MOTT MACDONALD

ALABAMA DEPARTMENT OF TRANSPORTATION

I-10 Mobile River Bridge and Bayway Project - Storm Surge Impact Analysis Level 3

June 8th, 2018

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Executive summary

The primary goals of this report are to provide an accurate and robust analysis associated with environmental conditions and forces from tropical storms and hurricanes on the existing I-10 Mobile Bay Bayway Bridge (MBBB) and to provide environmental conditions for the proposed MBBB with the goal of facilitating the design of the bridge to mitigate damage from these forces. This report uses the AASHTO (2008) Level 3 method of analysis providing a discussion of coastal processes associated with extreme storm events at the Mobile Bay Bayway Bridge site, an evaluation of changes to those coastal processes due to climate change over the project lifetime, and computation of forces on the existing Bayway Bridge for the prescribed design condition.

Based on AASHTO (2008), “the vertical clearance of highway bridges should be sufficient to provide at least 1 ft of clearance over the 100-yr design wave crest elevation, which includes the design storm water elevation.” Thus, the design condition for the bridge was set as 100 year SLR and 100-yr return period event on the main span, and the 100 year Sea Level Rise and 50-yr storm event on the bridge ramps. Therefore, the results presented in this report include the 100 year SLR and 100-yr return period event and 100 year Sea Level Rise and 50-yr storm event.

The Level 3 analysis included first a review of bathymetric data and existing flood insurance studies. A review of existing climate change literature was conducted during Level 1 analysis to select the most appropriate Sea-Level Rise (SLR) scenario (Mott MacDonald, 2016). Based on discussions with the project team and ALDOT, Level 3 analysis employes the SLR scenarios described in Level 1. Following the recommendation from the scientific community, the 2117 (100-yr SLR) were extracted from the IPCC RCP 8.5 median scenario projections and used in the storm surge model development.

Level 3 analysis involved the dynamically coupled modeling of wind, surge, and waves. The numerical modeling was conducted using the ADCIRC+SWAN model. Numerical simulations were conducted to calculate the significant wave height, water surface elevation, peak wave period, and current velocity associated with a set of synthetics storms that could potentially impact the project area. Level 3 analysis employs a probabilistic framework to encompass the range of possible variations in the storm conditions. The set of reduced storms was extracted from the existing FEMA Flood Insurance Study (FEMA, 2012). Sensitivity testing was conducted to select a reduced set of storms from the FEMA storm suite that reproduce the 50-, 100-, and 500-yr water surface elevation extremal statistics with reasonable accuracy. Consequently, a set of 80 storms was selected that recreated the 50-, 100-, and 500-yr extremal conditions. After simulation of the selected storms, an extremal statistical analysis was performed on the model results to develop input parameters for the environmental conditions and wave forces.

Because the environmental conditions and wave forces depend on a combination of parameters coinciding in time, including water surface elevations, significant wave height, wave period, and current velocity, two separate methods were employed to determine the environmental conditions and wave forces: (Method 1) The maximum water surface elevations and associated significant wave height, peak wave period, and current velocity, and (Method 2) the maximum significant wave height and the associated water surface elevations, peak wave period, and current velocity. The wave results were further analyzed to account for the maximum possible wave height and the nonlinearity of wave crest asymmetry, which occurs with large waves in shallow water.

These wave conditions were used as input to the AASHTO guide specification calculations (AASHTO, 2008) to determine two design case force scenarios for the existing bridge. The design cases evaluated are Design Case I: maximum vertical force and associated horizontal, pile, and slamming force, and Design Case II: maximum horizontal force and associated vertical, slamming, and pile force. These Design Cases were evaluated on the existing bridge for the design condition (100 year SLR and 100-yr return period event). Results indicate the majority of existing bridge is impacted by the design wave crest particularly in the marsh areas. As a result, for the design condition, the existing bridge experience vertical and horizontal superstructure wave-induced forces as high as 9.3 kip/ft and 4.3 kip/ft, respectively; the pile loads were as high as 16.9 kip/pile.

A final comparison between Level 1 and Level 3 analysis was performed on the existing bridge. In general, for the project design criteria, Level 3 analysis results show lower water surface and wave crest elevations everywhere on the bridge when compared to the Level 1 analysis results. In some areas the different between Level 1 and Level 3 crest elevation is as high as 10 ft. Such areas correspond to the highest wave heights observed at the deeper channels. Level 3 surge elevations also showed lower values when compared to the Level 1 results, with Level 3 being 1.5 ft lower than Level 1 fairly uniformly along the bridge span.

Overall, Level 3 analysis results are considered more accurate and more robust. Hence, it is recommended to consider Level 3 results on the design of the I-10 Mobile River Bridge and Bayway Project.

1 Introduction

The primary goal of the Mobile Bay Bridge Storm Surge Impact Analysis is to describe the environmental conditions and forces from tropical storms and hurricanes on the Mobile Bay Bayway Bridge (MBBB) for various levels of risk. To date, this analysis has been based on a Level 1 analysis based on the methods described in AASHTO (2008) and HEC-25 (2008 and 2014), with results were presented in the Level 1 Report (Mott MacDonald, 2016). The Level 3 analysis is described in detail in this report.

1.1 Level 1 Analysis

Computation of the forces on the bridge requires knowledge of the storm surge elevation and corresponding wave conditions for a given risk level (return period). In the Level 1 analysis discussed in the Level 1 Report (Mott MacDonald, 2016), the environmental forcing conditions, primarily wind and storm surge elevation, were based on existing data. In the Level 1 analysis, wave heights at the bridge were computed by modeling a single, simple hurricane wind field across Mobile Bay, which was elevated to a static surge elevation throughout the entire Mobile Bay and Gulf of Mexico basin. The surge level was developed based on historical data fit to an extreme value distribution. This resulted in what we expected to be a conservative scenario. The surge elevation was based on a limited historical record, and was not dynamically linked with the waves. For the Level 1 analysis, the surge and associated waves did not move through the Bay together – the surge was static at a constant high elevation across the Bay and the waves were propagated into the site on that elevated water level. The Level 1 method removed the dynamic coupling of wind, surge, and waves, as well as ignored the possible variations to storm track and storm size. According to AASHTO (2008), the Level 1 results should only be used for planning purposes. The Level 1 analysis results were summarized in the Mobile Bay Bridge Storm Surge Impact Analysis report submitted to ALDOT (Mott MacDonald, 2016).

1.2 Level 3 Analysis

Wave response to hurricanes is a complex process that is affected by the dynamics of the storm including surge, storm track (which influences direction of the winds, the approach speed of the hurricane, wind velocity and fetch), circulation currents, and wave generation and transformation processes on that dynamic field. The Level 3 assessment includes modeling the coupled wind-surge-wave processes to produce the best-possible prediction of surge and waves at the site. In other words, the Level 3 assessment uses the best methods to simulate the physics accurately, where the Level 1 used simplifying assumptions to get to a conservative result.

The challenge with simulating the physics of a coastal storm correctly is that there are many possible variations in the storm conditions that may impact the site. For example, a very slow moving, large Category 1 storm can produce much larger storm surge than a very fast, but smaller Category 4 storm. Therefore, the Level 3 method employs a probabilistic framework to encompass the range of possible variations in the storm conditions. This probabilistic framework, called the JPM-OS (Joint Probability Method with Optimal Sampling), provides a method to quantify the statistics of the input storm parameters and output conditions (waves, forces) that provide an accurate level of risk associated with the resulting forces.

Level 3 analysis involves the modeling of a set of storms using the dynamically coupled wind-surge-wave model. A set of hundreds of unique storm events are developed based on five different fundamental parameters: storm landfall location, central pressure difference, storm radius, forward velocity, and storm heading. Each storm is assigned a unique probability based on empirical data derived from the 5 fundamental parameters near Mobile Bay. Thus, the Level 3 analysis simulates hundreds of storms, whereas the modified Level 1 analysis only evaluated one storm scenario.

After modeling each storm, the resulting surge elevations and significant wave heights are output for each point in the model domain. These surge and significant wave height data points are then fit to a probability distribution. This allows us to develop the surge and significant wave heights as a function of return period. With these results, we can then compute forces on the bridge elements using the storm surge elevation and associated wave conditions for a given return period. This Level 3 analysis provides the greatest reduction in uncertainty and accounts for probability in an objective manner.

1.3 Project Location and Extents

The project site is along Interstate Highway 10 (I-10) located in the northern end of Mobile Bay, and spans the eastern I-10 tunnel exit to bridge landing at Spanish Fort, AL as shown in Figure 1.

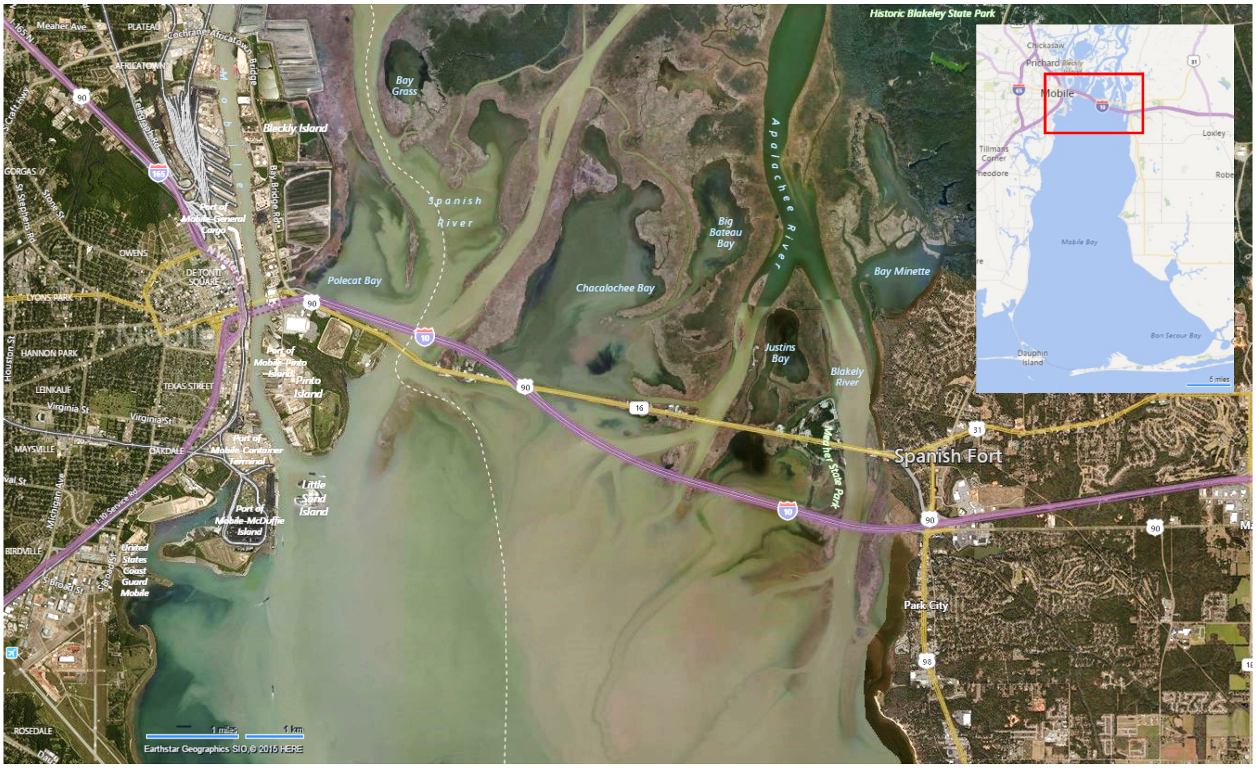


Figure 1. Project vicinity in northern Mobile Bay near Mobile, AL.

1.4 Project Design Criteria

Based on discussions with the project team and ALDOT, the bridge design lifetime is set at 100 years. The analyses presented in this report employs 2017 as the start of service life and 2117 as the end. The return period has been set as 100-yr for the main spans and 50-yr for the ramps (see Table 1).

