and Return Wave Heights for Gumbel and Weibull are given in Figure 5 and 6.

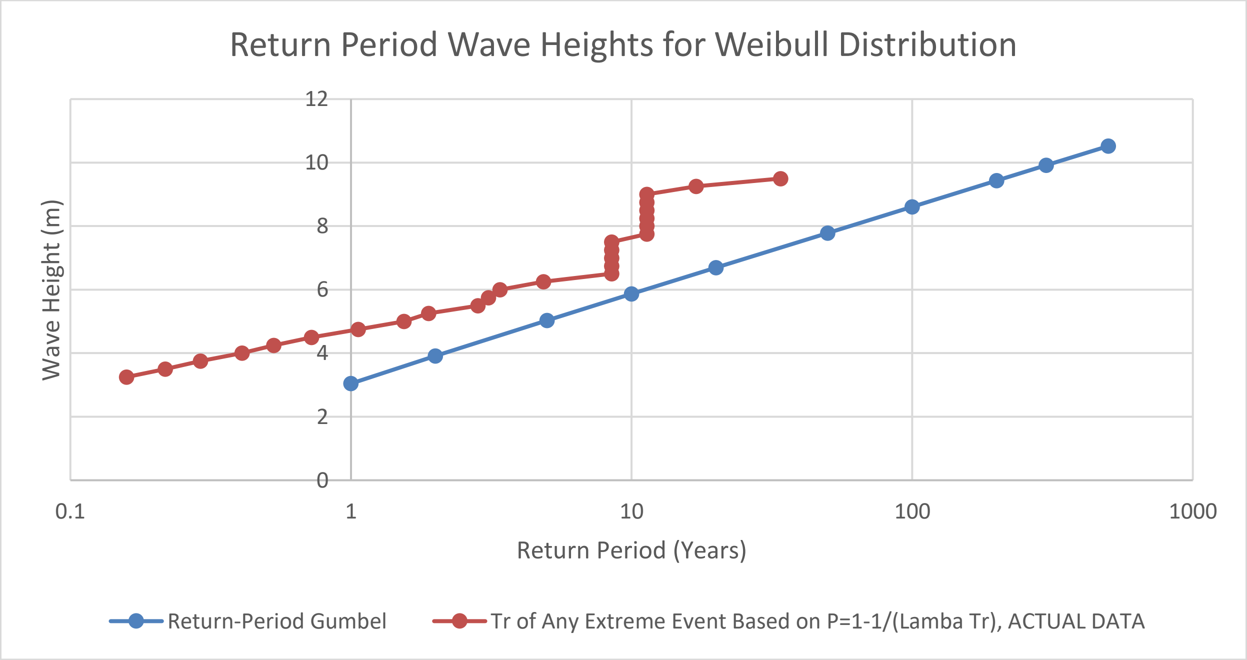

Figure 5: Return Period Wave Height for Weibull Distribution with an alpha of 0.8.

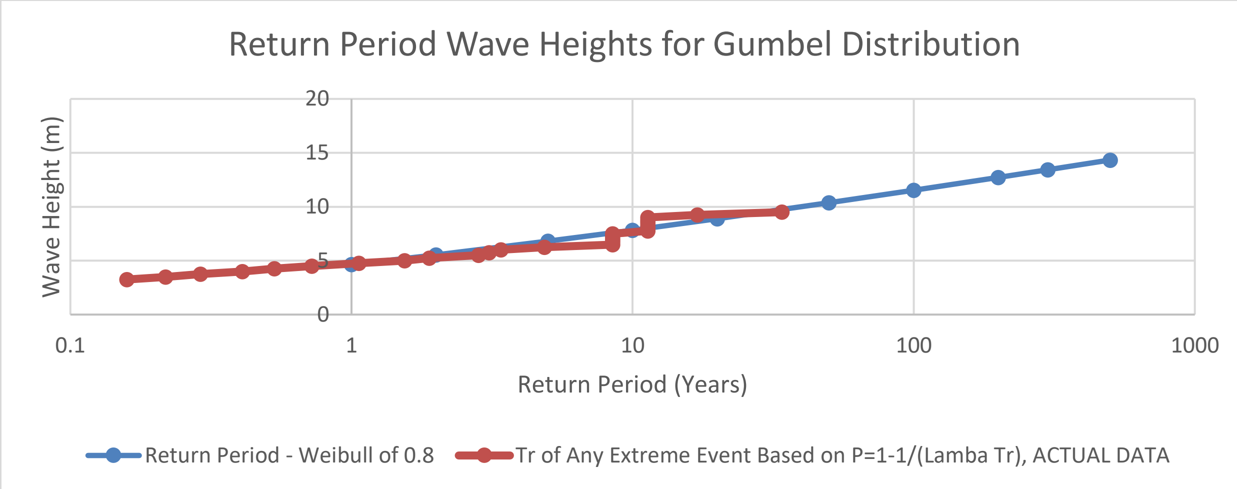

Figure 6: Return Period Wave Height for Gumbel Distribution.

Altogether, through analysis of the closeness of the Extreme Wave Height using POT over the Return Wave Heights, the Return Periods and Wave Heights associated with Weibull closely parallel the results. Hence, **the Return Wave Height for 500 Years of Weibull is chosen to be 14.334 m.**

2.2. Wave Period associated with a 500 Year Storm Event

The Design Peak Wave Period would be found using the Joint Distribution Method (Power Function). The Long-Term Distribution of Wave Period would be found by Plotting the Extreme Wave Heights from the years 1980 – 2014 with its respective periods at each point. Figure 6 identifies the Power Function key to identifying the Design Peak Wave Period.

