Coastal Texas Protection and Restoration Feasibility Study

Final Feasibility Report

Appendix D – Annex 4:

Mott MacDonald (MM) Report #3 - Wave Loading Report

August 2021

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Coastal Texas Study

Wave Loading Report

December 16, 2020

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Coastal Texas Study

Wave Loading Report

December 16, 2020

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Contents

Executive summary

Mott MacDonald has conducted a wave loading analysis to determine the 100-year wave loads on the proposed Clear Creek and Dickinson Bayou structures using results from the USACE storm simulation suite to determine the extreme values of wave and water surface elevation conditions at each project site.

The following paragraphs summarizes the pertinent findings from Mott MacDonald's wave loading analysis:

Water Surface Elevation: The 100-year water surface elevation (WSE) was derived from the extremal statistics provided by the USACE. The extremal statistics were calculated by the USACE using the Joint Probability Method (JPM), which provides an advantage over historical point gauge analysis by simulating a suite of synthetic storms that cover the entire probability space of tropical storms. In accordance with HSDRSS (USACE, 2012) design guidelines, the 100-year water surface elevation at the 90% confidence level (CL) was used. To include relative sea level rise (RSLR) in the wave loading calculations, the future with project (2085) 90% CL, 100-year water surface elevation was used for all wave loading calculations. The (2085) 90% CL, 100-year WSE is 12.8 ft NAVD88 at Dickinson Bayou, and 13.5 ft NAVD88 at Clear Creek.

Wave Conditions: Extremal statistics for wave height and wave period have not yet been generated at the time of this report writing. However, the USACE has provided MM with wave period and wave height results for all 20 storms simulated in the JPM simulation suite. To determine the extremal wave conditions, MM extracted the wave period and wave height for all storms within a 2 foot bounds of the extremal WSE. This analysis results in a 100-year significant wave height of 7.0 feet at Clear Creek, and 5.9 ft at Dickinson Bayou.

Wave Loads: Wave loads were calculated at each site using the methodology of Goda (USACE, 2012). This methodology calculates the hydrostatic and wave forces on a wall structure. The force calculations account for differential head on the wall structure. Wave loads were computed for each of the 20 storm conditions. A summary of the maximum wave load conditions for all 20 storms is included in Appendix C. When the statistics are available, these loads can be used to derive the 100-yr load conditions based on the probability of occurrence of each storm. In order to generate loads for conceptual design during this phase of the work before the statistics are available, a conservative estimate was established. All resulting wave heights within the WSE bounds, along with their corresponding peak periods, were used as input to compute wave loads using Goda's formulation for wave loads on a vertical wall (USACE 2012). The storm causing the maximum resulting wave load was then taken as the design condition. This is a conservative result and can be improved upon (likely decreased) when the statistics become available. In general, wave forces are higher on the Clear Creek than the Dickinson Bayou structure. See Section 3.2 for further discussion of the maximum loading, and Appendix B for wave loading distributions.

1 Introduction

The purpose of this technical memorandum is to describe the wave loading analysis performed on the Clear Creek and Dickinson Bayou structures. This memorandum summarizes the data collection effort conducted by Mott MacDonald (MM). As part of the data collection effort, MM summarized extremal water surface and wave conditions to be used in the wave loading analysis. Then, MM calculated wave loads on the Dickinson Bayou and Clear Creek structures.

2 Data Collection

The sections below outline the data provided by the USACE to MM and the data collected by MM, along with the sources from which the data was collected.

2.1 Datum

The data provided by USACE was referenced to both MLLW and NAVD88. The conversion from MLLW to NAVD88 is specific to a given location. These conversions are provided on the NOAA Tides & Currents website (NOAA, 2018). The available gage locations for Galveston Bay are shown in Figure 1. Due to its proximity to both project sites, MM recommends using the datum conversions from the Eagle Point gage at both the Dickinson Bayou and Clear Creek project sites. The NOAA website does not provide the conversion to NAVD88 for the Eagle Point gage. The following is an extract from the NOAA's website as to why the NAVD88 conversion is not shown at a gage.

> *"The NAVD88 elevation is shown on the Elevations of Tidal Datums Table Referred to MLLW only when two or more of the bench marks listed have NAVD88 elevations. The NAVD88 elevation relationship shown in the table is derived from an average of several bench mark elevations relative to tide station datum. As a result of this averaging, NAVD88 bench mark elevations computed indirectly from the tidal datums elevation table may differ slightly from NAVD88 elevations listed for each bench mark in the NGS database."*

In summary, NOAA only provides the NAVD88 conversion when 2 benchmarks can be used to verify the elevation of the gage. For this location only 1 benchmark was available for use. The conversion based on 1 benchmark is shown on the National Geodetic Survey website (NGS, 2018). The datum conversion can be found at (https://www.ngs.noaa.gov/Tidal_Elevation/diagram.xhtml?PID=AJ4424&EPOCH=1983-2001). Table 1 shows the difference between the NGS and NOAA datum conversions. The difference for any datum is 0.01 feet or less. MM recommends using the NOAA values for all conversions except for the NAVD88. MM recommends using the NGS conversion for NAVD88, which is shown below:

- NAVD88 = 0.24 feet MLLW, based on NGS conversion at Eagle Point Gage.

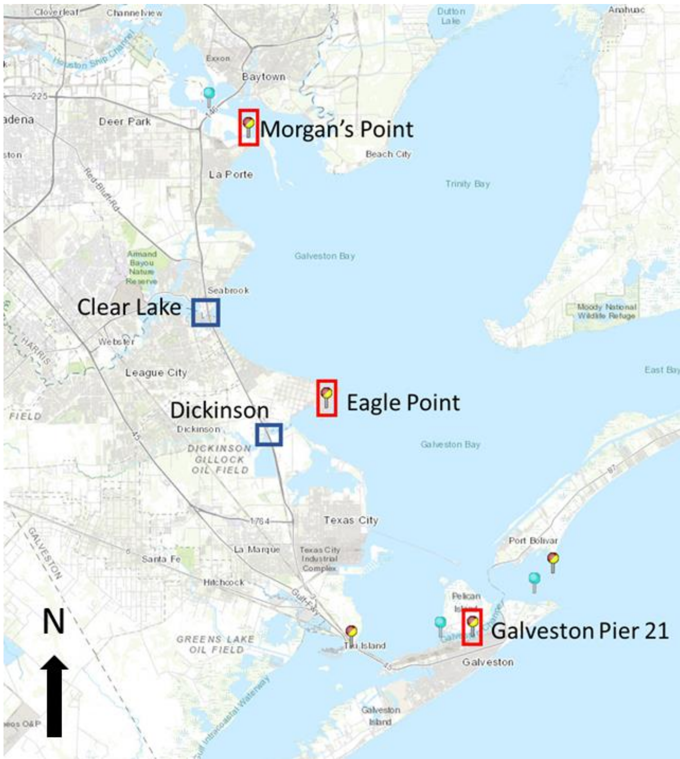


Figure 1. Location of NOAA Gages in Galveston Bay

Table 1. Datum Conversions at Eagle Point in feet.

Datum	Recommendation: NOAA [ft NAVD88]
MHHW	0.86
MHW	0.82
MSL	0.35
NAVD88	0
MLW	-0.19
MLLW	-0.24

2.2 Water Surface Elevation Extremal Statistics

The USACE provided MM with extremal water surface elevation (WSE) values for all relative sea level rise (RSLR) conditions. These files were provided for both the future without project (FWOP) and Alternative A (Alt-A) project conditions. The values provided included the 2%, 16%, 50%, 84%, and 98% confidence limit WSE statistics for the 0 SLR condition. To be consistent with the HSDRRS (USACE, 2012) requirements, the 90% confidence limit will be used for the project design.