Table 1: Project Design Criteria

Bridge Section	Return Period	Sea Level
Main Spans	100-yr	100-yr (2117)
Ramps	50-yr	100-yr (2117)

2 Review of Existing Data

For the Level 1 analysis, a comprehensive database of physical data and available knowledge on sea level rise, storm surge, and wave forces on bridge element relevant to Mobile Bay was developed. The data collection effort included compilation of existing and historical data from all available reports, design, and publications from previous studies and designs pertinent to the project area. In addition to the work for the Level 1 analysis, additional bathymetry and topography data was collected along the bridge for the Level 3 analysis.

2.1 Tides and Vertical Datums

Tides at Mobile Bay are mixed semidiurnal in character, with a mean range of 1.45 ft and a highest annual tide (not storm-induced) estimated at +3.86 ft (NOAA, 2013). Tide elevations at the nearest vertically controlled tide gage located in Mobile State Docks (NOAA Tide Gage 8737048) are shown in Table 2 relative to NAVD88.

Table 2: Tide Elevations relative to NAVD88 at Mobile State Docks NOAA Tide Gage 8737048

Water level	Mobile, LA [ft NAVD88]
Mean Higher-High Water (MHHW)	1.15
Mean Sea Level (MSL)	0.34
Mean Lower-Low Water (MLLW)	-0.47
National Geodetic Vertical Datum of 1929 (NGVD29)	-0.085

2.2 Bathymetry and Topography Data

Two sets of bathymetry and topography data were available that have the required resolution at the project site. The first set was the bathymetry of the mesh used in the FEMA study (FEMA, 2011). The other set of data includes new bathymetric and topographic data collected for ALDOT in 2017 (Tremble, 2017). The new data collected is shown in Figure 2.

Figure 2. Bathymetry and Topography data collected for ALDOT (Tremble, 2017)

2.3 FEMA Flood Insurance Study: Florida Panhandle and Alabama Data

For the Level 3 analysis, synthetic tropical storms, water surface elevations, and waves from the FEMA Flood Insurance Study (FIS) (FEMA, 2012) were used as a baseline for the storm surge modeling and analysis. Further details are provided in Section 4.

2.4 Bridge Data

2.4.1 Existing Bridge Geometry

Per ASSHTO (2008) specifications (Figure 3), the existing bridge geometry was extracted and digitized from as-builts drawings provided by ALDOT. Mott MacDonald extracted existing bridge geometry during Level 1 analysis (Mott MacDonald, 2016); the same bridge geometrical data employed in in Level 1 analysis was used in this Level 3 study. The extraction points were taken along the east bound span (south span on plan view) since it is the existing bridge span most exposed to wave action. Mott MacDonald's structural team provided the locations (northing and easting) of each bent of the existing bridge along the proposed bridge PGL. The station data as well as northing and easting are provided in the digital appendix.

Three different alternatives were found on the as-built drawings for the substructure: (A) 24 in square pile, (B) 36 in square pile, and (C) 54 in cylindrical pile. The as-built drawings do not specify which alternative was utilized or at what location. As there is uncertainty on the final construction, Alternative C, which is the most conservative alternative in terms of wave forces, was used in the analysis.

Another source of uncertainty is the vertical control of the bridge. No discussion on vertical control to a vertical datum or benchmark has been found on the as-built drawings. The plan elevations are Proposed to 1971 MSL which is referred to as “200”. It is unclear if this is Proposed to a local benchmark. MSL is not a constant vertical datum and varies over time. Historical records show MSL increased 0.18 ft from 1971 to the modern MSL Proposed at the NOAA gage at the Mobile State Docks, but without a more known vertical control, this in an unreliable adjustment.

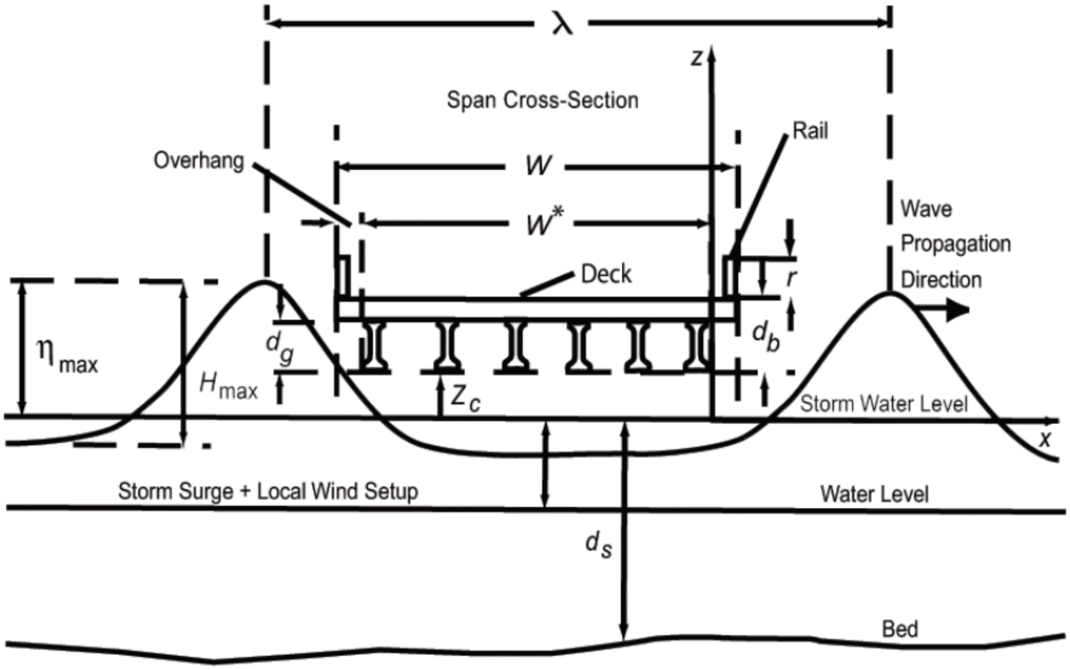


Figure 3. Parameters used in wave-induced force calculations (AASHTO, 2008)

2.4.2 Proposed Bridge Location

Mott MacDonald’s structural team provided the locations (northing and easting) of each bent of the proposed bridge along the pertinent PGL. For reference purposes, the stations along the PGL for each bent of the proposed bridge were also extracted. The results shown on this report are illustrated with respect to the PGL stationing. The station data as well as northing and easting are provided in the digital appendix.

The Proposed bridge consists of 2 mains spans and 12 ramps for a total of 14 bridges (see Figure 4, Figure 5, Figure 6, Figure 7) and 1624 bents listed below. Based on discussions with the bridge team, 54 in cylindrical piles were assumed in the force calculations.

- East Bound, 581 bents
- West Bound, 582 bents
- East Tunnel A, 14 bents
- East Tunnel B, 27 bents
- East Tunnel C, 75 bents
- Eastern Shore A, 52 bents
- Eastern Shore B, 30 bents
- Eastern Shore D, 14 bents
- I-10 Business East Bound (I-10 Bus EB), 71 bents
- I-10 Business West Bound (I-10 Bus WB), 24 bents
- Midbay A, 30 bents
- Midbay B, 46 bents
- Midbay C, 37 bents
- Midbay D, 41 bents