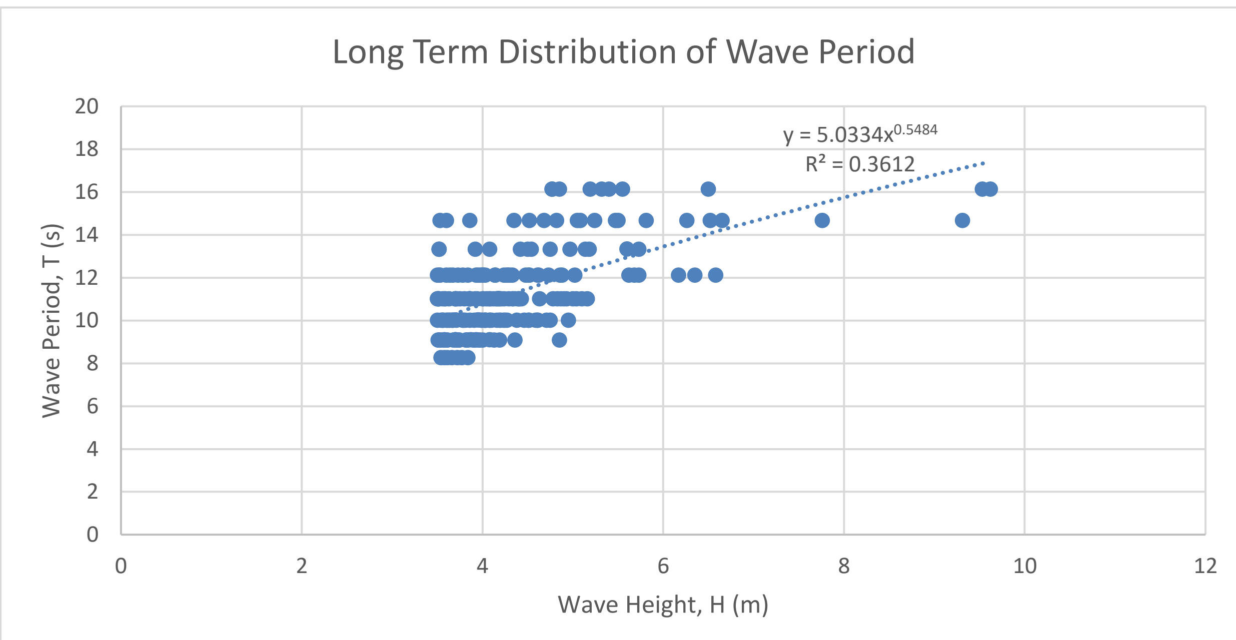

Figure 7: Long Term Distribution of Wave Period

Solving the equation: $Y = 5.0334x^{0.5484}$ generated by doing Long Term Distribution of Wave Period, the 500 Year Wave Height of 14.334 m would be plugged in as x. This would generate a **500 Year Wave Period of 21.677 s.**

3. DESIGN WAVE AND WATER LEVEL FOR SEA DEFENSE STRUCTURE

In order to identify the design wave and water level for a sea defense structure, interpolation of the distance and bottom elevation must be done from Station 0 to Station 9 (Buoy to the Toe of the Structure) in order to generate more data points. The interpolation between each station was done with small incremental Δx values of 100 m, we would linearly interpolate the Bottom Elevation along with each respective Distance values. As listed in Table 1, the surge may be removed to zero at Station 5, we would begin a process by linearly decreasing the Surge Values given in Table 1 to 0 by Station 5.

3.1. The Process of Superposing the Wave and Surge Levels

The Surge Levels in Table 1 do not take into consideration the ground surface elevation near the toe of the structure which is 2 m MSL. Therefore, each Surge Level in Table 1 would be reduced by 2 m. The Storm Surge at Hour 1 at Point 39 would be reduced to 2.3 m, the storm surge at hour 2 at Point 39 would be reduced to 3.7 m, the storm surge at hour 3 at Point 39 would be reduced to 4.1 m, the storm surge at hours 4,5, and 6 would be reduced to 4.2 m, 3.6 m, and 2.8 m respectively. The results of the linearly reduced Storm Surges to Station 5 would be added to the Bottom Elevations from Stations 1 to 9. This would allow us to begin Wave Transformation, where we would generate and identify the design wave and water levels. Each Storm Surge that was reduced in Hours 1 to 6 and added to the Bottom Elevations would undergo wave transformation. The Plots of the Wave Height vs. Distance from Sea Defense for various Surges are given in Figures 7 to 12. (For Detailed Values and the process of undergoing wave transformation for each Surge please see attached Excel File)

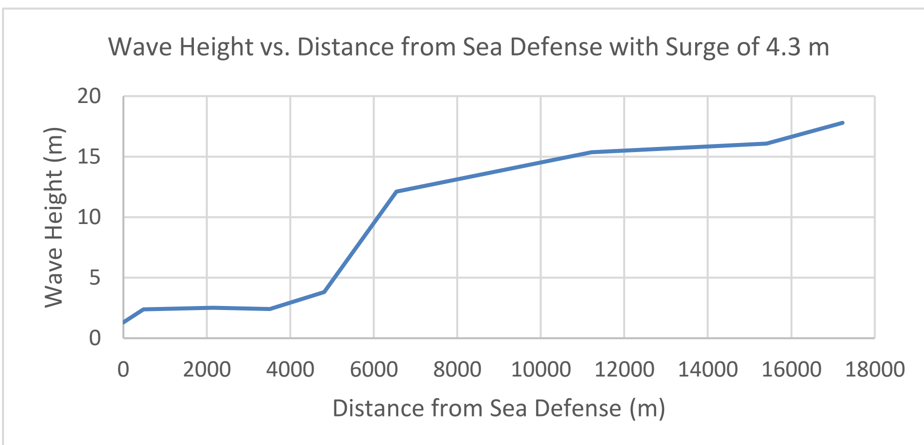

Figure 8: Wave Height vs. Distance from Sea Defense with Surge of 4.3 m (Hour 1)

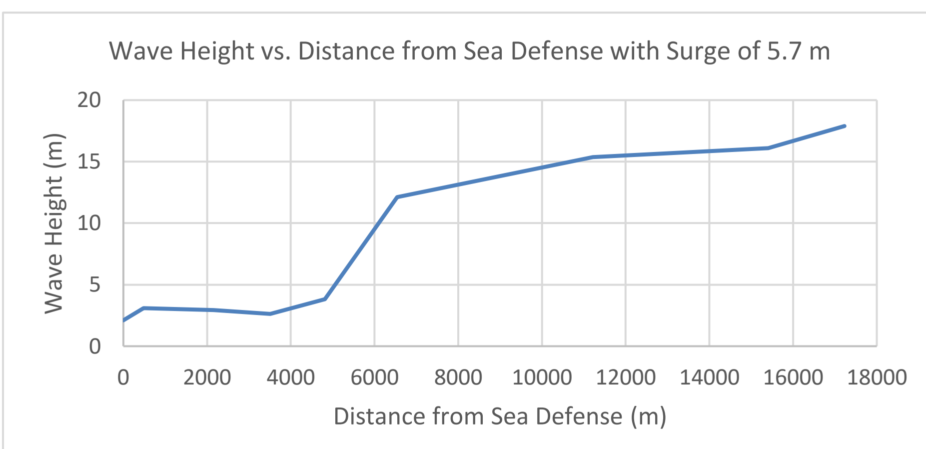

Figure 9: Wave Height vs. Distance from Sea Defense with Surge of 5.7 m (Hour 2)