The WSE value for the 90% confidence limit were computed by extrapolating from the provided confidence intervals at the project locations. The WSE values for Dickinson Bayou are shown in Table 2 for both Alt-A and without project conditions. The WSE values for Clear Creek are shown in Table 3 for both Alt-A and FWOP. Both extraction points were taken on the Galveston Bay side of the proposed structure Alignment. The extraction locations are shown in Figure 2 and Figure 3 for Dickinson Bayou and Clear Creek respectively. For the 2035 and 2085 scenario, the same equation for extrapolation that was created from the confidence limit values for 0 RSLR conditions was used to compute the 90% confidence limit WSE.

Figure 2. Polygon used to extract WSE values for Dickinson Bayou.

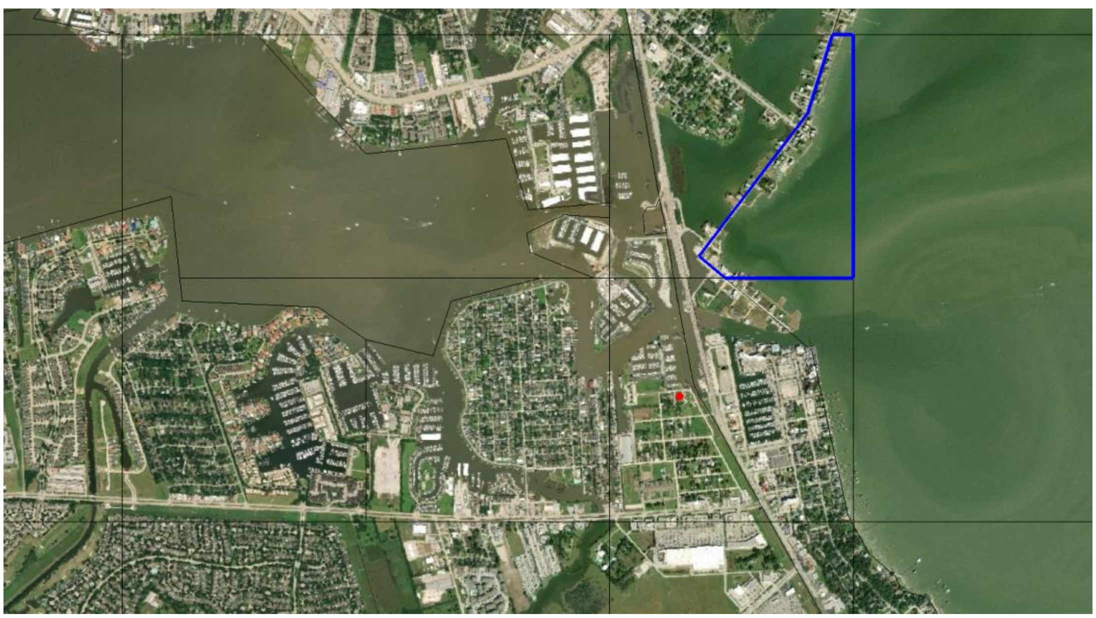


Figure 3. Polygon used to extract WSE values for Clear Creek.

Table 2. 90% Confidence limit WSE [ft NAVD88] for Dickinson Bayou.

SLR	Scenario	100-yr Return Period	200-yr Return Period	500-yr Return Period
2017	FWOP	14.1	16.5	19.0
2035	FWOP	14.7	17.1	19.6
2085	FWOP	16.7	19.1	21.6
2017	Alt A	10.0	11.9	14.6
2035	Alt A	10.7	12.6	15.3
2085	Alt A	12.8	14.7	17.4

Table 3. 90% Confidence limit WSE [ft NAVD88] for Clear Creek.

SLR	Scenario	100-yr Return Period	200-yr Return Period	500-yr Return Period
2017	FWOP	14.8	17.4	20.0
2035	FWOP	15.5	18.0	20.6
2085	FWOP	17.4	20.0	22.6
2017	Alt A	10.7	12.2	14.1
2035	Alt A	11.4	12.9	14.8
2085	Alt A	13.5	15.1	16.9

2.3 Structure and Sill Elevations

The proposed wall and sill elevations are shown in Table 4. The initial sill elevations were provided to MM in feet MLLW, while the Wall elevations were in feet NAVD88. Using the datum conversions from Section 2.1 both the NAVD88 and MLLW elevations for each location were calculated and are presented in Table 4.

Table 4. Elevation of Top of Wall and Sills.

	Sill Elevations		Top of Wall Elevations	
	[ft MLLW]	[ft NAVD88]	[ft MLLW]	[ft NAVD88]
Dickinson	-9	-9.24	18.24	18
Clear Creek	-12	-12.24	17.24	17

To refine the top of wall elevations, MM conducted an overtopping analysis at Clear Creek and Dickinson Bayou. HSDRRS (USACE, 2012) suggests a maximum overtopping rate of 0.1 cfs/ft (9.3 L/s/m). However, HSDRRS states that this is a site-specific overtopping rate. Since the HSDRRS guidelines are site specific, overtopping guidance from the *Coastal Engineering Manual* (USACE, 2012) was investigated. The chart below shows the safe overtopping rates for various structures as suggested by USACE, 2012.