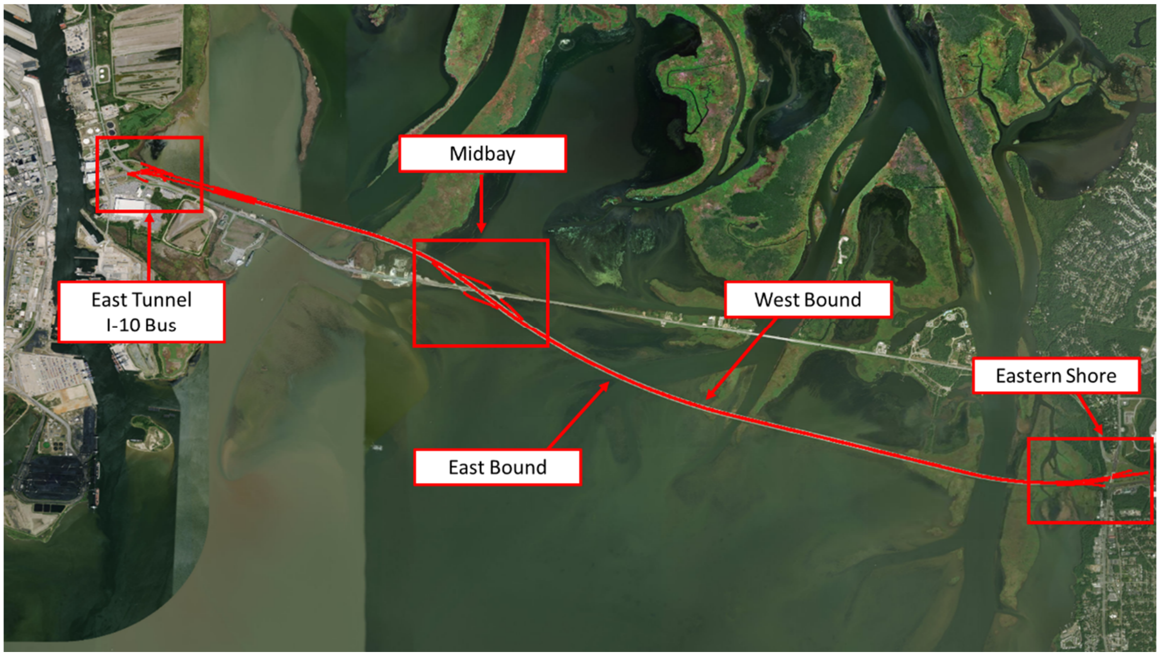


Figure 4. Overall Proposed Bridge

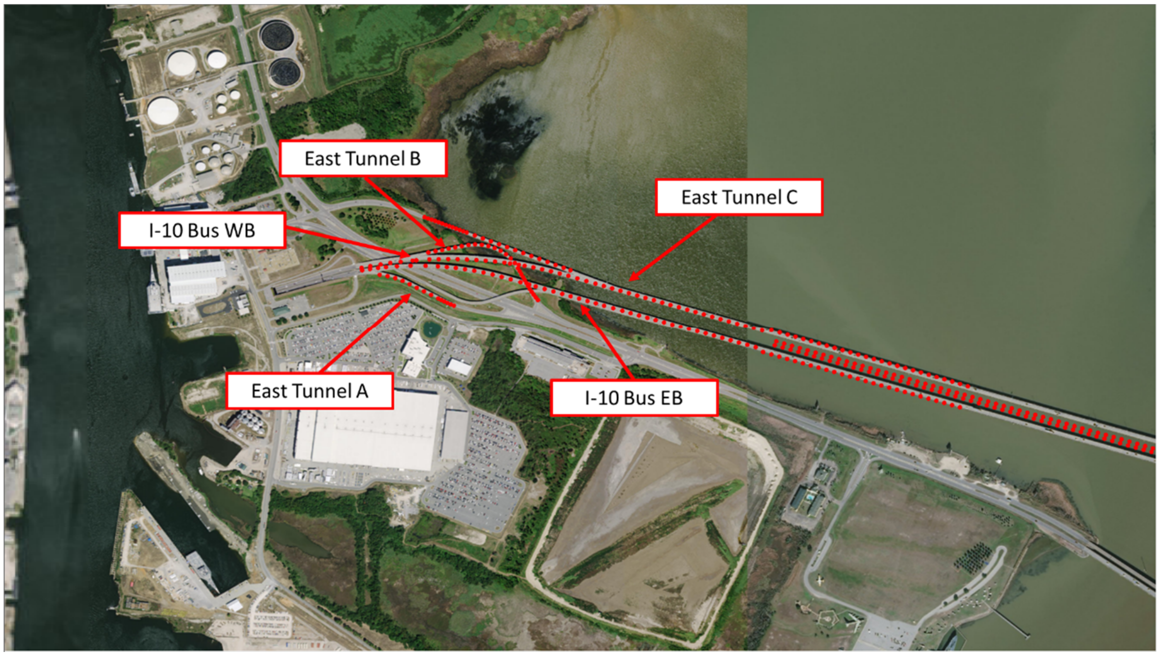


Figure 5. Proposed Bridge: East Tunnel and I-10 Business ramps

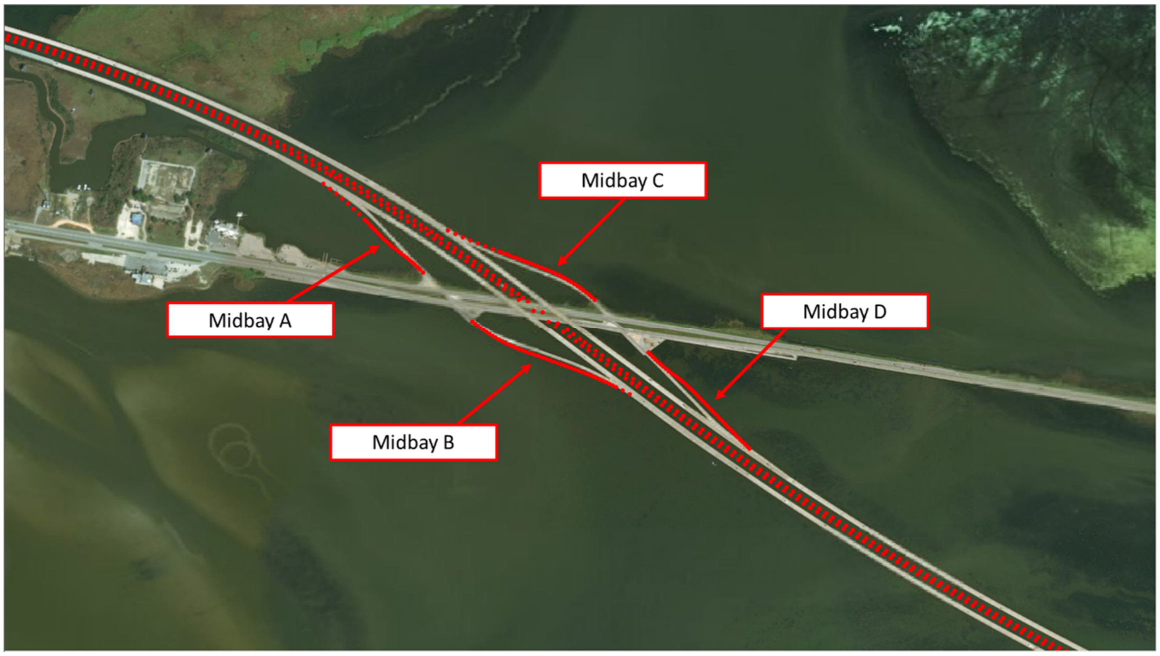


Figure 6. Proposed Bridge: Midbay Ramps

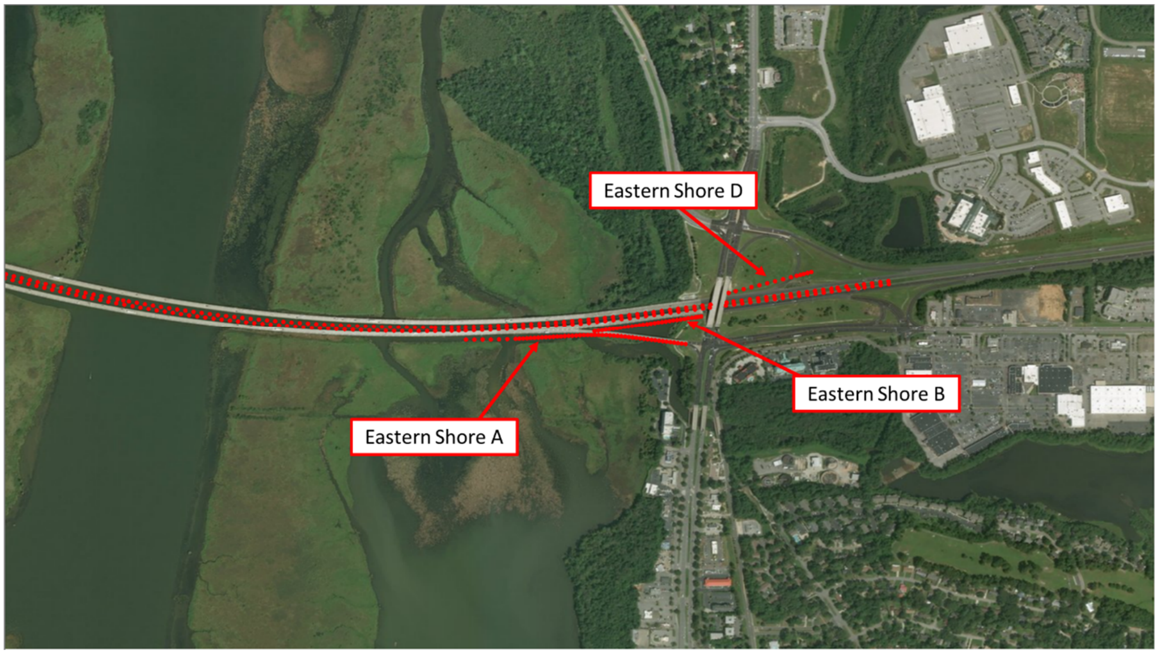


Figure 7. Proposed Bridge: Eastern Shore Ramps

3 Climate Change Considerations

Sea level has been observed to be rising across the globe and at an increasing rate in the Gulf of Mexico. Sea level rise (SLR) can increase the risk of coastal infrastructure damaged; it can enhance the surge resulting from hurricanes and thereby increase the hydrodynamic forces exerted by the water and waves on coastal bridges. The following section provides information on the SLR projections used in this Level 3 analysis.

3.1 Relative Sea Level Rise

The Level 3 analysis utilizes SLR projections set forth by the Intergovernmental Panel on Climate Change (IPCC). The IPCC was set up in 1988 by the World Meteorological Organization (WMO) and United Nations Environment Program (UNEP), and is endorsed by the United Nations General Assembly. The IPCC is the world body for assessing the science related to climate change to provide policymakers with regular assessments of the scientific basis of climate change, its impacts, and future risks. The IPCC assesses thousands of scientific papers published each year and identifies where there is agreement in the scientific community, where there are differences of opinion, and where further research is needed. The IPCC completed the Fifth Assessment Report in November 2014 with the participation of more than 830 scientists from over 80 countries evaluating more than 30,000 scientific papers (ASBPA, 2015).

During the Level 1 analysis, Mott MacDonald conducted a literature review and analysis of different SLR projections including NOAA, USACE, IPCC, and others. The majority of the SLR projections fall in the range bounded by the NOAA low/USACE low in the lower bound and the IPCC RCP8.5 in the upper bound. These projections are in good agreement with the more likely SLR scenario (Mott MacDonald, 2016).

From the scientific community, the American Shore and Beach Preservation Association (ASBPA) suggests citing the IPCC values as the current most credible sea level rise projections for 2100, from a minimum of 0.9 ft for the lower limit of the most benign scenario, to 3.2 ft for the upper limit of the worst scenario (ASBPA, 2015). In addition, the North Carolina Coastal Resources Commission Science Panel (2015) has recommended using the most recent report of the IPCC to provide scenario-based global sea level rise projections; “the scenarios chosen to model sea level rise over the next 30 years are the IPCC’s low greenhouse gas emissions scenario (RCP 2.6) and the high greenhouse gas emissions scenario (RCP 8.5), as all other [IPCC] scenario projections fall within the range of these two.”

Consequently, this analysis recommends and employes the IPCC RCP 8.5 median scenario projection for analyzing the impact of sea level rise in the project area. The values of SLR used in the Level 3 analysis are provided in Table 3.

Table 3: Recommended SLR projection based on IPCC RCP 8.5 median scenario

Year	Recommended SLR projection [ft]
2017 (0-yr SLR)	0.00
2067 (50-yr SLR)	1.22
2117 (100-yrSLR)	3.04

4 Storms in Mobile Bay

The quantification of extreme water surface elevations and wave heights is a central component in designing and/or assessing a coastal bridge. Methods typically employed to calculate extremal wave and water surface elevation statistics include design storm analysis, historical gauge analysis, and joint probability analysis. The following Sections provide an overview of the JPM-OS approach and the selected storms used in this study for assessing environmental conditions and wave loading inputs.

4.1 Method Description

Level 3 analysis employs a probabilistic framework to encompass the range of possible variations in the storm conditions. Such method is called JPM-OS and was employed in this study. The JPM approach describes a storm in terms of that have the greatest influence on storm surge: (1) storm landfall location, (2) central pressure difference, (3) storm radius, (4) forward velocity, and (5) storm heading, as shown in Figure 8. Statistical distributions are calculated for each storm parameter based on historic data near the area of interest. The original work by (Ho, 1975) divided each storm parameter distribution into distinct segments, then proposed simulation of all possible combinations of storm parameters. The modern implementation of this approach performs Optimal Sampling (JPM-OS) to select a smaller suite of representative storms (Resio, 2007). Based on the joint probability distributions developed for the storm parameters, occurrence probabilities are assigned to each storm in the synthetic storm suite. The selected storms are then simulated using a coupled wave and surge numerical model.

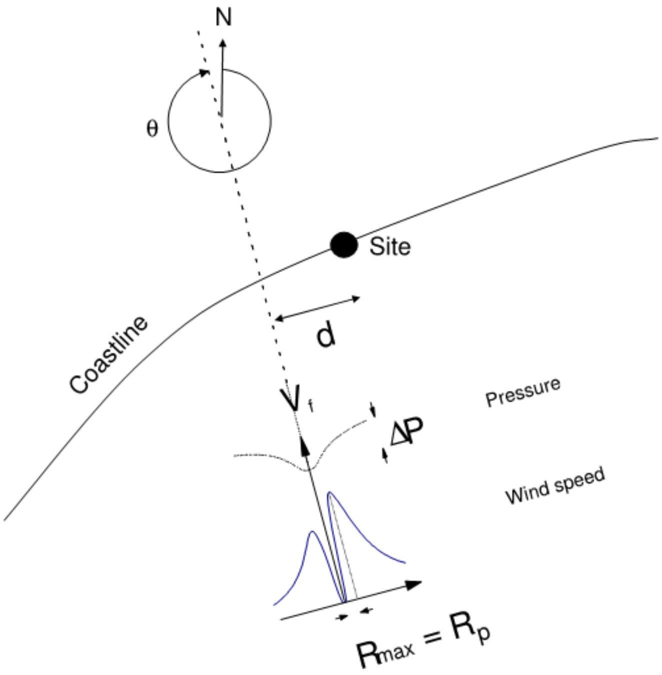


Figure 8. Storm Characteristics as it approaches the coast Source: (Lettis Consultants International, 2012)

4.2 Storm Selection

This study employs the 295 synthetic storms used in the FEMA FIS (Lettis Consultants International, 2012) which consists of synthetic tracks that varied landfall positions from New Orleans, LA on the west to the Florida panhandle. The analysis conducted in this study focused on the 50-, 100-, and 500-yr return period statistics at the project site. Therefore, the set of 295 storms (Figure 9) was reduced to 80 storms (Figure 10) by filtering out the synthetics storms that would cause minimal surge at the project site; only the storms that had an impact of the high return period events were evaluated in this study.