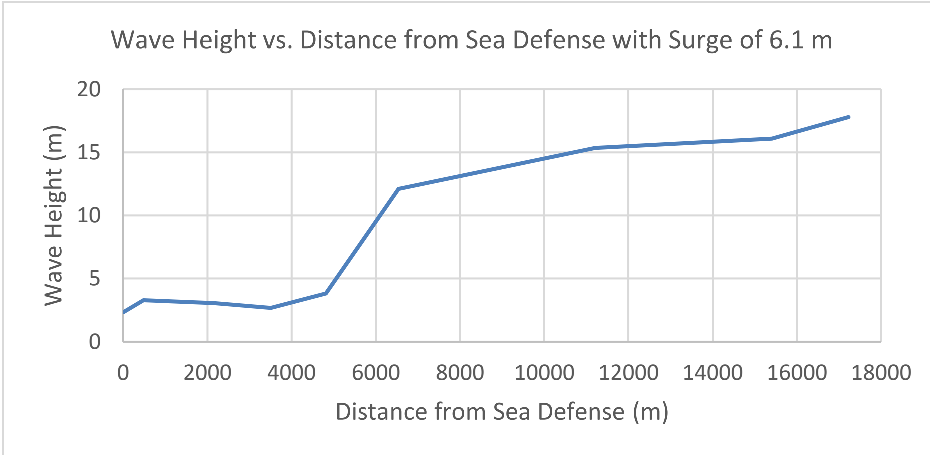

Figure 10: Wave Height vs. Distance from Sea Defense with Surge of 6.1 m (Hour 3)

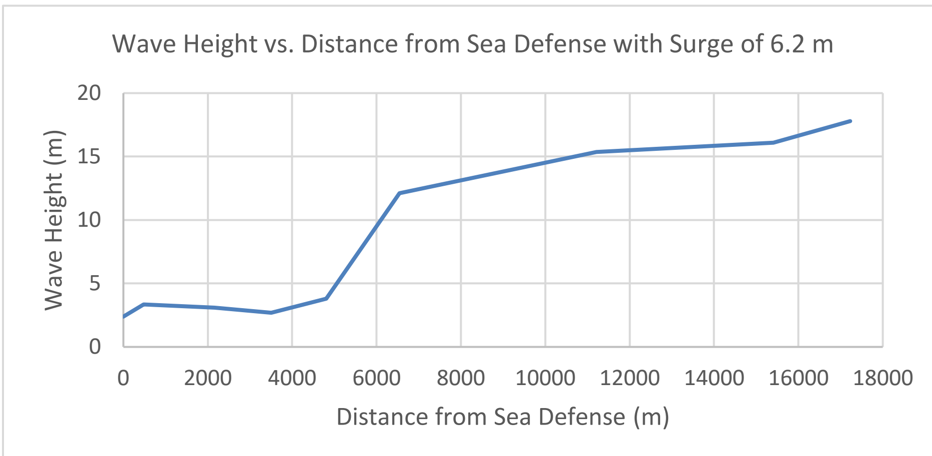

Figure 11: Wave Height vs. Distance from Sea Defense with Surge of 6.2 m (Hour 4)

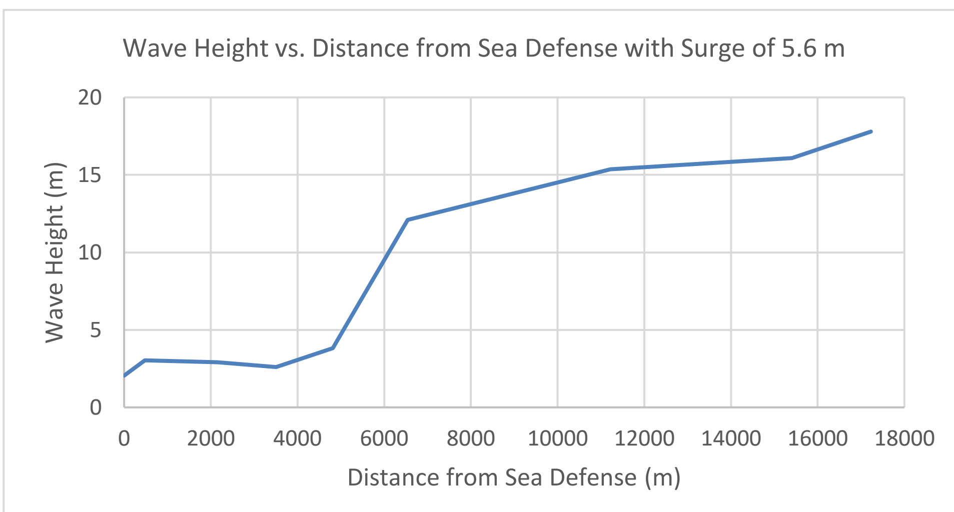

Figure 12: Wave Height vs. Distance from Sea Defense with Surge of 5.6 m (Hour 5)

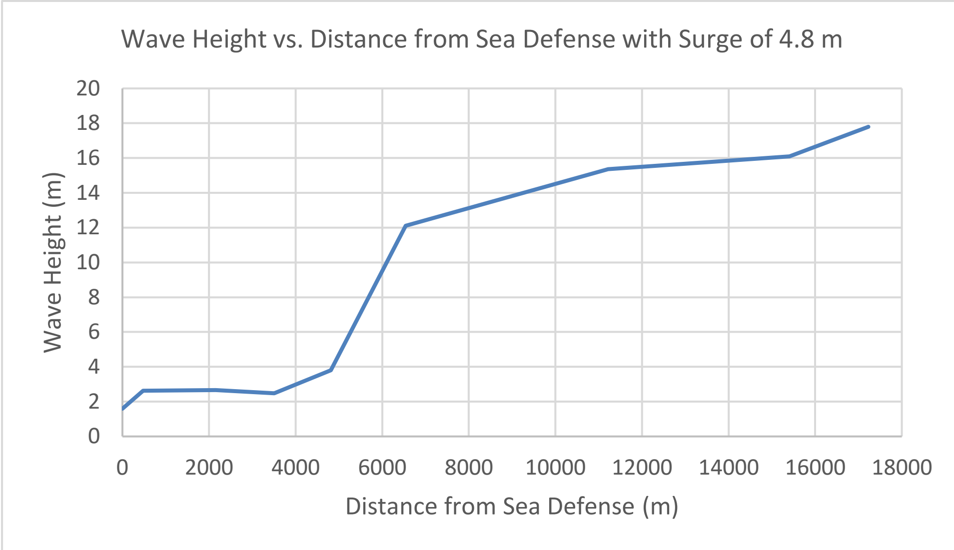

Figure 13: Wave Height vs. Distance fr*om* Sea Defense with Surge of 4.8 m (Hour 6)