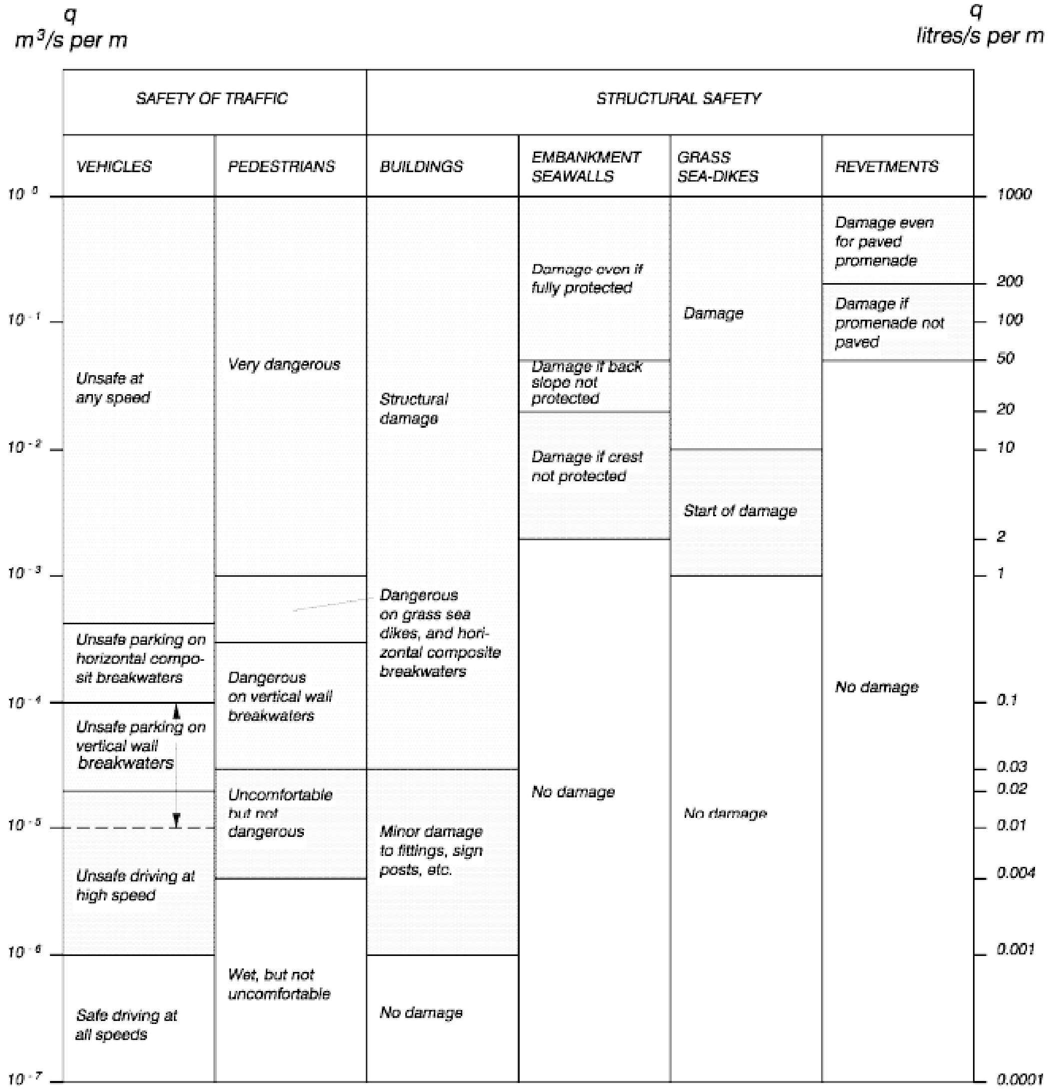


Figure 4. Safe overtopping guidelines provided in USACE, 2012a

Varying top elevations of the floodwall were tested at each site. A peak overtopping rate of 0.39 cfs/ft (36 l/s/m) was calculated at Clear Creek with a +17 ft NAVD88 wall, and 0.48 cfs (45 l/s/m) at Dickinson Bayou with a +18 ft NAVD88 wall. Both flowrates fall under the “Damage if back slope not protected” category for embankments and seawalls. Since there is no infrastructure on the immediate backside of the Clear Creek and Dickinson structures, these overtopping rates were deemed appropriate so long as protection is added to the backside of the structures. The peak overtopping rates were included in the pump station design conducted by MM. See the drainage memorandum for further discussion of the overtopping calculations and pump station design.

2.4 Waves

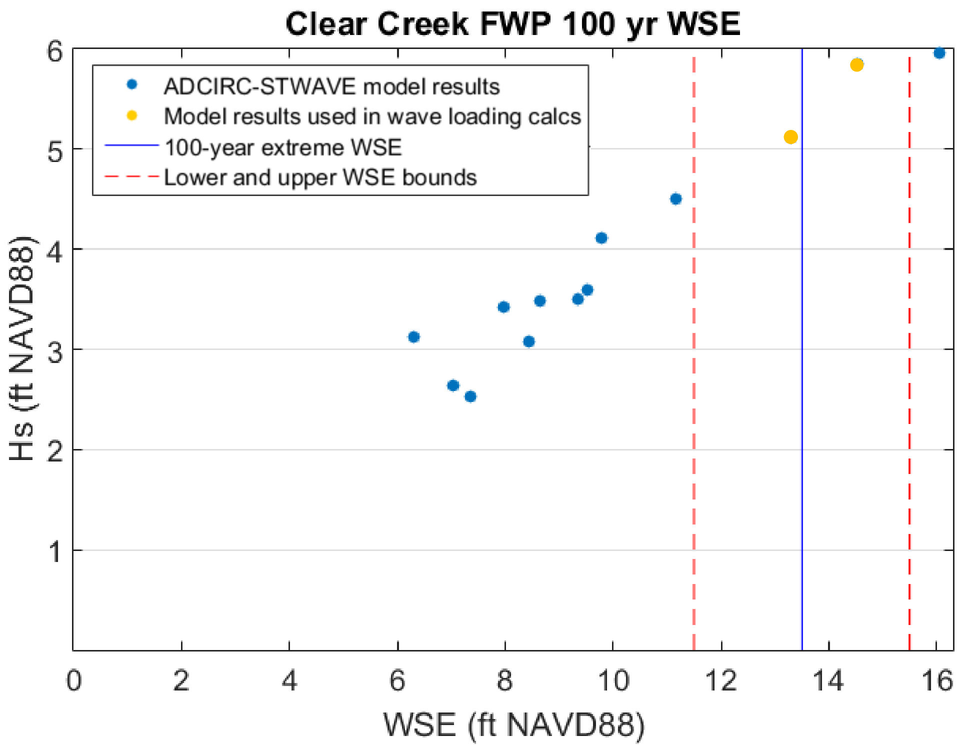
To find the wave height and period associated with the extremal WSE for each scenario, MM extracted wave data results from the ADCIRC-STWAVE storm simulations conducted by the USACE. The storm suite modeled by the USACE consisted of 20 runs, which comprise the Joint Probability Analysis (JPA) developed by the USACE. These 20 storms were simulated with a high fidelity numerical model to determine the various storm responses at selected save points. The storm responses include wave height, peak period, water surface elevation, and other parameters resulting from tropical cyclone forcing. The USACE then used the joint probability method with optimal sampling (JPM-OS) to perform a statistical analysis of the tropical cyclones, and to generate extremal water surface elevation statistics at all save points. Figure 5 and Figure 6show the extraction points used to generate the extremal wave loading input conditions at Dickinson Bayou and Clear Creek, respectively.

Figure 5: Extraction Points and Proposed Structures for Dickinson Bayou

Figure 6: Extraction Points and Proposed Structures for Clear Creek

To compute a 100-yr load on the walls, we recommend computing the max load for each individual storm in the JPM set, and then use the JPM to derive the extreme value statistics to derive the 100-yr load from these load cases. The maximum loads from each storm are proided in Appendix B.

At the time of this report writing, the statistics for the storm conditions at the site are in development. Therefore, to determine the wave loading conditions for use in conceptual design for each feature, a conservative method was developed to determine the loads. For each return period, MM selected the results from the USACE study that displayed WSEs within 2 feet of the extremal WSE for a given return period. Figure 7 illustrates the methodology employed to determine the simulation results used for wave loading calculations. All resulting wave heights within the WSE bounds, along with their corresponding peak periods, were used as input to compute wave loads using Goda's formulation (USACE 2012). The maximum resulting wave load for each return period was then taken as the design condition. Wave heights and periods for the corresponding return periods are given in Table 5.



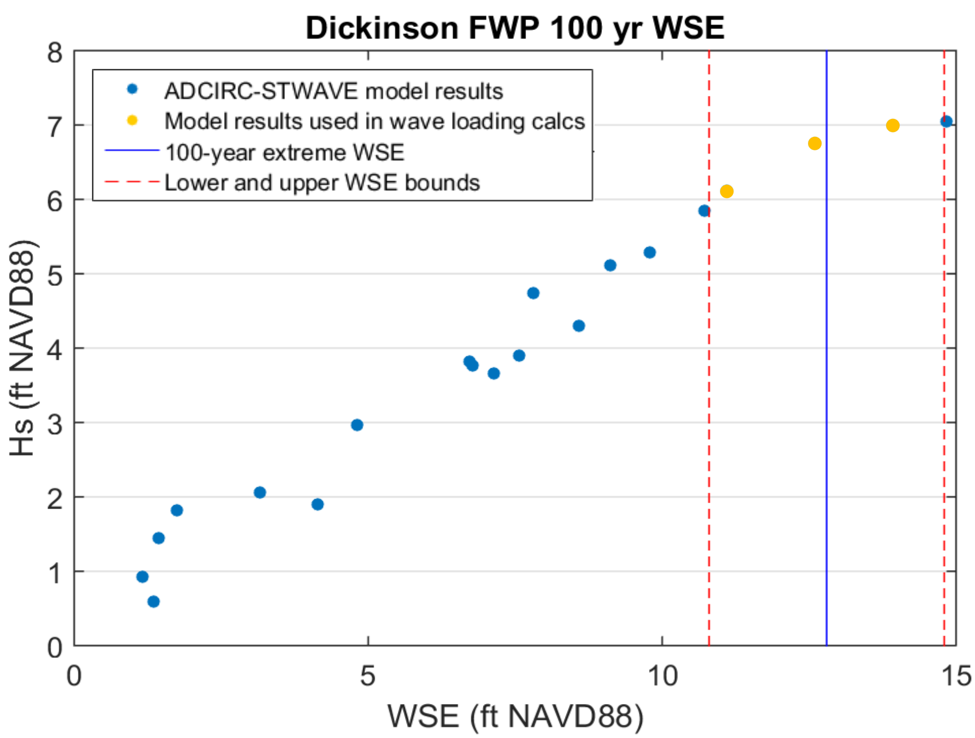


Figure 7: Model results for all storm simulations and extremal WSE for 100-year return period with +- 2ft bounds