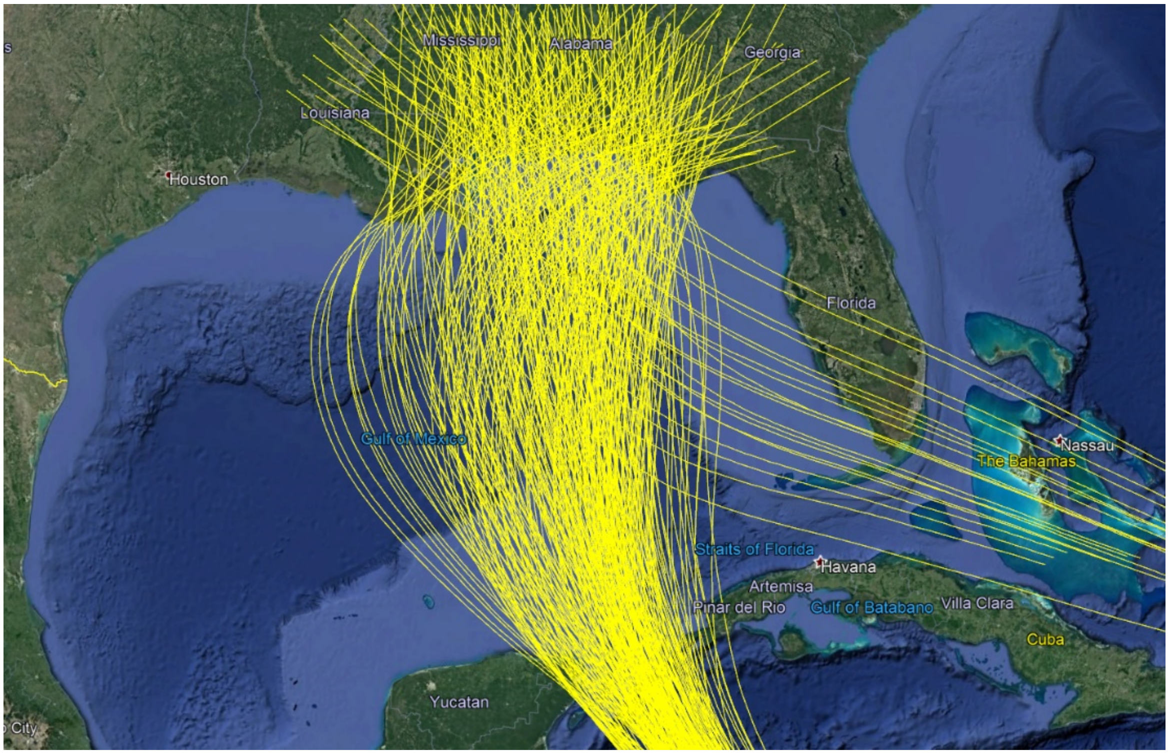


Figure 9. Full set of 295 synthetic storm from FEMA (2013b)

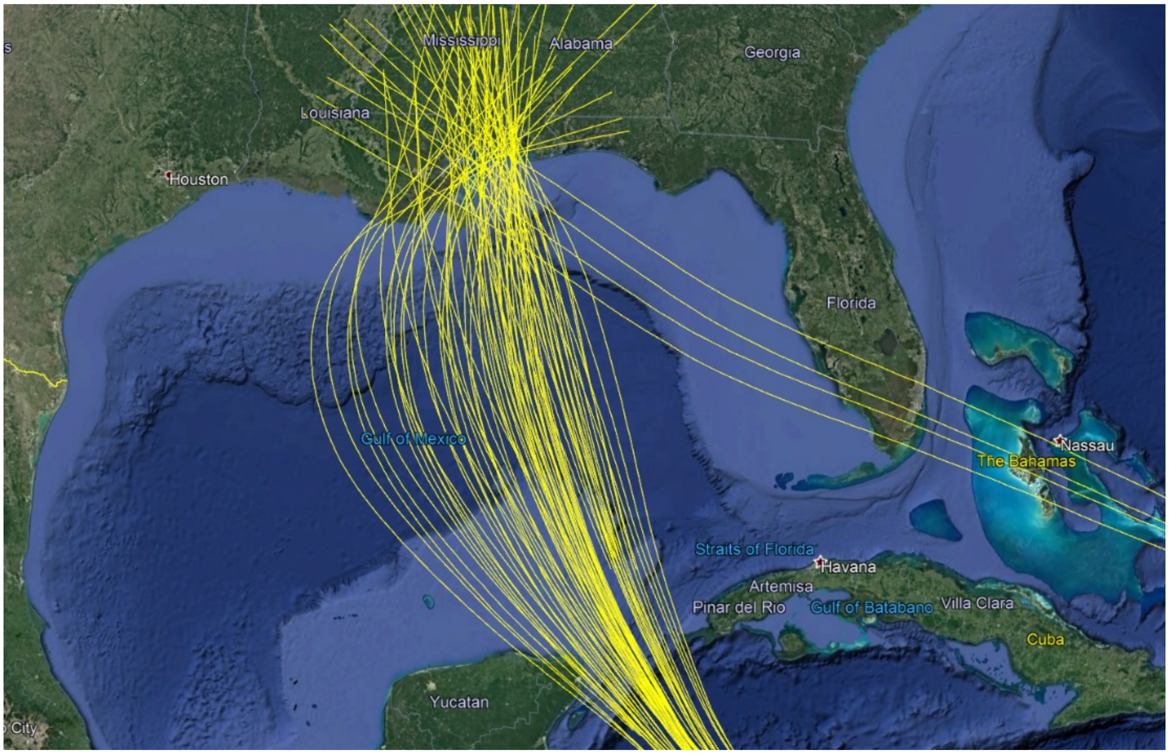


Figure 10. Reduced set of 80 synthetic storms from FEMA (2013b)

Sensitivity testing was conducted to select a reduced set of storms that reproduce the 50, 100, and 500-yr statistics with reasonable accuracy. The sensitivity testing was conducted along the project site by setting an extreme maximum water level filter elevation where all storms causing maximum elevations below the corresponding filter level were removed from the extremal analysis.

A reduced set of 80 storms was selected, which resulted in -0.07 ft, -0.01 ft, and 0 ft maximum change in water surface elevation (WSE) for the 50-, 100- and 500-yr return period when compared to the full suite of storms. Such values were deemed appropriate to serve as input in the modeling and statistical analysis. A summary of the sensitivity testing is shown in Table 4.

Table 4: Summary of selected sensitivity testing case. Columns 3 to 5 represent the maximum change in extremal WSE for a given filter when compared the reduced storm set to the full storm suite

Number of Storms	Filter Elevation [ft]	Max Change in 50yr WSE [ft]	Max Change in 100yr WSE [ft]	Max Change in 500yr WSE [ft]
120	3.0	-0.02	0.00	0.00
94	4.0	-0.04	-0.01	0.00
80	**4.5**	**-0.07**	**-0.01**	**0.00**
68	5.0	-0.14	-0.03	0.00
56	5.5	-0.20	-0.05	0.00
43	6	-0.46	-0.11	0.00

5 Storm Surge Model

Level 3 analysis involved the dynamically coupled modeling of wind, surge, and waves. Numerical simulations were conducted to calculate the wave height, water surface elevation, peak period, and velocity associated with a set of synthetics storms that could potentially impact the project area. The following Sections elaborate on the development of the Storm Surge Model used in this study.

5.1 Model Description

The Storm Surge modeling was conducted using the ADCIRC+SWAN model. ADCIRC (Luettich & Westerink, 2015) is a 2D circulation modeling tool based on shallow water equations that can be used to simulate water level fluctuations and current velocities forced by tides, winds, river flows, hurricanes and other natural forcing. ADCIRC uses unstructured triangular meshes under the finite element approach. The ADCIRC+SWAN version of the model features fully dynamic coupling with the unstructured SWAN (Delft, 2012) models for use in hurricane simulations (Dietrich, et al., 2011).

The computational mesh employed in this study was a combination of the mesh developed for the FEMA Florida Panhandle Flood Insurance Study (FEMA, 2013a) and the mesh developed for the Level 1 analysis (Mott MacDonald, 2016). The final mesh had approximately 480,000 nodes and had a resolution ranging from 25 meters to 50 meters around the project location. The bathymetry and topography for the mesh was a combination of the FEMA bathymetry and the newly collected bathymetry discussed in Section 2.2. The mesh with the combined bathymetry is shown in Figure 11.

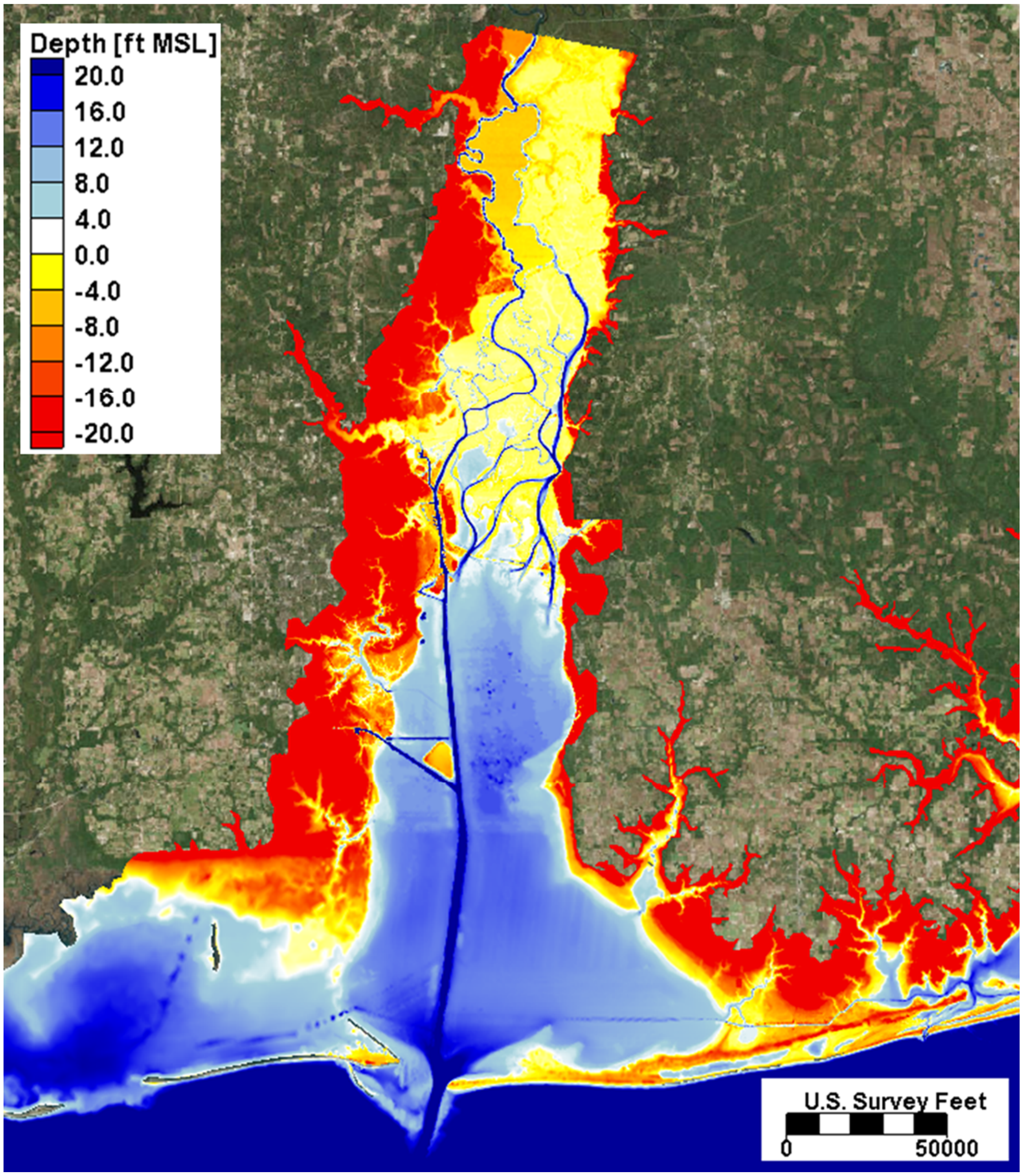


Figure 11. Computational Mesh with Bathymetry for Production Runs.