After analyzing the results from the Wave Heights vs. Distance from Sea Defense for the 6 different surges. **The most critical combination would occur at the 4th Hour Surge of 6.2 m. The wave height at Station 1 or the shoreline would be 2.388 m.** The identification of this result would enable the selection of various parameters to design a seawall as a shore protection measure (Vertical Breakwater). The process of designing a structure would be iterative, and repetitive until a selected Height of the wall would be chosen that would have a crest that can serve as an access road, and would be within the allowable wave overtopping rate, $q_{allowable} = 10\frac{l}{m\,s}$.

4. DESIGN OF A VERTICAL BREAKWATER STRUCTURE

The overall parameters to design a seawall or vertical breakwater structure as a shore protection measure would involve determining the height of the wall, the water depth in front of the breakwater H, the depth above the armor layer of rubble mound foundation, d. It would also involve the determination of the Crest Elevation. The overall design of the foundation would also require the calculation of the armor unit size, and the scour protection toe block. Although the design values for the height, depth, crest elevation, etc. are given these values can be different depending on the situation or parameters revolving the site, including but not limited to cost, material, accessibility. The values for the Design of the Vertical Breakwater are given below in Table 5. Calculations for the design will be followed. (For a definition of each term please see Appendix #)

Table 5 – Water Crest and Depth Elevation

H (m)	4.6	B (m)	10
d (m)	2.5	Berm (m)	3
H' (m)	3.5	f (Rubble Mound Foundation)	0.6
H_C (m)	6.0	ρ ($^{kg}/_{m^3}$)	1030
Ho (m)	2.388	ρ_{armor} ($^{kg}/_{m^3}$)	2650
T_p (s)	1.500	Ma (kg/m)	204,000
m	0.01		

These values were made pertaining to Figure 13 provided below, the Height of the Crest Hc, the water depth in front of the breakwater H, the depth above the armor layer d, the distance from the design water level to the bottom of the upright section h'. The width and the berm size, B and Berm are given. The relative location of each is provided.

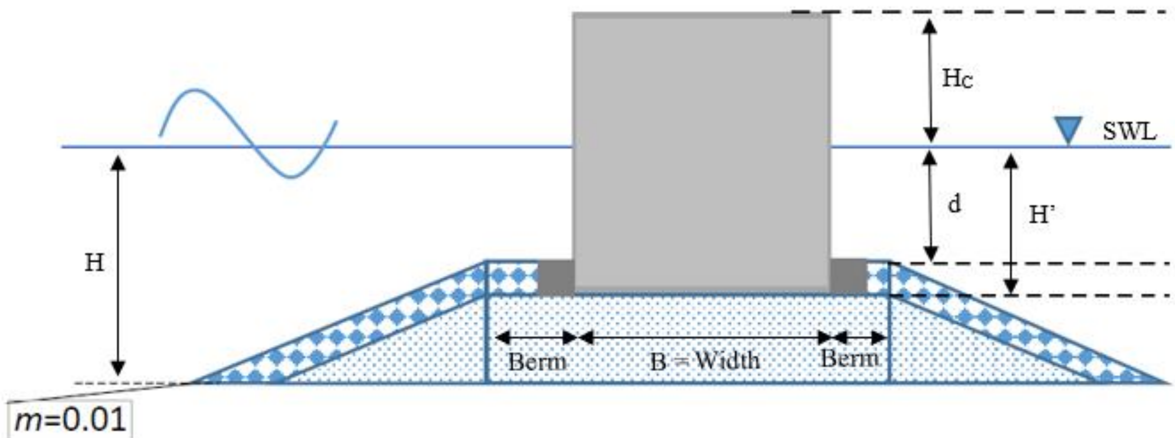

Figure 14: General Design Layout of the Vertical Breakwater Structure

4.1. Wave Induced Pressure Components, Forces, and Moments

Identify Wavelength, Lo

$L_o = 1.56\ x\ T_P^2$

$L_o = 1.56\ x\ (21.667\ s)^2$

$L_o = 732.36\ m$

Find Hs at H = 4.6 m

$L = L_o \tanh\left(\frac{2\pi H}{L}\right)$

$L = (732.36\ m) \tanh\left(\frac{2\pi(4.6\ m)}{L}\right)$

Through an iterative process
$L = 144.53\ m$

<u>Finding n</u>

$$n = 0.5(1 + (\frac{\frac{4\pi H}{L}}{sinh\left(\frac{4\pi H}{L}\right)})$$

$$n = 0.5(1 + (\frac{\frac{4\pi(4.6\ m)}{144.53\ m}}{sinh\left(\frac{4\pi(4.6\ m)}{144.53\ m}\right)})$$

$n = 0.987$

<u>Finding Ks</u>

$$Ks = \frac{1}{\sqrt{2ntanh(\frac{2\pi H}{L})}}$$

$$Ks = \frac{1}{\sqrt{2(0.987)tanh(\frac{2\pi(4.6\ m)}{144.53\ m})}}$$

$Ks = 1.602$

We need to identify these values in order to find the Significant Wave Height, Hs, in addition to the Maximum Wave Height, Hmax and the Height of the Breaking Water, Hb.