Table 5: Wave Heights and Peak Periods Extracted from USACE's ADCIRC-STWAVE Study

Location	SLR	WSE Return Period	Scenario	Wave Height [ft]	Wave Period [sec]
Dickinson	2085	100-yr	Alt A	7.0	4.7
Dickinson	2085	200-yr	Alt A	7.1	7.6
Dickinson	2085	500-yr	Alt A	7.1	7.6
Clear Creek	2085	100-yr	Alt A	5.9	7.6
Clear Creek	2085	200-yr	Alt A	6.0	7.6
Clear Creek	2085	500-yr	Alt A	6.0	7.6

3 Wave Loading Analysis

3.1 Extremal Input Conditions

Table 6 below summarizes the input used for the extreme wave loading calculations for the Alt-A scenario. Scenarios considered include 100, 250 and 500-year return period extreme Max WSEs with the SLR projection from the year 2085. The Low WSE reflects the WSE on the landward side of the flood wall, which will be maintained via a pumping system. The seabed elevation (Depth) at each site is listed in the table and the slope was considered 1/50 at both sites. Necessary input to complete Goda's formula include the significant wave height (H_s) and peak period (T_p), which were extracted from the USACE's ADCIRC-STWAVE wave study as described in the previous section.

Table 6: Input for Wave Loading Calculations

Case #	SLR [year]	WSE Return Period [year]	Scenario	Location	Outer WSE [ft NAVD88]	Inner WSE [ft NAVD88]	Depth [ft NAVD88]	H_s [ft]	T_p [sec]
1	2085	100	Alt-A	Dickinson	12.80	-1.24	-13.24	7.0	4.7
2	2085	200	Alt-A	Dickinson	14.70	-1.24	-13.24	7.1	7.6
3	2085	500	Alt-A	Dickinson	17.40	-1.24	-13.24	7.1	7.6
10	2085	100	Alt-A	Clear Creek	13.50	-1.24	-28.24	5.7	7.6
11	2085	200	Alt-A	Clear Creek	15.10	-1.24	-28.24	6.0	7.6
12	2085	500	Alt-A	Clear Creek	16.90	-1.24	-28.24	6.0	7.6

3.2 Goda's Wave Forces

Goda's formulation (USACE, 2012) was used to compute the hydrostatic and hydrodynamic forces caused by the waves impacting the wall. The wave induced force assumes a breaking wave impacting the structure head on. The hydrostatic forces are caused by the difference in water surface elevation on either side of the wall. An example calculation sheet detailing this methodology is shown in Appendix A, and a force distribution illustration and graphs are given in Appendix B. The maximum combined force (Max Force) and the corresponding Hydrostatic and Wave Force components along with the elevation of the resultant maximum combined forces (Resultant Elev.) are shown in Table 7.

Table 7: Waves Forces for Project Locations

	SLR	Return Period	Scenario	Max Force [kip/ft]	Wave Force [kip/ft]	Hydrostatic Force [kip/ft]	Resultant Elev. [ft NAVD88]
Dickinson	2085	100-yr	Alt-A	28.8	11.7	17.1	0.1
Dickinson	2085	200-yr	Alt-A	38.2	17.8	20.4	0.3
Dickinson	2085	500-yr	Alt-A	42.5	17.1	25.5	0.5
Clear Creek	2085	100-yr	Alt-A	50.2	17.8	32.5	-8.2
Clear Creek	2085	200-yr	Alt-A	54.6	17.7	36.8	-8.0
Clear Creek	2085	500-yr	Alt-A	59.2	17.2	41.9	-7.8

4 References

Goda, Y. (2011). Reanalysis of Regular and Random Breaking Wave Statistics. *Coastal Engineering Journal*.

NGS. (2018). Retrieved from National Geodetic Survey: https://www.ngs.noaa.gov

NOAA. (2018). *Tides & Currents*. Retrieved from https://tidesandcurrents.noaa.gov

USACE. (2012). *Coastal Engineering Manual.*

USACE. (2012). *Hurricane and Storm Damage Risk Reduction System Design Guidelines.*

Appendices

A. Example Calculation Sheet

Example Wave Loading Calculations

TABLE OF CONTENTS

1 Calculation Scope

Calculate wave loads on the flood wall and gate structure at Dickinson Bayou and Clear Creek using methodology prescribed in the Coastal Engineering Manual (USACE, 2012). This example calculation shows the methodology used for all wave loading calculations.

2 Technical Requirements

To be consistent with the HSDRRS design requirements, the 90% confidence limit water surface elevation (WSE) will be used for the project design.

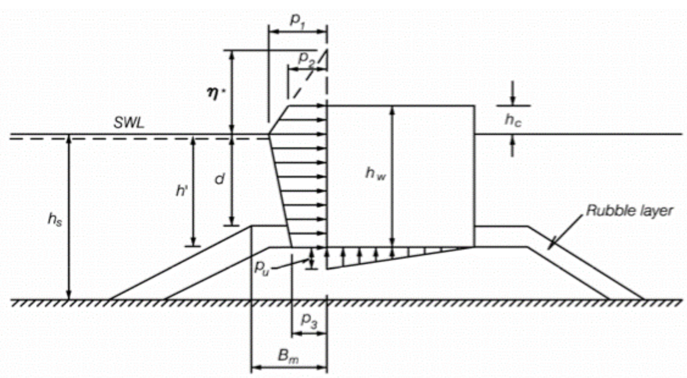
3 Criteria, Codes, and Standards

USACE Coastal Engineering Manual (2012)
Hurricane and Storm Damage Risk Reduction System Design Guidelines (2012)

4 Methodology and Assumptions

Background

Total loading on the structure includes hydrostatic and hydrodynamic forces caused by the head difference across the structure and waves impacting the wall. The wave forces were computed using Goda's formulation in the Coastal Engineering Manual (USACE, 2002) with the assumption of a breaking wave impacting the structure head on in order to give the worst-case scenario. The calculations considered no berm around the structure as they are essentially sheet piles drove directly into the seabed. The hydrostatic forces were calculated considering the water levels on both side of the structure. The total loading considers a combination of both the hydrostatic and wave forces. Calculations are given for the 100-yr storm event at Dickinson Bayou.



$$\eta^* = 0.75(1+cos\beta)\,\lambda_1 H_{design} \qquad (VI-5-147)$$

$$p_1 = 0.5(1+cos\beta)(\lambda_1\alpha_1+\lambda_2\alpha_* cos^2\beta)\,\rho_w\, g\, H_{design} \qquad (VI-5-148)$$

$$p_2 = \begin{cases} \left(1-\frac{h_c}{\eta^*}\right)p_1 & \text{for } \eta^* > h_c \\ 0 & \text{for } \eta^* \le h_c \end{cases} \qquad (VI-5-149)$$

$$p_3 = \alpha_3\, p_1 \qquad (VI-5-150)$$

$$p_u = 0.5(1+cos\beta)\lambda_3\alpha_1\alpha_3\rho_w g\, H_{design} \qquad (VI-5-151)$$

where

β Angle of incidence of waves (angle between wave crest and front of structure)

H_{design} Design wave height defined as the highest wave in the design sea state at a location just in front of the breakwater. If seaward of a surf zone Goda (1985) recommends for practical design a value of 1.8 H_s to be used corresponding to the 0.15% exceedence value for Rayleigh distributed wave heights. This corresponds to $H_{1/250}$ (mean of the heights of the waves included in 1/250 of the total number of waves, counted in descending order of height from the highest wave). Goda's recommendation includes a safety factor in terms of positive bias as discussed in Table VI-5-55. If within the surf zone, H_{design} is taken as the highest of the random breaking waves at a distance $5H_s$ seaward of the structure.