5.2 Model Input Parameters

5.2.1 Winds

The wind fields used in the modeling were provide by FEMA from the Florida Panhandle FIS study (FEMA, 2013a). The wind fields are synthetic storms developed from a statistical representation of historical storms that could or have occurred in the project area. A reduced set of 80 storms (see Section 4.2). The wind fields consisted of the 30-minute averaged winds for the synthetic storms. A wind multiplier of 1.09, consistent with the FEMA study, was used in the modeling to covert the 30-minute winds to a 10-minute winds to be used in ADCIRC.

5.2.2 Nodal Attributes

Several input parameters have been calculated for initial calibration testing. The roughness coefficient influences the storm surge due to its impact with wind speed and inundation; to accurately estimate the roughness, land cover data was downloaded from the national land cover database (NOAA, 2006). Land cover data is the result of converting raw satellite data into different land categories. Based on these different types of land, an associated Manning's coefficient was applied to different areas of the mesh (Luettich & Westerink, 2015).

5.3 Model Testing

The FEMA FIS study (FEMA, 2013a) used a loosely coupled wave method that calculated the waves separately from the surge by using an iterative process described in the Wave Validation Report (Slinn, 2012); in addition, the FEMA FIS also employed older versions of ADCIRC and SWAN. The analysis presented in this report was conducted using a tightly coupled version of the ADCIRC+SWAN model (Dietrich, et al., 2011), which dynamically couples the interaction of wind, surge, and waves.

To verify the accuracy of the storm surge model results, the FEMA FIS model input conditions were used to recreate the selected storms from the Florida Panhandle study (FEMA, 2013a). The maximum elevations from the ADCIRC+SWAN model results were compared with the FEMA FIS maximum elevations for selected storms (storm 231 and storm 199); results shown on Figure 12. Storm 231 represents the best agreement to the FEMA FIS values (FEMA, 2013a) showing a difference of 0.1 ft at the project site. Storm 199 represents the least agreement resulting in WSE 0.5 ft than FEMA FIS values (FEMA, 2013a). Both storms illustrate a slight water surface elevation overprediction west of Mobile Bay; however, this overprediction does not affect the surge at the project site.

Overall, when looking at the reduced set of 80 storms, ADCIRC+SWAN produces slightly higher water surface elevations at the bridge compared to the FEMA FIS values (FEMA, 2013a) by an average of 0.16 ft. ADCIRC+SWAN is expected to produce more refined results at the project site since it employs a more refined mesh and dynamic coupling between the waves and surge; therefore, the average variation of 0.16 ft is deemed appropriate for this study.

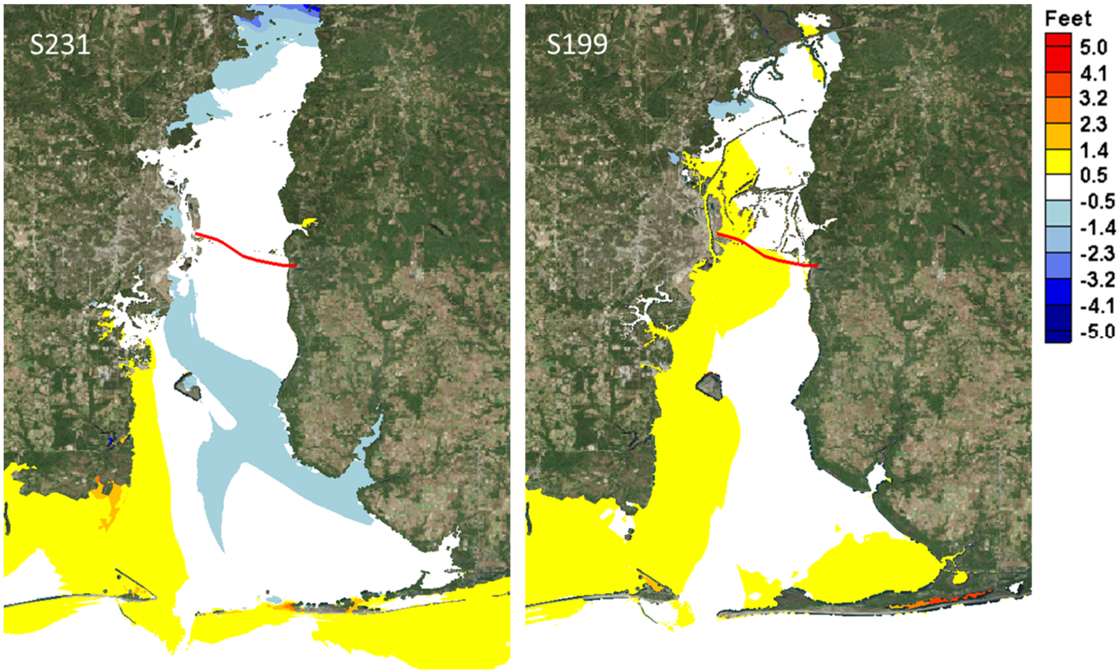


Figure 12: Difference of Mott MacDonald Max WSE and FEMA Max WSE (ft). Blue areas are where Mott MacDonald computes lower values than the FEMA WSE; red areas are where Mott MacDonald computes higher WSE.

6 Extreme Value Statistical Analysis

Following the numerical simulation of the storms, an extreme value statistical analysis was performed on the model results to develop the environmental conditions at the project site as well as input parameters for wave force calculations. The following Sections elaborate on the methods and results associated with the recurrence interval analysis, discusses the incorporation of uncertainty terms into the JPM-OS results, and compares the proposed JPM-OS approach for this study to previous results presented in the FEMA study (FEMA, 2013b).

6.1 Method Description

Once the JPM-OS suite of storms are simulated as described in Section 5, a statistical analysis is performed to determine the Annual Exceedance Probability (AEP) at each point of interest. The AEP at each analysis point represents the probability of a storm exceeding a given water surface elevation in one year. In this report, AEP values are reported as return periods e.g., the 50-yr event represent the 2% AEP. The return period calculations performed in this study were conducted using Version 1.1 of the SURGE_STAT program obtained from FEMA (FEMA, 2012). The program uses the following inputs to perform the extremal analysis:

- Numerical model results at each output point
- Recurrence rates for each storm
- Uncertainty parameters
- AEP values to extract from the results

A summary of the calculation performed by the SURGE_STAT program is described below. See FEMA (2013b) and FEMA (2012) for further discussion of the SURGE_STAT program methodology.

1. For each extraction point in the analysis, a water surface elevation histogram is developed using bins of width 0.1 ft., ranging from 0 to 36.09 ft.
2. The recurrence rate for each storm is summed into the appropriate bin.
3. The results of the histogram are then spread into adjacent bins to account for the uncertainty due to secondary parameters. See Section 6.2 for further discussion of the uncertainty terms.
4. The modified histogram is then summed from the highest bin to the lowest bin to form a cumulative distribution of water surface elevation.
5. The water surface elevation for the user specified AEP is then interpolated from the cumulative water surface of elevation distribution curve.

The SURGE_STAT program was used to calculate the extremal water surface and extremal significant wave heights. However, quantification of uncertainty terms for significant wave heights has not been conducted in previous JPM-OS analyses. Hence, uncertainty terms associated with significant wave heights were not included in this analysis.

6.2 Uncertainty Discussion

The uncertainty in JPM-OS analysis in the water surface elevation is quantified by various error terms, which describe error due to “secondary factors”. The following error terms are used to quantify secondary quantities which are not implicitly included in model simulations:

- *Astronomical tide level:* random phasing between storm surge and astronomical tide. This parameter is constant for a given project site.
- *Numerical model Error:* difference between modeled and measured storm results. This parameter is constant for a given project site.
- *Idealized wind field Error:* uncertainty describing idealized wind fields. This parameter is constant for a given project site.
- *Error due to measurements*: high water mark measurements error. This parameter is constant for a given project site.
- *Holland B Parameter:* uncertainty due to the Holland B term (Resio, 2007). This parameter is scaled by the surge elevation.

The standard deviation associated with each error parameter is shown in Table 5 as described in FEMA (2013b).

Table 5: Uncertainty parameter standard deviation

Uncertainty Parameter	Standard Deviation	Value [ft]
Astronomical Tide	$\sigma_{\varepsilon 1}$	0.637
Holland B	$\sigma_{\varepsilon 2}$	0.15*surge elevation
Model Simulation[1]	$\sigma_{\varepsilon 3}$	1.730
Idealized Wind Field	$\sigma_{\varepsilon 4}$	1.180

Notes: [1] Model simulation uncertainty is calculated as the square root of the sum of the squares of model uncertainty (1.83 ft) and High-Water Mark (HWM) measurement uncertainty (0.60 ft).

Two separate uncertainty calculations are used as inputs into the SURGE_STAT program (FEMA, 2012). The constant uncertainty term is comprised of the uncertainty due to astronomical tide (σ_{ε}1), model simulation (σ_{ε}3), and idealized wind fields ($\sigma_{\varepsilon 4}$). The uncertainty associated with the Holland B parameter ($\sigma_{\varepsilon \rho}$) scales with water surface elevation. The calculation of each value is shown below in Equation 1 and Equation 2.

$$\boldsymbol{\sigma_{constant}} = \sqrt{\boldsymbol{\sigma_{\varepsilon 1}^2 + \sigma_{\varepsilon 3}^2 + \sigma_{\varepsilon 4}^2}} = \mathbf{2.19}\,\boldsymbol{ft} \qquad \textbf{Eq. (1)}$$

$$\boldsymbol{\sigma_{scaled}} = \mathbf{0.15} * \boldsymbol{WSE}(\,\mathbf{0.15}\,\boldsymbol{WSE}\,) \qquad \textbf{Eq. (2)}$$

6.3 Extreme Conditions Comparison: Mott MacDonald vs FEMA

FEMA FIS (FEMA, 2013b) computed extreme water surface elevations and significant wave height conditions in the project vicinity. As discussed in Section 5, Mott MacDonald built upon the FEMA model to provide more accurate physics and to compute future conditions at the bridge. A comparative analysis between Mott MacDonald model results for the present-day conditions and FEMA FIS results (FEMA, 2013b) was conducted to confirm the present Mott MacDonald study produces reasonable results.

As described in Section 4, a reduced set of 80 storms was selected to develop the extremal statistics presented in this study. The set of 80 storms were simulated using the ADCIRC+SWAN model described in Section 5. Then, extremal statistics were developed at every node in the computational mesh for both water surface elevations and wave heights separately. The extremal statistics were developed with the SURGE_STAT program, using the methodology described in Section 6.1. Figure 13 and Figure 14 show the difference between the FEMA and Mott MacDonald extremal water surface elevations (WSE) and significant wave heights (Hs), respectively.