Check the Ratio of H/Lo
$^{H}/_{Lo} = 0.0063 < 0.2$

Therefore, the Significant Wave Height formula can be given as follows:

$$H_s = \min\ (\beta_0 H_0 + \beta_1 H, \beta_{max} H_0, K_s H_0)$$

<u>We need to calculate β_0, β_1 and β_{max}</u>

$$\beta_o = 0.028(\frac{H_o}{L_o})^{0.38}\exp(20tan^{1.5}\theta)$$

$$\beta_o = 0.028\left(\frac{2.388\ m}{732.36\ m}\right)^{0.38}\exp(20(0.01)^{1.5})$$

$\beta_o = 0.252$

$\beta_1 = 0.52\exp(4.2tan\theta)$
$\beta_1 = 0.52\exp(4.2(0.01))$
$\beta_1 = 0.542$

$$\beta_{max} = \max\ (0.92, 0.32\left(\frac{H_o}{L_o}\right)^{-0.29}\exp(2.4tan\theta)$$

$$\beta_{max} = \max\ (0.92, 0.32\left(\frac{2.388\ m}{732..355\ m}\right)^{-0.29}\exp(2.4(0.01))$$

$\beta_{max} = 1.724$

$H_s = \min\ (3.095, 4.118, 3.826)$
H_s = 3.095 m

<u>Calculate the Breaking Depth</u>

$H_b = H + m(5H_s)$
$H_b = 4.6\ m + 0.01(5(3.095\ m))$
$\boldsymbol{H_b = 4.755\ m}$

<u>Check the Ratio of Hb/Lo</u>
$^{Hb}/_{Lo} = 0.0065 < 0.2$

To find Hmax, we need to calculate a series of equations
$H_{max} = \min\ (\beta_0^* H_0 + \beta_1^* H_b, \beta_{max}^* H_0, 1.8K_s H_0)$

<u>We need to recalculate L, n, and Ks using our Hb values</u>

$L = L_o \tanh\ (\frac{2\pi H}{L})$
$L = (732.355\ m)\tanh\ (\frac{2\pi(4.754\ m)}{L})$
Through an iterative process

$L = 146.91\ m$

$$n = 0.5(1 + (\frac{\frac{4\pi H_b}{L}}{sinh\left(\frac{4\pi H_b}{L}\right)})$$
$$n = 0.5(1 + (\frac{\frac{4\pi(4.755\ m)}{146.91\ m}}{sinh\left(\frac{4\pi(4.755\ m)}{146.91\ m}\right)})$$

$n = 0.986$

$$Ks = \frac{1}{\sqrt{2ntanh(\frac{2\pi H_b}{L})}}$$
$$Ks = \frac{1}{\sqrt{2(0.986)tanh(\frac{2\pi(4.755\ m)}{146.91\ m})}}$$

$Ks = 1.589$

Find the values of $\boldsymbol{\beta_0^*, \beta_1^*, and\ \beta_{max}^*\ to\ find\ Hmax}$

$$\beta_0^* = 0.52\left(\frac{H_o}{L_o}\right)^{-0.38} \exp(20tan^{1.5}\theta)$$
$$\beta_0^* = 0.52\left(\frac{2.388\ m}{732.355\ m}\right)^{-0.38} \exp(20(0.01)^{1.5})$$
$\beta_0^* = 0.467$

$\beta_1^* = 0.63(exp(3.8tan\theta))$
$\beta_1^* = 0.63(exp(3.8(0.01))$
$\beta_1^* = 0.630$

$$\beta_{max}^* = \max\ (1.65, 0.53\left(\frac{H_o}{L_o}\right)^{-0.29} \exp(2.4tan\theta)$$

$$\beta^*_{max} = \max\left(1.65, 0.53\left(\frac{2.388\ m}{732.355\ m}\right)^{-0.29} \exp(2.4(0.01))\right)$$
$$\beta^*_{max} = 2.856$$

$$H_{max} = \min(\beta^*_0 H_0 + \beta^*_1 H_b, \beta^*_{max} H_0, 1.8 K_s H_0)$$
$$\boldsymbol{H_{max} = 4.112\ \mathrm{m}\ at\ H_b = 4.755\ m}$$

<u>Identify the Wave Pressure Coefficients</u>

$$\alpha_1 = 0.6 + 0.5\left(\frac{4\pi H/L}{\sinh(4\pi H/L)}\right)^2$$

$$\alpha_1 = 0.6 + 0.5\left(\frac{4\pi(4.6\ m)/(146.91\ m)}{\sinh\left(\frac{4\pi(4.6\ m)}{146.91\ m}\right)}\right)^2$$

$$\alpha_1 = 1.075$$

$$\alpha_2 = \min\left(\left(\frac{H_b - d}{3H_b}\right)\left(\frac{H_{max}}{d}\right)^2, \frac{2d}{H_{max}}\right)$$

$$\alpha_2 = \min\left(\left(\frac{4.755\ m - 2.5\ m}{3(4.755\ m)}\right)\left(\frac{4.112\ m}{2.5\ m}\right)^2, \frac{2(2.5\ m)}{4.112\ m}\right)$$

$$\alpha_2 = 0.428$$

$$\alpha_3 = 1 - \frac{H'}{H}\left(1 - \frac{1}{\cosh\left(\frac{2\pi H}{L}\right)}\right)$$

$$\alpha_3 = 1 - \frac{3.5\ m}{4.6\ m}\left(1 - \frac{1}{\cosh\left(\frac{2\pi(4.6\ m)}{144.53\ m}\right)}\right)$$

$$\alpha_3 = 0.985$$

We assume that the wave is normal to the structure

β=0^o

$$\xi^* = Run - up\ Height = 1.5 H_{max}$$

$$\boldsymbol{\xi^* = 6.17\ m}$$

<u>Identify the Pressure Components</u>

$$P_1 = 0.5(1 + cos\beta)(\alpha_1 + \alpha_2 cos^2\beta)(\rho g H_{max})$$
$$P_1 = 0.5(1 + cos0)(1.075 + 0.428 cos^2 0)(1030\ {}^{kg}/_{m^3})(9.81\ \frac{m}{s^2})(4.112\ m)$$
$$\boldsymbol{P_1 = 62.450\ \frac{kN}{m^2}}$$

$$P_2 = \frac{P_1}{\cosh\left(\frac{2\pi H}{L}\right)}$$

$$P_2 = \frac{62.450\ \frac{kN}{m^2}}{\cosh\left(\frac{2\pi(4.6\ m)}{144.531\ m}\right)}$$

$$\boldsymbol{P_2 = 61.221\ \frac{kN}{m^2}}$$

$$P_3 = \alpha_3 P_1$$

$P_3 = (0.985)\left(62.450\ \frac{kN}{m^2}\right)$

$\boldsymbol{P_3 = 61.515\ \frac{kN}{m^2}}$

$P_4 = P_1(1 - \frac{H_c}{\xi^*})$ Given that $\xi^* > H_c$

$P_4 = (\mathbf{62.450}\ \frac{\boldsymbol{kN}}{\boldsymbol{m^2}})(1 - \frac{6\ m}{6.17\ m})$