Goda's formula for irregular waves (Goda 1974; Tanimoto et al. 1976):

$$\alpha_* = \alpha_2$$

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

$$\alpha_2 = \text{the smallest of } \frac{h_b - d}{3\,h_b}\left(\frac{H_{design}}{d}\right)^2 \text{ and } \frac{2d}{H_{design}}$$

$$\alpha_3 = 1 - \frac{h_w - h_c}{h_s}\left[1 - \frac{1}{cosh\,(2\pi h_s/L)}\right]$$

L Wavelength at water depth h_b corresponding to that of the significant wave $T_s \simeq 1.1T_m$, where T_m is the average period.

h_b Water depth at a distance of $5H_s$ seaward of the breakwater front wall.

λ_1, λ_2 and λ_3 are modification factors depending on the structure type. For conventional vertical wall structures, $\lambda_1 = \lambda_2 = \lambda_3 = 1$. Values for other structure types are given in related tables.

Modifications including impulsive forces from head-on breaking waves (Takahashi, Tanimoto, and Shimosako 1994a):

$$\alpha_* = \text{largest of } \alpha_2 \text{ and } \alpha_I$$

$$\alpha_2 = \text{the smallest of } \frac{h_b - d}{3\,h_b}\left(\frac{H_{design}}{d}\right)^2 \text{ and } \frac{2d}{H_{design}}$$

$$\alpha_I = \alpha_{I0} \cdot \alpha_{I1}$$

$$\alpha_{I0} = \begin{cases} H_{design}/d & \text{for} \quad H_{design}/d \le 2 \\ 2.0 & \text{for} \quad H_{design}/d > 2 \end{cases}$$

$$\alpha_{I1} = \begin{cases} \frac{\cos\delta_2}{\cosh\delta_1} & \delta_2 \le 0 \\ \frac{1}{\cosh\delta_1 \cdot (\cosh\delta_2)^{\frac{1}{2}}} & \delta_2 > 0 \end{cases}$$

$$\delta_1 = \begin{cases} 20 \cdot \delta_{11} & \text{for} \quad \delta_{11} \le 0 \\ 15 \cdot \delta_{11} & \text{for} \quad \delta_{11} > 0 \end{cases}$$

$$\delta_{11} = 0.93\left(\frac{B_m}{L} - 0.12\right) + 0.36\left(\frac{h_s - d}{h_s} - 0.6\right)$$

$$\delta_2 = \begin{cases} 4.9 \cdot \delta_{22} & \text{for} \quad \delta_{2\,2} \le 0 \\ 3 \cdot \delta_{22} & \text{for} \quad \delta_{22} > 0 \end{cases}$$

$$\delta_{22} = -0.36\left(\frac{B_m}{L} - 0.12\right) + 0.93\left(\frac{h_s - d}{h_s} - 0.6\right)$$

$$\alpha_* = \text{largest of } \alpha_2 \text{ and } \alpha_I$$

$$\alpha_2 = \text{the smallest of } \frac{h_b - d}{3\,h_b}\left(\frac{H_{design}}{d}\right)^2 \text{ and } \frac{2d}{H_{design}}$$

$$\alpha_I = \alpha_{I0} \cdot \alpha_{I1}$$

$$\alpha_{I0} = \begin{cases} H_{design}/d & \text{for} \quad H_{design}/d \le 2 \\ 2.0 & \text{for} \quad H_{design}/d > 2 \end{cases}$$

$$\alpha_{I1} = \begin{cases} \frac{\cos\delta_2}{\cosh\delta_1} & \delta_2 \le 0 \\ \frac{1}{\cosh\delta_1 \cdot (\cosh\delta_2)^{\frac{1}{2}}} & \delta_2 > 0 \end{cases}$$

$$\delta_1 = \begin{cases} 20 \cdot \delta_{11} & \text{for} \quad \delta_{11} \le 0 \\ 15 \cdot \delta_{11} & \text{for} \quad \delta_{11} > 0 \end{cases}$$

$$\delta_{11} = 0.93\left(\frac{B_m}{L} - 0.12\right) + 0.36\left(\frac{h_s - d}{h_s} - 0.6\right)$$

$$\delta_2 = \begin{cases} 4.9 \cdot \delta_{22} & \text{for} \quad \delta_{2\ 2} \le 0 \\ 3 \cdot \delta_{22} & \text{for} \quad \delta_{22} > 0 \end{cases}$$

$$F_{sol1} = \left[\frac{(p_1 + p_2)}{2}\right] \cdot h_c + \left[\frac{(p_1 + p_3)}{2}\right] \cdot h'$$

$$F_{sol2} = \left[\frac{(p_1 + p_2)}{2}\right] \cdot \eta_* + \left[\frac{(p_1 + p_3)}{2}\right] \cdot h'$$

$$F_{sol3} = \left[\frac{(p_4 + p_3)}{2}\right] \cdot h_w$$

$$F_{wave} := if\left(h_c \ge 0 \wedge h_c \le \eta star, F_{sol1}, if\left(h_c \ge 0 \wedge h_c > \eta star, F_{sol2}, if\left(h_c < 0, F_{sol3}, 0\right)\right)\right)$$

5 Computations and Results

INPUT

100-ye extreme water surface elevation	WSE	3.90	m NAVD88
Depth below 0 NADV88	d_{datum}	4.04	m NAVD88
Top of wall elevation	h_w	5.49	m NAVD88
Total Water depth = h_w + d_{datum}	h_s	7.94	m
Depth at toe of Structure	d	7.94	m
Structure draft = min(h_s, h_w)	h'	7.94	m
Freeboard = h_w - d	h_c	1.59	m
Significant wave height	Hs	2.13	m
Peak Period	Ts	4.74	s
Seabed slope	slope	1/50	
Angle of incidence of waves	β	0	degrees
Modification factors (1 for conventional vertical walls)	$\Lambda_{1\text{-}3}$	1	
Berm Width in front of structure	B_m	0	m
Head difference across wall	h_{diff}	4.3	m

INTERMEDIATE CALCUATIONS

Depth 5*H_s seaward of structure	h_b	$d + 5H_s/slope$	8.15	m
Deep water wavelength	L_o	$\frac{gT^2}{2\pi}$	35.08	m
Wavelength at structure	L	$\frac{(2\cdot\pi)}{\frac{\left[d\cdot\frac{\frac{(2\cdot\pi)}{T_p}}{\sqrt{g\cdot d}}\right]^2}{d}\cdot\left[1-exp\left[-1\left[d\cdot\frac{\left(\frac{2\cdot\pi}{T_p}\right)}{\sqrt{g\cdot d}}\right]^{(2.5)}\right]\right]^{\left(\frac{-1}{2.5}\right)}}$	31.89	m
Goda's recommended max wave height seaward of surf zone	H_{br1}	$1.8 * H_s$	3.83	m
Goda's max breaking wave height in surf zone	H_{br2}	$\left(\frac{h_b\cdot 0.17}{\frac{h_b}{L_0}}\right)\cdot\left[1-exp\left[\left(\frac{-1.5\cdot\pi\cdot h_b}{L_0}\right)\cdot\left[1+11\cdot\frac{1}{slope_{offshore}^{\left(\frac{4}{3}\right)}}\right]\right]\right]$	4.09	m
Design wave height	H_{design}	$min(H_{br1}, and\ H_{br2})$	3.83	m