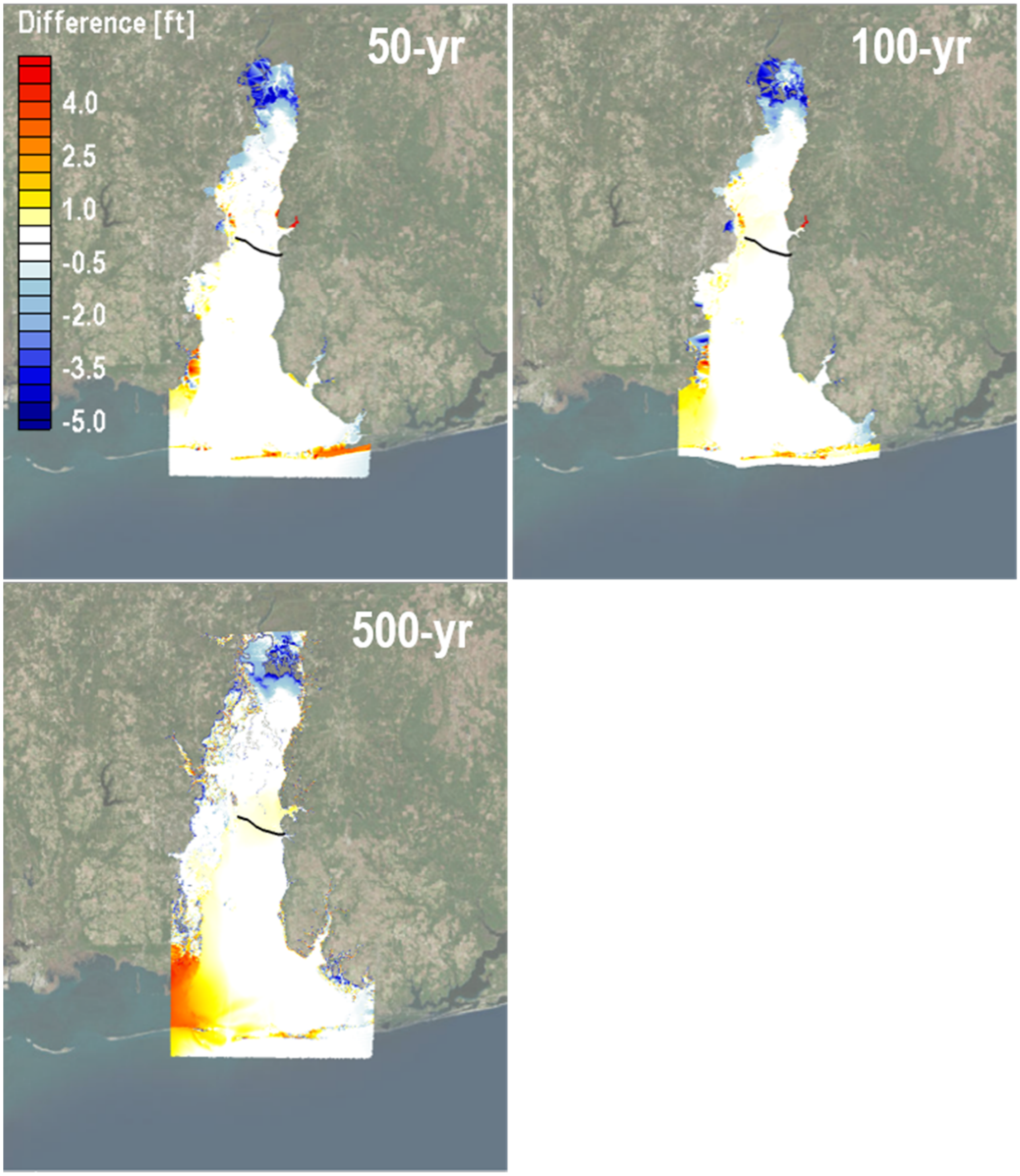


Figure 13. Difference of FEMA Extremal WSE and MM Extremal WSE (ft) for the 50-yr (top-left), 100-yr (top-right), and 500-yr (bottom-left) water levels. Blues are where MM computes smaller WSE than FEMA, reds are where MM computes larger WSE

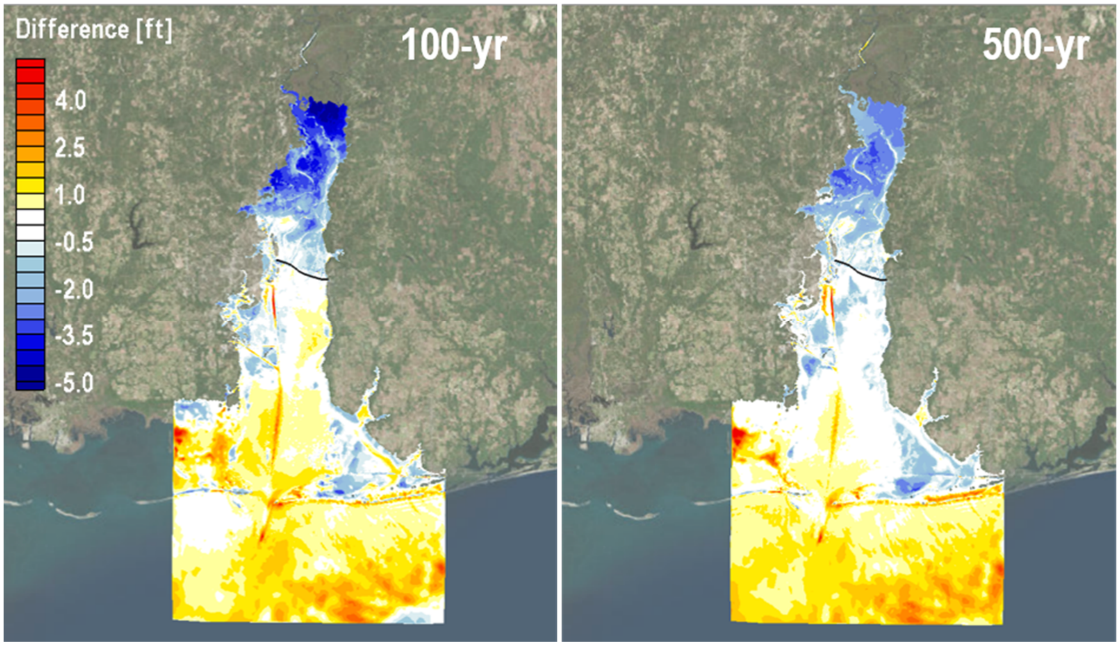


Figure 14. Difference of FEMA Extremal Hs and MM Extremal Hs (ft) for the 100-yr (left) and 500-yr (right) year levels. Blue areas are where MM computes smaller Hs than FEMA and yellow to red areas are where MM computes larger Hs. Note that FEMA 50-yr results were not available.

Differences between the FEMA (FEMA, 2013b) and Mott MacDonald extremal water surface elevations along the Mobile Bay Bridge range from approximately 0.6 feet for the 50-yr condition to approximately 0.8 feet for the 500-yr condition, results shown on Table 6.

Table 6: Comparison of FEMA and MM extremal WSE at project site

T_r [yrs]	FEMA [ft MSL]	MM [ft MSL]	Difference [ft] (MM-FEMA)
50	9.15	9.72	0.57
100	10.86	11.51	0.65
500	15.26	16.11	0.84

The extremal WSE values developed by Mott MacDonald are slightly higher than those developed by FEMA (FEMA, 2013b). The discrepancies can be associated to coupling of ADCIRC+SWAN and to a new bathymetric data set at the project site. First, the Mott MacDonald model simulations were conducted using a coupled ADCIRC+SWAN model whereas the FEMA results used a loosely coupled ADCIRC and SWAN model. This is expected to result in WSE differences due to the differences in the handling of the radiation stresses in the different models. Second, the Mott MacDonald grid utilized updated bathymetric survey data in the immediate vicinity of the bridge which will modify surge and wave results.

The differences between the FEMA and Mott MacDonald extremal significant wave heights were also examined along the Mobile Bay Bridge alignment. Differences in significant wave height varied across the bridge for both the 100- and 500-yr conditions. In general, most differences are approximately 0.2 ft to 0.5 ft, with some areas showing differences of up to 1.0 ft. Overall, the extremal significant wave heights values developed by Mott MacDonald are slightly lower than those developed by FEMA. Table 7 shows the difference in extremal significant wave height at the project site.

Table 7. Comparison of FEMA and Mott MacDonald extremal Hs at extraction point 5.

Tr [yrs]	FEMA [ft]	MM [ft]	Difference [ft] (MM-FEMA)
50	N/A	2.99	N/A
100	4.34	3.88	-0.46
500	6.67	6.15	-0.52

6.4 Extreme Value Statistics

Water surface elevation, significant wave height, peak wave period, and current velocity are not independent values. Environmental conditions and wave forces depend on the combination of such parameters; hence, two different methods were evaluated at each extraction point: (1) maximum WSE and associated significant wave height, period, and velocity, and (2) maximum significant wave height and the associated WSE, period, and velocity. These two separate methodologies were employed since the maximum significant wave height and WSE do not necessarily occur at the same time. The extraction points consisted of the bent locations of the existing and proposed bridges as described in Section 2.4. A total of 2,223 extraction points were evaluated.

The Level 3 methodology to determine the extremal met-ocean conditions consisted of extracting the maximum water surface elevation (or significant wave height) at each extraction point from model simulation results. Then the associated significant wave height (or water surface elevation), velocity, and peak period were extracted at the timestep where the maximum water surface elevation (or significant wave height) occurs. SURGE_STAT program (see Section 6.1) was used to calculate the extreme value statistics and a linear regression was performed on each associated parameter to calculate its corresponding extreme value.

Overall, the two statistical methods of analysis which served as input for environmental conditions and wave-induced forces go as follows:

- **Method 1 - Extremal Water Surface Elevations**: corresponds to the extremal water surface elevation and associated peak period, velocity, and significant wave height.
- **Method 2 - Extremal Significant Wave Heights:** corresponds to the extremal significant wave height, and associated peak period, velocity, and water surface elevation.

Figure 15 and Figure 16 show the 50-,100-, and 500-yr water surface elevations and significant wave heights, respectively.