$\boldsymbol{P_4 = 1.711\ \frac{kN}{m^2}}$

$P_U = 0.5(1 + cos\beta)(\alpha_1\alpha_3)(\rho g H_{max})$

$P_U = 0.5(1 + cos0)((1.075)(0.985))(1030\ {kg}/{m^3})(9.81\ \frac{m}{s^2})(4.112\ m)$

$\boldsymbol{P_U = 44.004\ {kN}/{m^2}}$

$\boldsymbol{H_c^* = \min(\xi^*, Hc) = 6\ m}$

$P = \frac{1}{2}(P_1 + P_3)H' + \frac{1}{2}(P_1 + P_4)H_c^*$

$P = \frac{1}{2}\left(62.450\frac{kN}{m^2} + 61.515\frac{kN}{m^2}\right)(3.5\ m) + \frac{1}{2}\left(62.450\frac{kN}{m^2} + 1.711\frac{kN}{m^2}\right)(6\ m)$

$\boldsymbol{P = 409.42\ kN/m}$

$U = \frac{1}{2}(P_U B)$

$U = \frac{1}{2}\left(44.004\ \frac{kN}{m^2})(10m)\right)$

$\boldsymbol{U = 220.018\ kN/m}$

The Pressure Distribution Diagram is given below at Figure 14

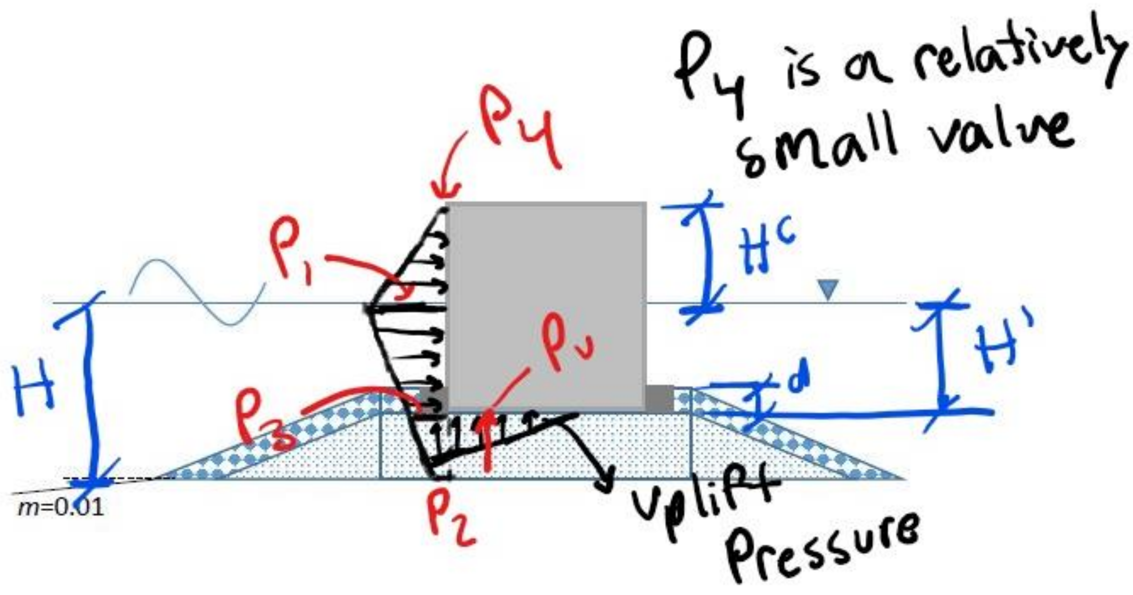

Figure 15: Pressure Distribution Diagram

Moment of Wave Pressure

$M_P = \frac{1}{6}(2P_1 + P_3)H'^2 + \frac{1}{2}(P_1 + P_4)H'H_c + \frac{1}{6}(P_1 + 2P_4){H_c}^2$

$M_P = \frac{1}{6}\left(2(62.450\frac{kN}{m^2}) + 61.515\frac{kN}{m^2}\right)(3.5\ m)^2 + \frac{1}{2}\left(62.450\frac{kN}{m^2} + 1.711\frac{kN}{m^2}\right)(3.5\ m)(6\ m) + \frac{1}{6}\left(62.450\frac{kN}{m^2} + 2(1.711\frac{kN}{m^2})\right)(6\ m)^2$

$\boldsymbol{M_P = 1449.53\ kN\ \frac{m}{m}}$

$M_U = \frac{1}{3}(P_U B^2)$

$M_U = \frac{1}{3}\left((44.004\ \frac{kN}{m^2})(10m\ )^2\right)$

$\boldsymbol{M_U = 1466.78\ kN\frac{m}{m}}$

Check for Weather Impulsive Wave Pressure

Waves normally break on the Upright Section, because we have wave induced pressure, we must conduct a check to ensure that impulsive wave pressure does not exist. The procedure to check for weather impulsive wave pressure is highlighted below:

$\delta_{11} = 0.93(\frac{Berm}{L} - 0.12 +\ 0.36(0.4 - \frac{d}{H})$

$\delta_{11} = 0.93(\frac{3\ m}{144.531\ m} - 0.12 +\ 0.36(0.4 - \frac{2.5\ m}{4.6\ m})$

$\delta_{11} = -0.144$

δ_{11}< 0, therefore: $\delta_1 = 20\delta_{11}$

$\delta_1 = 20(-0.144)$

$\delta_1 = -2.879$

$\delta_{22} = -0.36(\frac{Berm}{L} - 0.12 +\ 0.93(0.4 - \frac{d}{H})$

$\delta_{22} = -0.36(\frac{3\ m}{144.531\ m} - 0.12 +\ 0.93(0.4 - \frac{2.5\ m}{4.6\ m})$

$\delta_{22} - 0.041$

δ_{22}< 0, therefore: $\delta_2 = 4.9\delta_{11}$

$\delta_2 = 4.9(-0.041)$

$\delta_2 = -0.202$

Since $\delta_2 < 0, \alpha_{IB} = \frac{cos\delta_2}{cosh\delta_1}$

$\alpha_{IB} = \frac{\cos(-0.202)}{\cosh(-2.879)}$

$\alpha_{IB} = 0.109$

$\alpha_{IH} = \min\ (\frac{H_{max}}{d}, 2.0)$

$\alpha_{IH} = 1.645$

$\alpha_I = (\alpha_{IH})(\alpha_{IB})$

$\boldsymbol{\alpha_I = 0.181 < \alpha_2 = 0.428}$**, therefore there is no impulsive wave pressure**

Note: If need be, we can repeat this check for the backside, since the Berm Length is the same on both sides, in this example there is no need for further calculations.