Wave pressure intermediate calculations (formulas given in Methodology)

δ_{22}	-0.5148
δ_2	-2.5225
δ_{11}	-0.3276
δ_1	-6.552
α_{I1}	-0.0023
α_{I0}	0.483
α_I	0.637
α_2	0.002
α_3	0.401
α_*	0.002

RESULTS

η^*		5.75	m	18.86	ft
p_1	Wave pressure at SWL	24.69	kN/m/m	0.516	kip/ft/ft
p_2	Wave pressure at top of vertical wall	17.89	kN/m/m	0.374	kip/ft/ft
p_3	Wave pressure at base of the vertical wall	9.91	kN/m/m	0.207	kip/ft/ft
p_u	Wave uplift pressure at base of the wall	9.87	kN/m/m	0.206	kip/ft/ft
P_{Hst}	Max Hydrostatic pressure = $\rho g^* h_{diff}$	43.1	kN/m/m	0.90	kip/ft/ft

Max. Hydrostatic Pressure	$P_{bot} = \rho g^* h_{diff}$	43.1	kN/m/m	0.90	kip/ft/ft
Hydrostatic Force (bay side)	$F_{hst1} = \rho_w g^* d^2/1000$	316.99	kN/m	21.72	kip/ft
Hydrostatic Force (bayou side)	$F_{hst2} = \rho_w g^*(d-h_{diff})^2/1000$	67.32	kN/m	4.61	kip/ft
Resultant Hydrostatic Force	$F_{hst} = F_{hst1} - F_{hst2}$	249.68	kN/m	17.11	kip/ft
Horizontal Wave Force	F_{Wave} (eq. in Methodology)	181.12	kN/m	11.71	kip/ft
Resultant Maximum Force	$F_{Wave} + F_{hst}$	420.60	kN/m	28.82	kip/ft

6 Conclusions

For the 100 year return period storm event, assuming SLR from 2085,
the resultant forces on the proposed structure at Dickinson Bayou is 28.82 kip/ft.

B. Wave Loading Distributions

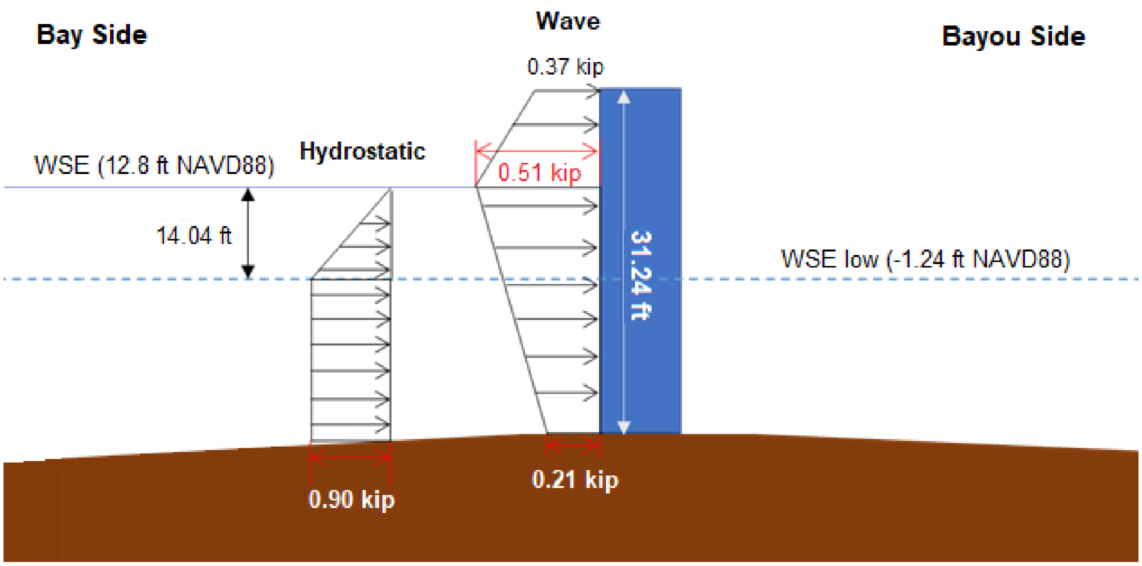


Figure B 1: Illustration of Force Distributions on Dickinson Bayou Structure

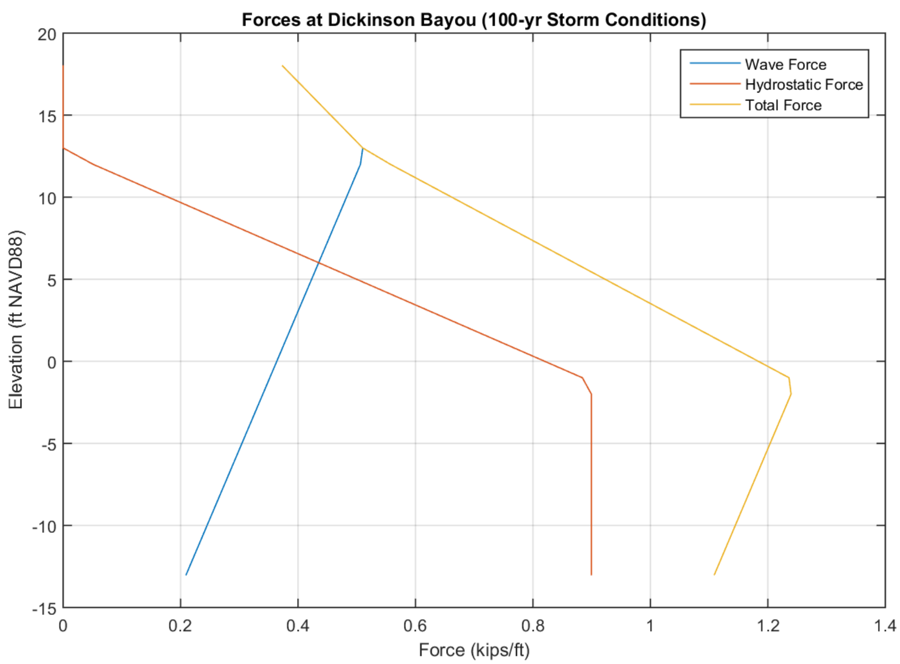


Figure B 2: Force Distribution Illustration of 100-year storm conditions at Dickinson Bayou

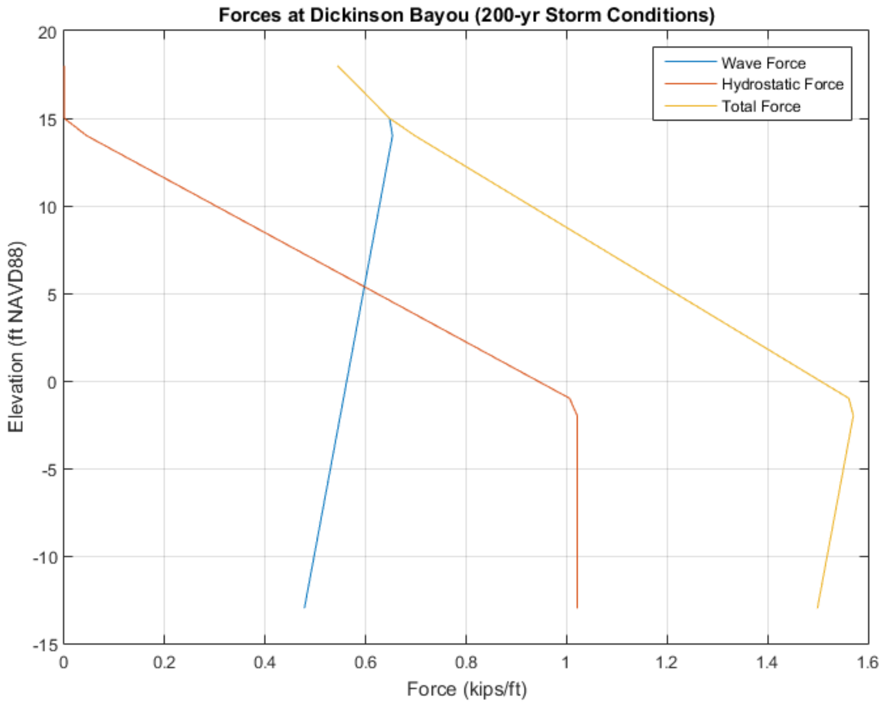


Figure B 3: Force Distribution Illustration of 200-year storm conditions at Dickinson Bayou