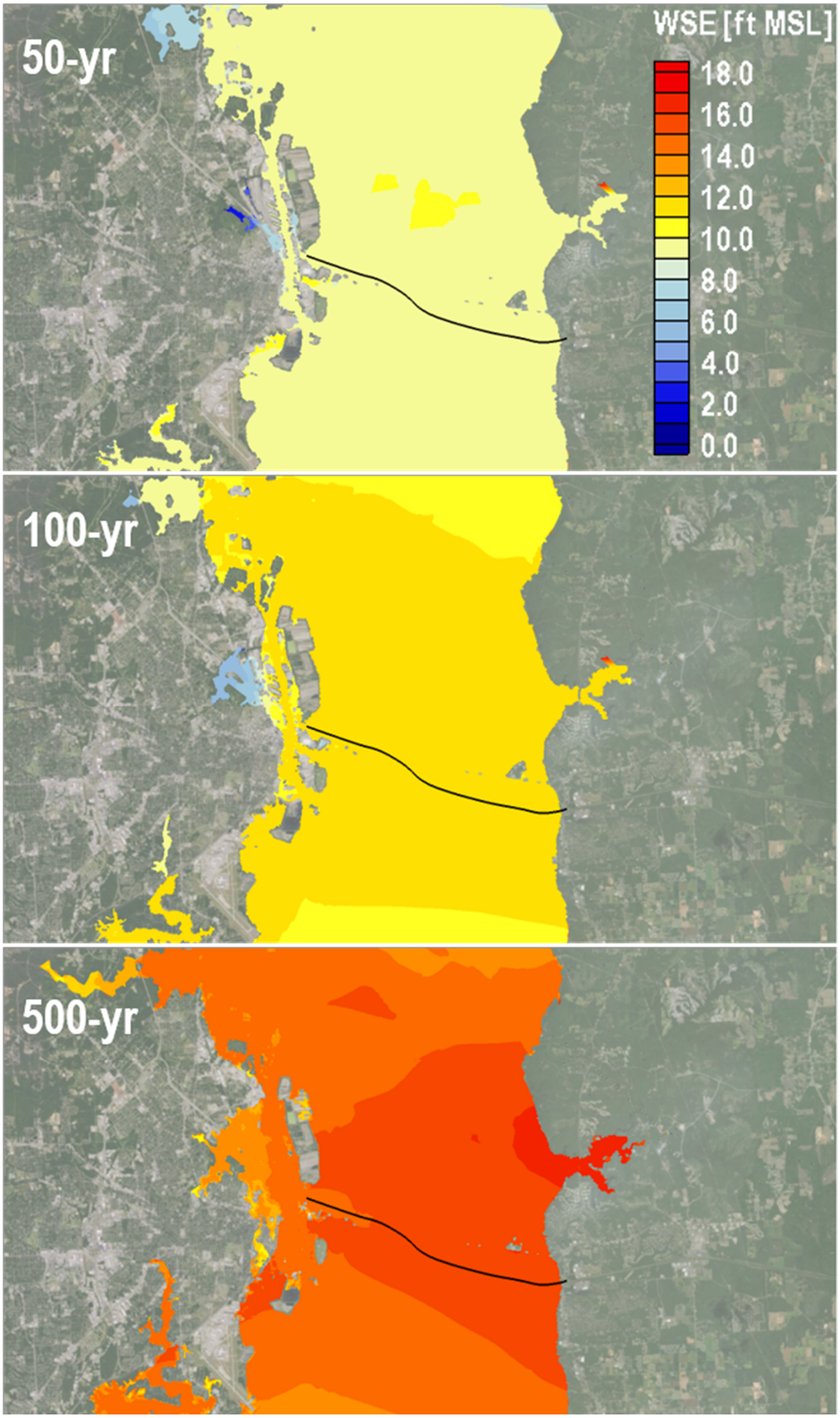
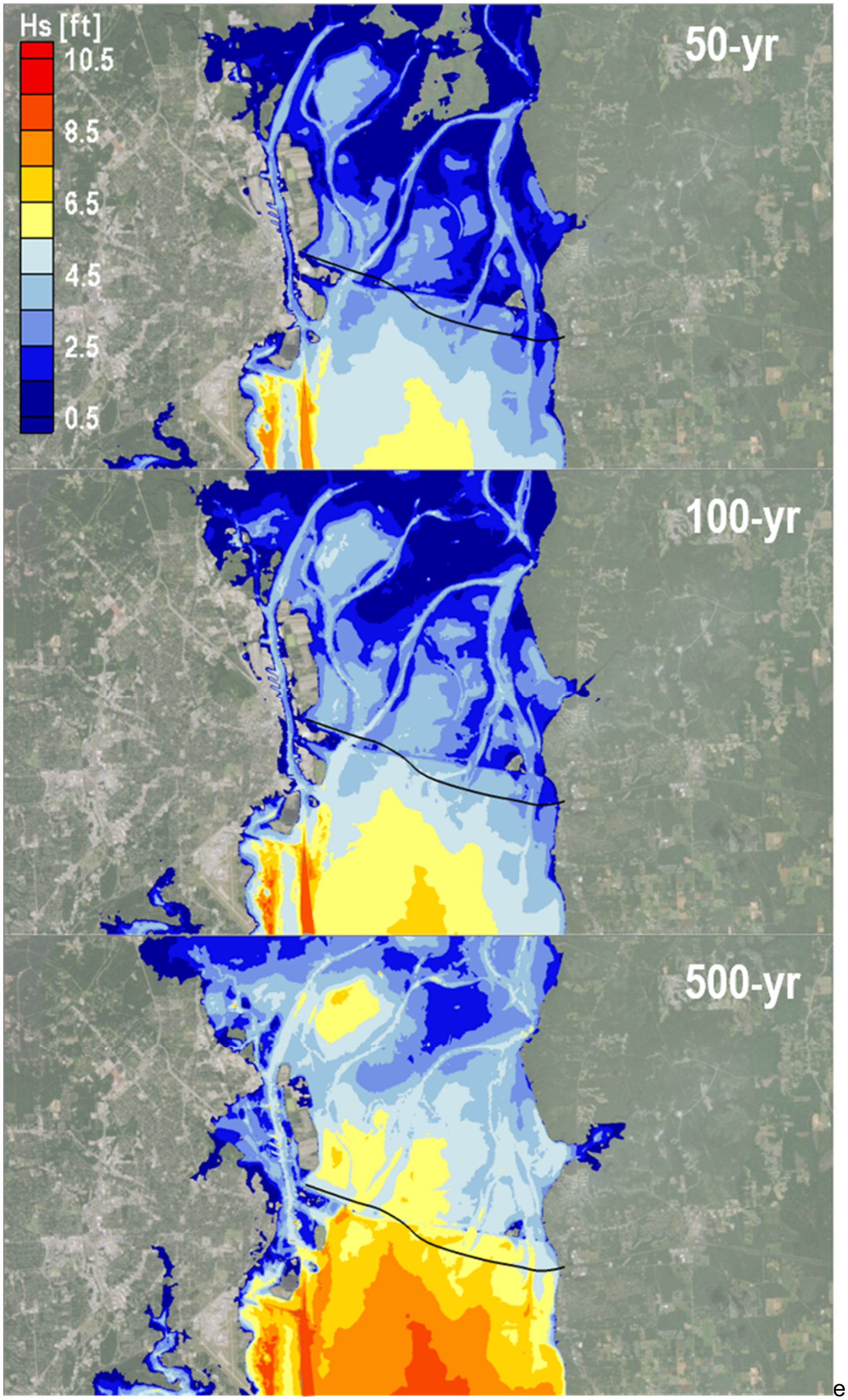


Figure 15. Extremal WSE calculated using MM storm suite for 2017 SLR. Proposed MBB alignment designated by black line.



e

Figure 16. Extremal Hs calculated using MM storm suite for 2017 SLR. Proposed MBB alignment designated by black line.

6.5 Wave Adjustment

The environmental conditions and wave-induced forces were calculated based on the largest possible nonlinear wave at each extraction point. Thus, the extreme value results from Section 6.4 were adjusted to account for the maximum possible wave and the nonlinear wave height.

The maximum possible wave height (H_{max}) that could impact the structure at each bent was evaluated by taking the minimum between the breaking wave and the maximum wave in the spectrum. The maximum possible wave height was then adjusted to account for nonlinear effects.

Environmental conditions and wave-induce forces on bridge elements require wave amplitude and crest elevation in addition to wave heights. Wave characteristics in shallow water (relative to wavelength) are not well represented by simple linear wave theory. In general, Mobile Bay Bridge is considered to be in shallow water relative to the wave conditions; therefore, the waves at the project site are nonlinear. Nonlinear waves differ from linear waves because their profiles do not hold a sinusoidal shape as linear waves do. As seen in Figure 17, the crests of nonlinear waves are more narrowly peaked than those of linear waves, and the troughs of nonlinear waves are typically wider and shallower than those of linear waves (Varma, 2014).

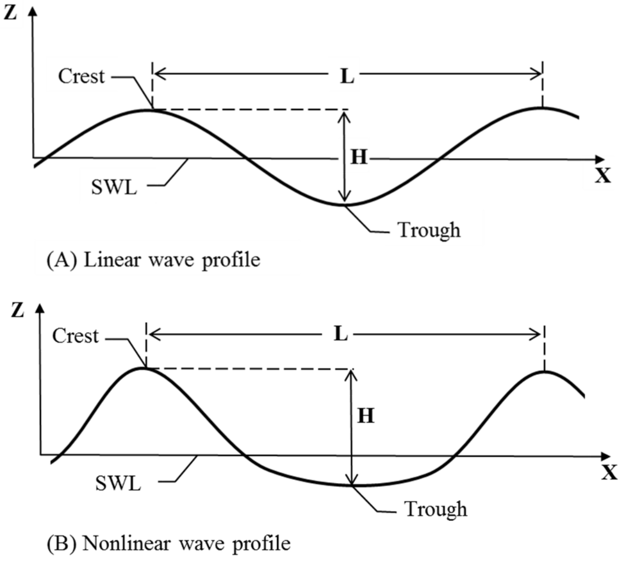


Figure 17. Linear (A) and nonlinear (B) wave profiles, where L is the wavelength and H is the wave height.

Therefore, to account for nonlinear wave characteristics in the force calculations, more sophisticated methods are required to correctly represent the wave profile and kinematics. This Level 3 analysis utilizes the Stream Function Wave Theory to calculate the nonlinear wave characteristics of the waves outputted by the model simulations. The Stream Function code developed by Chaplin, J.R. (1999) was used in this analysis. The resulting wave amplitude were used in the environmental conditions, and the resulting wave kinematics were used in the existing bridge wave-induced force calculations.

7 Environmental Conditions

To investigate the vulnerability of the existing and proposed bridges to potential effects of both sea level rise and storm surge, environmental conditions were calculated using the results from Section 6. The environmental conditions consisted of the maximum wave crest elevation and the associated surge. The results presented in this Section correspond to the results associated with existing bridge and the project design criteria described in Section 1.4 –100 year SLR with 100-yr event for main span and 50-yr event on ramps. Even though only the design criteria results are presented in this report and digital appendix, this analysis included evaluation of the following scenarios for both existing and proposed bridges:

- 0-yr SLR (2017) + 50-yr storm
- 0-yr SLR (2017) + 100-yr storm
- 0-yr SLR (2017) + 500-yr storm
- 50-yr SLR (2067) + 50-yr storm
- 50-yr SLR (2067) + 100-yr storm
- 50-yr SLR (2067) + 500-yr storm
- 100-yr SLR (2117) + 50-yr storm
- 100-yr SLR (2117) + 100-yr storm
- 100-yr SLR (2117) + 500-yr storm

The environmental conditions were computed following the results from the two methods described in Section 6: (1) Extremal Water Surface Elevations and (2) Extremal Significant Wave Heights. The longitudinal wave profile along each bridge was calculated based on the maximum wave crest elevation and its associated surge that could impact the bridge at each bent. The environmental conditions were evaluated by taking the maximum wave crest elevation between Method 1 (extreme water surface elevation) and Method 2 (extreme significant wave height). The environmental conditions reported in this document and digital appendix correspond to the maximum result between the two methods.

The Level 3 analysis results indicate the existing bridge is being impacted by the wave crest under the design condition on the majority of the bridge expect at the overpasses. The existing bridge is not being impacted by the storm surge itself since no bent on the bridge appears to be submerged. Results for the existing bridge are shown on Figure 18.

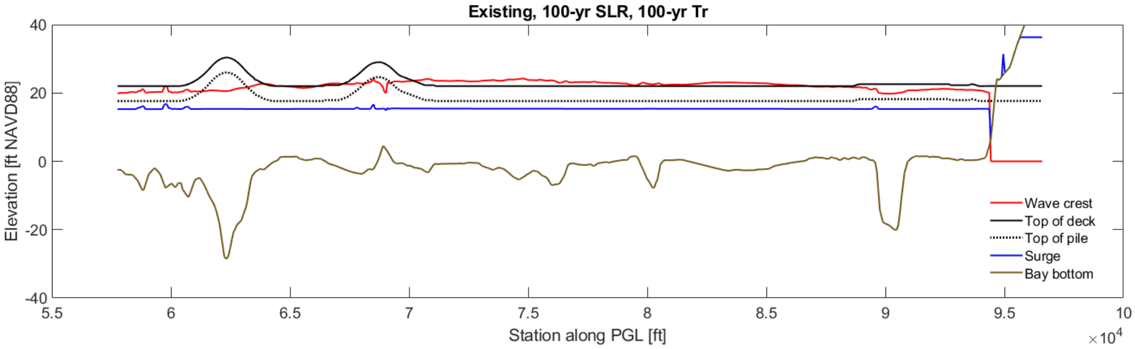


Figure 18. Elevations of the existing bridge top of deck, top of pile, bay bottom, 100-yr surge level, and 100-yr wave crest elevation

8 Forces on Existing Bridge

The sensitivity of coastal bridges to extreme events and climate change can be evaluated by estimating the effect of storm surge, wave heights, and sea level rise on the wave forces. Structural integrity of bridges exposed to wave-induced forces can be evaluated using available methods (e.g. HEC-25 Appendix E, A Method for Estimating Wave Forces on Bridge Superstructures or AASHTO 2008) and comparing such forces to existing structural bridge resistance (weight and connections). This section discusses the methods and wave force results for Mobile Bay Bridge Level 3 analysis.