<u>Check for Structural Stability against Overturning and Sliding</u>

$\boldsymbol{M_A = 204{,}000 \frac{kg}{m}} = (\gamma_{Concrete})(A) = (2400\ \frac{kg}{m^3})((11.6\ m - 3.1\ m)(10\ m))$

$M_A = Mass\ per\ Unit\ Width\ of\ the\ Upright\ Section$

<u>Submerged Weight for the Front Berm, W</u>

$W = ((M_A - (\rho)(B)(Berm))g$

$W = ((204000\ \frac{kg}{m} - \left(1030\ \frac{kg}{m^3}\right)(10\ m)(3\ m))(9.81\ \frac{m}{s^2})$

$\boldsymbol{W = 1698.11\ kN/m}$

<u>Safety Factor Against Sliding</u>

The Sliding Safety Factor > 1.2 in order to be considered Stable

$\text{Safety Factor Against Sliding} = \frac{f(W-U)}{P}$

$\text{Safety Factor Against Sliding} = \frac{(0.6)(1698.11\ kN/m - 220.018\ kN/m)}{409.42\ kN/m}$

$\textbf{Safety Factor Against Sliding} = \boldsymbol{2.16 > 1.2}$

The Structure is stable against Sliding

<u>Safety Factor Against Overturning</u>

The Overturning Safety Factor > 1.2 in order to be considered Stable

$\text{Safety Factor Against Overturning} = \frac{0.5WB - M_U}{M_P}$

$\text{Safety Factor Against Overturning} = \frac{0.5(1698.11\ kN/m)(10\ m) - 1466.86 kN/m}{1449.53\ kN/m}$

$\textbf{Safety Factor Against Overturning} = \boldsymbol{4.846 > 1.2}$

The Structure is stable against Overturning

<u>Calculate the Maximum Bearing Pressure</u>

The Allowable Stress of the Foundation is to be 400 Kpa

We will analyze the Foundation Stability by checking the stress under the Caisson against the Allowable Stress of the Foundation

$W_e = W - U$

$W_e = 1698.11\ kN/m - 220.02\ kN/m$

$\boldsymbol{W_e = 1478.09\ kN/m}$

$M_e = 0.5W_eB - M_U - M_P$

$M_e = (0.5)(1478.09\ kN/m)(10\ m) - 1449.53\ kN/m - 1466.79\ kN/m$

$\boldsymbol{M_e = 4474.15\ kN/m}$

$T_e = {}^{M_e}/_{W_e}$

$T_e = \frac{4474.15\ kN/m}{1478.09\ kN/m}$

$T_e = 3.03$ < B/3 = 3.33

Therefore, we have Triangular Distribution

$P_e = \frac{2W_e}{3T_e}$

$P_e = \frac{2(1478.09\ kg/m)}{3(3.03\ m)}$

$\boldsymbol{P_e = 325.538\ kPa < Allowable\ Stress\ of\ Foundation = 400\ kPa}$

The Structure will experience no Foundation Failure

<u>Thickness and size of the Scour Protection Toe Block</u>

A = 0.21 for Breakwater Head

$t = AH_s(\frac{H'}{H})^{-0.787}$ Valid for 0.4 ≤ (H'/H) ≤ 1.0

$t = 0.21(3.096\ m)(\frac{3.5\ m}{4.6\ m})^{-0.787}$

$\boldsymbol{t = 0.81\ m}$

The thickness of the Scour Protection Toe Block is 0.81 m

The Size of the Scour Protection Toe Block can have a length of 1.2 m, a width of 2 m and a thickness of 0.81 m. The size of the scour protection block can vary in terms of length and width, it can also be 1 m long and 3 m wide or any configuration that fits the existing profile.

<u>Armor Unit Size and Mass</u>

$L' = 1.56(T_p^2)Tanh(\frac{2\pi H'}{L'})$

$L' = 1.56(21.667\ s)^2 Tanh(\frac{2\pi(3.5\ m)}{L'})$ with iteration

$L' = 126.27\ m$

$$k = \left(\frac{\frac{4\pi H'}{L'}}{\sinh\left(\frac{4\pi H'}{L'}\right)}\right) Sin^2\left(\frac{2\pi H'}{L'}\right)$$

$k = 0.029$

$N_s = \max\left(1.8, (1.3\frac{1-k}{\sqrt[3]{k}}\frac{H'}{Hs} + 1.8\exp\left(-1.5\frac{(1-k)^2}{\sqrt[3]{k}}\frac{H'}{Hs}\right)\right)$

$N_s = 4.58$

$M_A = \frac{\rho_{armor}}{N_S^3(Sr-1)^3)} H_S^3$

$S_r = \frac{\rho_{armor}}{\rho} = 2.58$

$\boldsymbol{M_A = 210.63\ kg}$

The Mass of the Armor Unit will be 210.63 kg

$\boldsymbol{D_n = (\frac{M_A}{\rho_{armore}})^{1/3}}$

$\boldsymbol{D_n = 0.434\ m < t = 0.81\ m}$

The Armor Unit Size for the foundation will be 0.434 m

Wave Overtopping Rate

$d^* = \frac{d}{H_s}\frac{2\pi H}{gT_p^2} = 0.005\ m$

$R_d = \frac{H_c}{H_s} d^* = 0.0098\ m$

$Q_{\#} = 0.03\exp\left(-2.05\frac{H_c}{H_s}\right) = 0.00056$

Q = Wave Overtopping Rate = $Q_{\#}(gH_S^3)^{0.5}$

Q = 9.626 $\boldsymbol{\frac{l}{m\ s}}$

The overtopping rate for this design will be 9.626 l/ms which is below the Allowable Overtopping rate of 10 $\boldsymbol{\frac{l}{m\ s}}$**. Overtopping rates within the vicinity of 10** $\boldsymbol{\frac{l}{m\ s}}$ **would indicate accuracy in the results.**