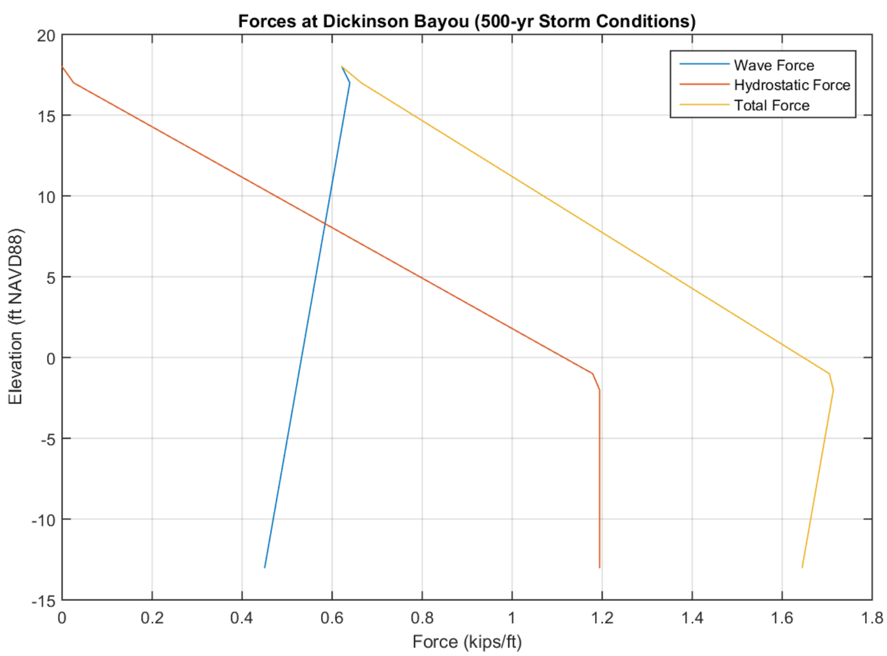


Figure B 4: Force Distribution Illustration of 500-year storm conditions at Dickinson Bayou

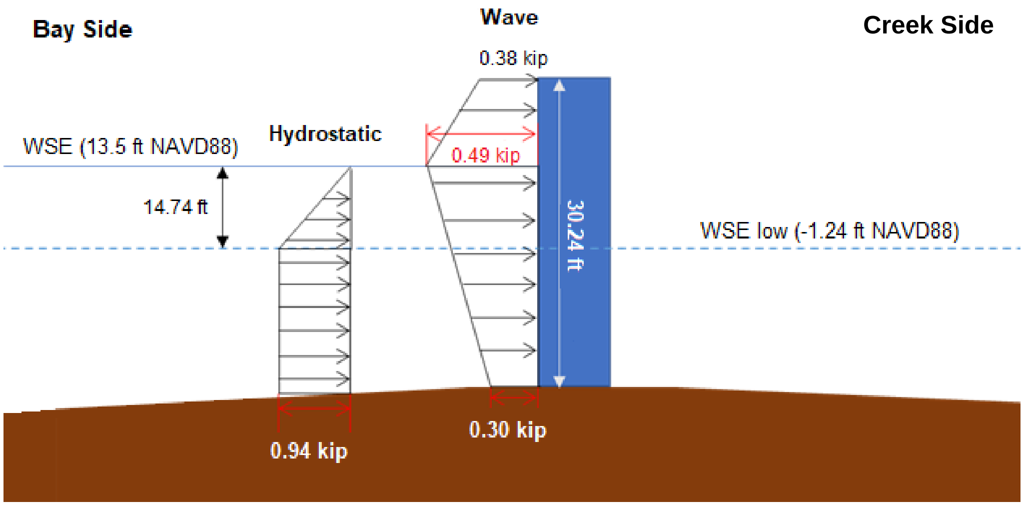


Figure B 5: Illustration of Force Distributions on Clear Creek Structure

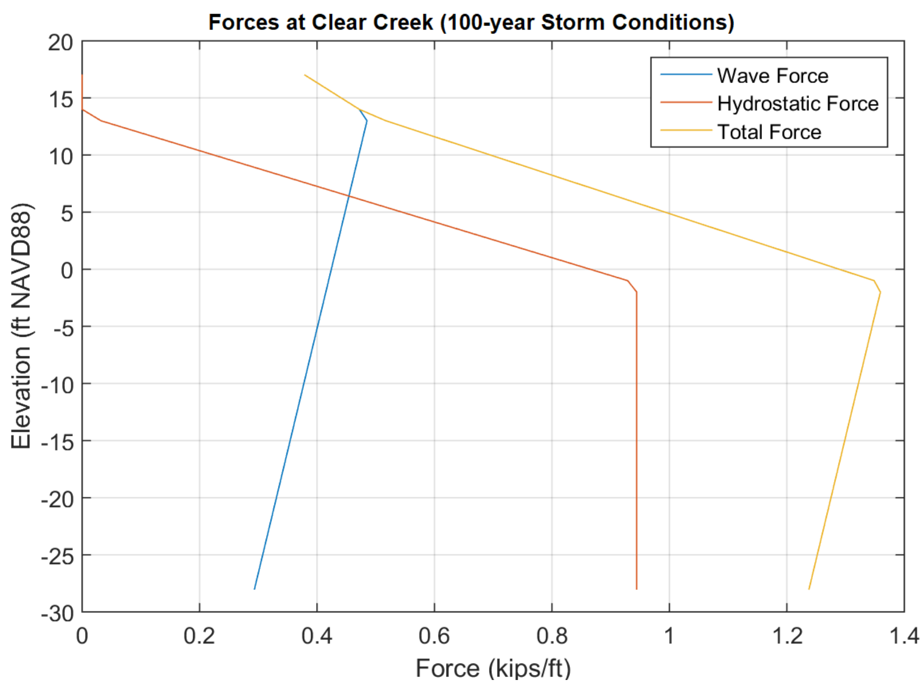


Figure B 6: Force Distribution Illustration of 100-year storm conditions at Clear Creek

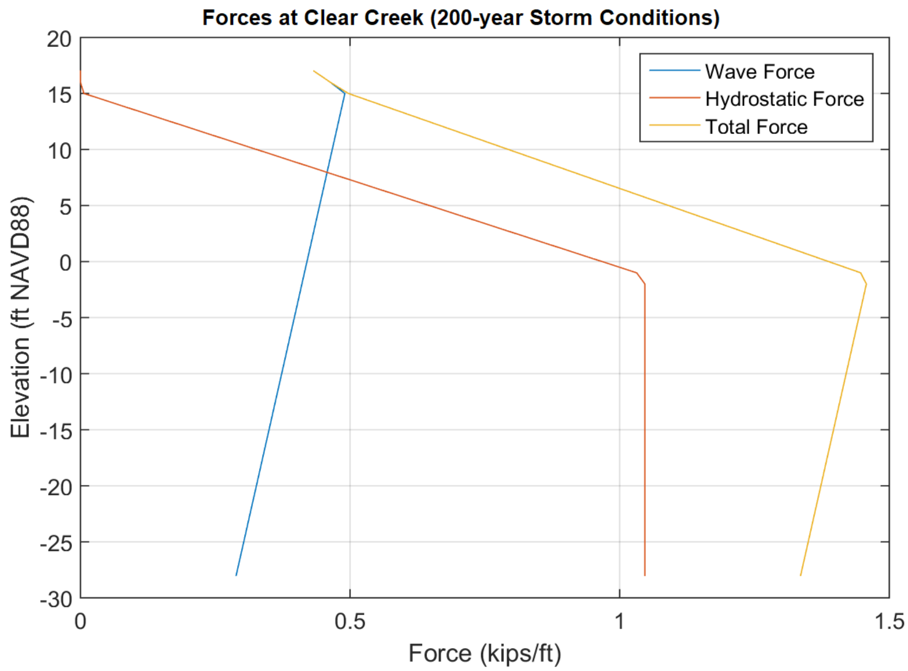


Figure B 7: Force Distribution Illustration of 200-year storm conditions at Clear Creek

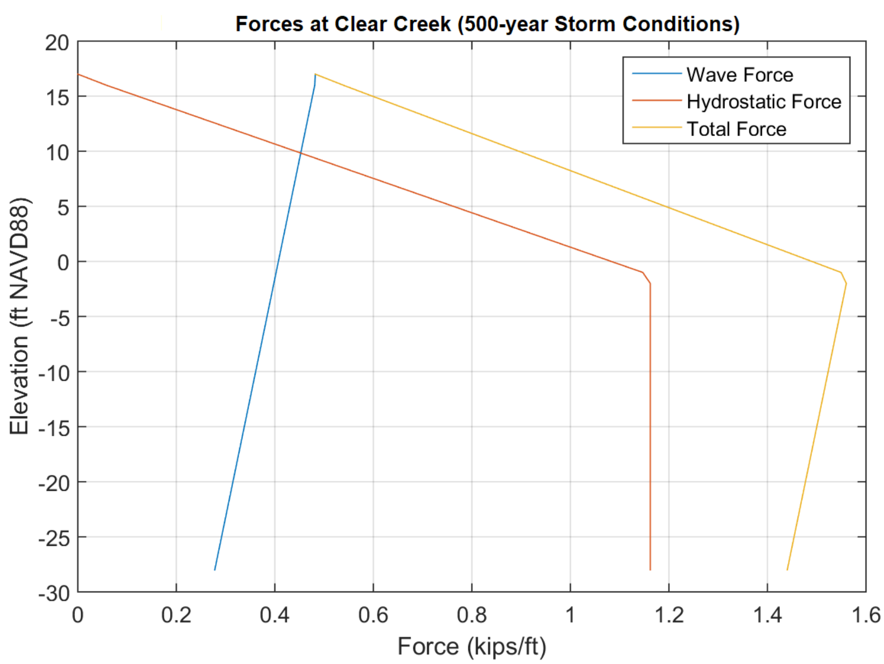


Figure B 8: Force Distribution Illustration of 500-year storm conditions at Clear Creek

C. Wave Loads for all Storms

Table C- 1: Wave Loadings at Dickinson Bayou for all USACE's ADCIRC-STWAVE Simulations

Storm #	Max WSE [ft NAVD88]	Low WSE [ft NAVD88]	Depth [NAVD88]	Hs [ft]	Tp [sec]	Max Force [kip/ft]	Wave Force [kip/ft]	Hydrostatic Force [kip/ft]	Location of Resultant Force [ft NAVD88]	WSE at Fmax [ft NAVD88]
66	9.12	-1.24	13.24	5.11	4.30	19.28	7.88	11.40	-1.13	9.12
73	12.60	-1.24	13.24	6.76	4.74	27.76	10.99	16.77	-0.24	12.60
77	8.59	-1.24	13.24	4.30	3.56	16.28	5.63	10.65	-1.55	8.59
154	9.79	-1.24	13.24	5.28	3.91	19.92	7.54	12.38	-0.93	9.79
159	3.17	-1.24	13.24	2.06	2.94	5.79	1.79	4.01	-3.18	3.17
167	1.16	-1.24	13.24	0.92	2.67	2.64	0.61	2.03	-3.99	1.16
270	1.75	-1.24	13.24	1.81	2.67	3.97	1.39	2.58	-3.02	1.75
277	1.44	-1.24	13.24	1.44	2.67	3.33	1.04	2.29	-3.33	1.44
342	7.15	-1.24	13.24	3.66	4.74	14.50	5.80	8.70	-2.35	7.15
356	11.10	-1.24	13.24	6.12	4.30	23.72	9.36	14.36	-0.52	11.10
384	1.36	-1.24	13.24	0.59	2.67	2.59	0.38	2.21	-4.73	1.36
437	4.14	-1.24	13.24	1.90	3.56	6.96	1.91	5.06	-3.43	4.14
447	7.56	-1.24	13.24	3.90	4.30	15.02	5.77	9.25	-2.07	7.56
453	14.84	-1.24	13.24	7.05	7.63	37.88	17.23	20.65	0.02	14.84
456	6.77	-1.24	13.24	3.76	3.56	12.91	4.69	8.22	-2.29	6.77
461	4.81	-1.24	13.24	2.96	2.94	8.78	2.95	5.83	-2.65	4.81
529	13.92	-1.24	13.24	6.99	4.74	30.05	11.04	19.01	-0.07	13.92
578	7.82	-1.24	13.24	4.74	3.56	15.88	6.29	9.59	-1.40	7.82
595	6.73	-1.24	13.24	3.83	3.24	12.68	4.52	8.16	-2.20	6.73
633	10.71	-1.24	13.24	5.85	3.91	22.09	8.33	13.77	-0.61	10.71

Table C- 2: Wave Loadings at Clear Creek for all USACE's ADCIRC-STWAVE Simulations*

Storm #	Max WSE [ft NAVD88]	Low WSE [ft NAVD88]	Depth [NAVD88]	Hs [ft]	Tp [sec]	Max Force [kip/ft]	Wave Force [kip/ft]	Hydrostatic Force [kip/ft]	Location of Resultant Force [ft NAVD88]	WSE at Fmax [ft NAVD88]
66	8.44	-1.24	28.24	3.08	6.31	27.30	7.56	19.74	-10.14	8.44
73	13.30	-1.24	28.24	5.12	6.93	46.05	14.13	31.92	-8.40	13.30
77	7.37	-1.24	28.24	2.53	6.31	23.27	6.00	17.27	-10.54	7.37
154	9.36	-1.24	28.24	3.50	6.93	31.60	9.67	21.93	-9.69	9.36
342	7.98	-1.24	28.24	3.42	5.21	25.75	7.07	18.68	-9.97	7.98
356	11.18	-1.24	28.24	4.51	6.93	39.06	12.64	26.42	-8.89	11.18
447	8.65	-1.24	28.24	3.48	5.21	27.50	7.26	20.24	-9.79	8.65
453	16.05	-1.24	28.24	5.96	7.63	56.98	17.49	39.49	-7.87	16.05
456	7.03	-1.24	28.24	2.65	5.21	21.72	5.22	16.50	-10.47	7.03
529	14.52	-1.24	28.24	5.85	7.63	52.79	17.56	35.22	-8.06	14.52
578	9.52	-1.24	28.24	3.60	4.30	28.92	6.60	22.32	-9.43	9.52
595	6.31	-1.24	28.24	3.13	3.24	19.67	4.79	14.88	-9.95	6.31
633	9.80	-1.24	28.24	4.11	6.31	33.47	10.48	23.00	-9.27	9.80

***Storms not shown resulted in a dry node during the storm event at clear creek, resulting in 0 wave loading.**

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