5. FINAL RESULTS

Overall, a seawall structure was made that would be able to withstand critical storm surges generated over a 500-year period, would allow a moderate overtopping rate, and allow for an access road. Setting up with a 500 Year Storm Surge event with the addition of zero surge waves from a period of 34 years. A comprehensive wave transformation allowed the selection of a Ho value. Utilizing the storm surge of 6.2 m as a basis for the selection of an H value, while also taking into consideration the 2 m elevation at the Toe of the proposed seawall. Values of H, H', d, and Hc were determined. After several designs which generated wave overtopping's exceeding 10, the following design was ultimately selected. Along with the thickness of the Toe, the mass and size of the armor units. The design and measurements for the height, the breakwater height, the maximum wave height, the significant wave height was determined. The results of these measurements are recorded in table 6. Figure 14 indicates the elevations of the seawall designed.

Table 6 – Design of Seawall Structure parameters

H (m)	**4.6**	U (kN/m)	220.02
D (m)	2.5	Mp (kN m/m)	1449.53
H' (m)	3.5	Mu (kN m/m)	1466.78
Hc (m)	6.0	**B (m)**	**10**
Tp @ 500 Years (s)	21.667	Berm (m)	3
Ho (m)	2.39	ρ (kg/m^3)	1030
L (m)	144.53	P_{armor} (kg/m^3)	2650
Hs (m)	3.095	Ma (kg/m)	204000
Hb (m)	4.755	f for rubble mound	0.6
Hmax (m)	4.113	S.F. Sliding	2.16
ξ^* (m)	6.169	S.F. Overtopping	4.84
P_1 (kN/m^2)	62.450	Pe for Bearing Capacity (kPa)	325.54
P_2 (kN/m^2)	61.221	t (m) thickness of toe block	0.806
P_3 (kN/m^2)	61.515	Ma (kg) Mass of Armor Layer	210.63
P_4 (kN/m^2)	1.711	Dn (Armor Unit Size) (m)	0.434
P_U (kN/m^2)	44.004	Q (l/ms) Overtopping Rate	9.63
P (kN/m)	409.42		

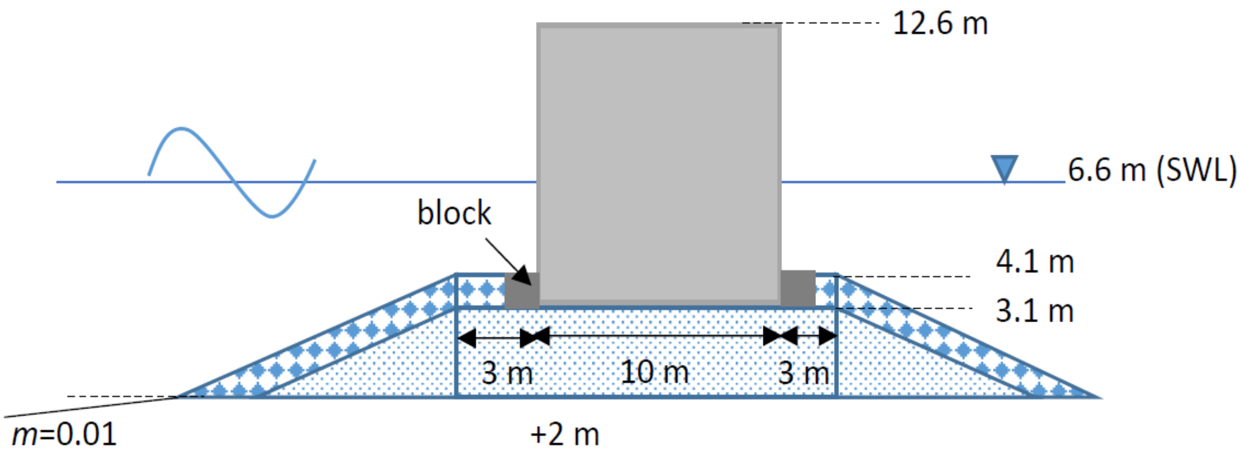

Figure 16: General Elevations of the designed Vertical Breakwater structure

Altogether this design works for the given wave surge and water level, depending on the criteria provided by the client, several factors may change. One example of a different structure with different values is recorded below on Tables 7.

Table 7 – Alternate Design of Seawall Structure parameters

H (m)	5.0	**U (kN/m)**	280.69
D (m)	2.5	**Mp (kN m/m)**	1506.88
H' (m)	3.0	**Mu (kN m/m)**	2245.53
Hc (m)	7.0	**B (m)**	12
Tp @ 500 Years (s)	21.667	**Berm (m)**	3
Ho (m)	2.39	**ρ (kg/m^3)**	1030
L (m)	150.59	**P_{armor} (kg/m^3)**	2650
Hs (m)	3.124	**Ma (kg/m)**	204000
Hb (m)	5.165	**f for rubble mound**	0.6
Hmax (m)	4.372	**S.F. Sliding**	1.85
ξ^* (m)	6.557	**S.F. Overtopping**	5.03
P_1 (kN/m^2)	70.62	**Pe for Bearing Capacity (kPa)**	279.67
P_2 (kN/m^2)	69.14	**t (m) thickness of toe block**	1.04
P_3 (kN/m^2)	69.74	**Ma (kg) Mass of Armor Layer**	422.18
P_4 (kN/m^2)	4.76	**Dn (Armor Unit Size) (m)**	0.545
P_U (kN/m^2)	46.78	**Q (l/ms) Overtopping Rate**	7.44
P (kN/m)	441.01		

Note: This Alternate Design may also work depending on the criteria given

Appendix A – Citations

Wave Overtopping of Seawalls. (n.d.). Retrieved from http://www.overtopping-manual.com/assets/downloads/EA_Overtopping_Manual_w178.pdf

Appendix B – Excel Files

Please see attached Excel File (Profile Modified for the Final Exam)

Note: The different surges are attached, Sheets Named Vertical Breakwater Design Fina and Alternate contains the results pertaining to the Final Exam