8.1 Method of Force Calculations

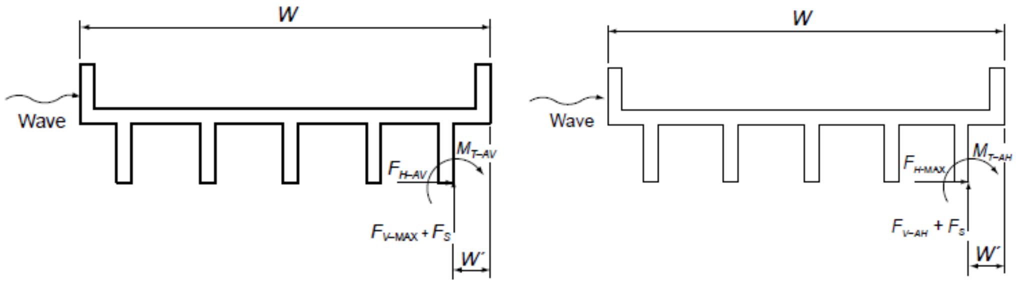
Similar to the environmental conditions (see Section 7) the wave-induces forces presented in this report correspond to the results associated with existing bridge and the project design criteria described in Section 1.4 –100 year SLR with 100-yr event for main span and 50-yr event on ramps. Even though only the design criteria results are presented in this report and digital appendix, this analysis included evaluation of the following scenarios for both existing and proposed bridges:

- 0-yr SLR (2017) + 50-yr storm
- 0-yr SLR (2017) + 100-yr storm
- 0-yr SLR (2017) + 500-yr storm
- 50-yr SLR (2067) + 50-yr storm
- 50-yr SLR (2067) + 100-yr storm
- 50-yr SLR (2067) + 500-yr storm
- 100-yr SLR (2117) + 50-yr storm
- 100-yr SLR (2117) + 100-yr storm
- 100-yr SLR (2117) + 500-yr storm

The wave-induced forces on the existing bridge were computed using the results from the two methods described in Section 6: (1) Extremal Water Surface Elevations and (2) Extremal Significant Wave Heights. The wave-induced forces reported in this document and digital appendix correspond to the maximum result between the two methods.

8.1.1 Forces on Superstructure

The hydrostatic and hydrodynamic forces on the bridge superstructure (deck and girder) due to storm surge and non-linear waves were calculated using the design criteria mentioned in Section 1.4 and the method described in the AASHTO guide specifications (AASHTO, 2008). The hydrostatic and hydrodynamic forces on the superstructure were calculated for Design Case I and Design Case II as illustrated in Figure 19. While the AASHTO guide specifications (AASHTO, 2008) include Design Case III corresponding to overhang design forces, this Level 3 analysis only covers Design Case I and Design Case II. Sample calculations for the superstructure forces are provided in Appendix B.



(a) Case I—$F_{V\text{-}MAX}$ with Associated Forces

(b) Case II—$F_{H\text{-}MAX}$ with Associated Forces

Figure 19. Location of forces and moments for each design case (AASHTO, 2008).

Design Case I – Vertical Forces

The following forces are associated with Design Case I:

- Maximum Quasi-Static Vertical Force, $F_{V\text{-}MAX}$
- Associated Horizontal Quasi-Static Wave Force, $F_{H\text{-}AV}$
- Associated Moment about the Trailing Edge due to Quasi-Static and Slamming Forces, $M_{T\text{-}AV}$
- Associated Vertical Slamming Force, F_S

Design Case I is used to evaluate the vertical resistance to keep the superstructure from separating from the substructure (piles) (AASHTO, 2008). The vertical forces were then identified where the maximum of $F_{V\text{-}MAX}$ + F_S was found. Therefore, the Design Case I results presented in this report are the $F_{V\text{-}MAX}$, $F_{H\text{-}AV}$, $M_{T\text{-}AV}$, and F_S forces associated with the maximum $F_{V\text{-}MAX}$ + F_S is found as prescribed on Figure 12, at each bridge bent.

Design Case II – Horizontal Forces

The following forces are associated with Design Case II:

- Maximum Horizontal Wave Force, $F_{H\text{-}MAX}$
- Associated Quasi-Static Vertical Force, $F_{V\text{-}AH}$
- Associated Moment About the Trailing Edge, $M_{T\text{-}AH}$
- Associated Vertical Slamming Force, F_S

Design Case II is used to evaluate the horizontal resistance of piers and horizontal restraints (AASHTO, 2008).

8.1.2 Forces on Substructure

For the substructure (or piles), the Morison equation as defined in AASHTO guide specifications (AASHTO, 2008) was used to evaluate the hydrodynamic force, F_{Pile}, on the exposed piles and the moment, M_{Pile}, at the base of the pile.

The Morison equation calls for a drag coefficient, Cd, and an inertia coefficient, Cm, which were determined to be Cd = 0.75 and Cm = 1.8 based on recommendations in the Coastal Engineering Manual (U.S. Army Corps of Engineers, 2002). Sample calculations for the pile forces are provided in Appendix B. It should be noted that wind forces are not included in AASHTO (2008) and hence; they have not been included in this report. When the water elevation is near or at the bridge superstructure, the wind speeds will be reduced due to sheltering of the bridge from the wind by the waves and the fact that the wind has a large boundary layer at the rough water surface which reduces the wind speeds. If horizontal wave forces are near the limit state, or when waves are below the girder elevations, a more detailed investigation of winds should be conducted and consideration should be given to including wind forces in structural design. This analysis is out of scope of this report.

8.2 Force Results

The resulting forces on the existing bridge for the design condition are shown on Figure 20. It is found that the piles experience substantial loading due to waves. High pile loads are seen along the entire bridge, but the highest pile loads are observed on the overpasses at the deep channels around station 62,500 ft and station 68,000 ft.

The vertical and horizontal superstructure forces vary significantly along the existing bridge. Such variation is associated with elevation of the wave crest with respect to the superstructure, particularly the bottom of the girder (or top of pile). High superstructure waves forces are observed along the marshes (between station 70,000 ft and 90,000 ft) where the wave crest elevation surpasses the top of deck; meanwhile, no superstructure forces are observed on the overpasses where the wave crest does not impact the bridge deck and/or girder.

Overall, the pile forces are the largest wave-induced force on the bridge, followed the vertical superstructure forces as the second largest wave-induced force, with the horizontal superstructure forces as the smallest wave-induced force on the bridge.

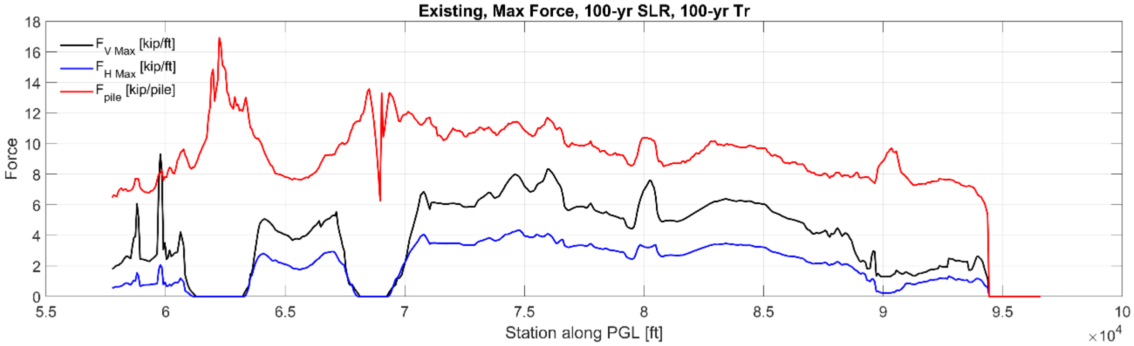


Figure 20. Maximum forces for the existing bridge under design condition 100-yr SLR and 100-yr return period

9 Level 1 and Level 3 Analyses Comparison

In the Level 1 analysis, the environmental conditions, primarily wind and storm surge elevation, were based on existing data. Wave heights were computed by modeling a single, simple hurricane wind field across Mobile Bay that is elevated to a static surge elevation throughout the entire Mobile Bay and Gulf. The surge level was developed based on historical high-water marks fit to an extreme value distribution. Level 1 results are considered an initial estimate and a conservative scenario where only one storm was examined, and the interaction of waves and the dynamic surge associated with hurricanes was not evaluated. The surge elevation was based on a limited historical record not dynamically linked with the waves. In Level 1 the surge was static at a constant, high elevation across the bay and the waves are propagated to the site on that elevated water level for a single storm. The Level 1 method removes the dynamic coupling of wind, surge, and waves as well as ignores the possible variations in storms.

Wave response to hurricanes is a complex process that is affected by the dynamics of the storm including surge, storm track, circulation currents, and wave generation and transformation processes on that dynamic field. The Level 3 analysis included such processes by modeling the coupled wind-surge-wave processes to produce the best-possible prediction of surge and waves at the site. Level 3 analysis involved the modeling of a set of 80 storms using the dynamically coupled wind-surge-wave ADCIRC+SWAN model. At every bridge bent, the surge and wave height results from each storm are fit to a probability distribution that to develop the surge and wave height as a function of return period.

Overall, Level 3 assessment used the best methods to simulate the physics accurately in a probabilistic framework; whereas, Level 1 used simplifying assumptions leading to a conservative result. With the surge and wave height results from Level 3 analysis, the height of the Proposed bridge can and should be optimized. For the bridge main span, the project design was set as the 100-yr SLR and the 100-yr return period event (see Section 1.4); the results comparing the Level 1 and Level 3 results on the Proposed East Bound bridge are shown on Figure 21.

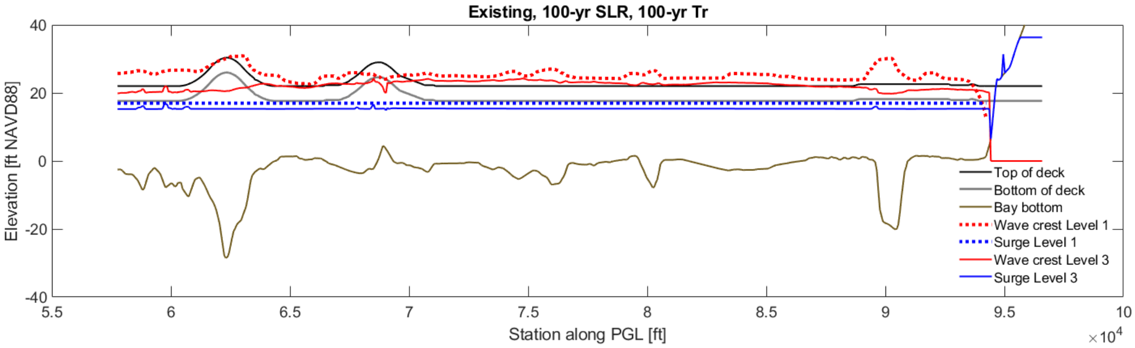


Figure 21. Level 1 versus Level 3 surge and wave crests elevation comparison for existing bridge

In general, for the project design criteria, Level 3 analysis results show lower water surface and lower wave crest elevations when compared to the Level 1 analysis results. In some areas the different between Level 1 and Level 3 crest elevation is as high as 10 ft and 9 ft such as, around station 900+00 and 630+00, respectively. Such areas correspond to the highest wave heights observed at the deeper channels. Level 3 surge elevations also showed lower values when compared to the Level 1 results, with Level 3 being 1.5 ft lower than Level 1 fairly uniformly along the bridge span.

Overall, Level 3 analysis wave crest and water surface elevations were lower than Level 1 results. In addition, Level 3 analysis results are considered more accurate and more robust. Hence, it is recommended to consider Level 3 results on the design of the I-10 Mobile River Bridge and Bayway Project.

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A. Digital Appendix Contents

Mott MacDonald will provide ALDOT a Digital Appendix which will include the final results associated with I-10 Mobile River Bridge and Bayway Project - Storm Surge Impact Analysis Level 3. The Digital Appendix will include the following contents with the pertinent metadata documentation:

A.1 ADCIRC+SWAN Storm Surge Modeling

A.1.1 0-yr SLR Production Runs

- Production run results
- Input files

A.1.2 50-yr SLR Production Runs

- Production run results
- Input files

A.1.3 100-yr SLR Production Runs

- Production run results
- Input files

A.2 Extreme Value Statistics

A.2.1 100-yr SLR Recurrence Interval Analysis

- Method 1 - Extremal Water Surface Elevations Results: Longitude, Latitude, PGL Station, Node Elevation [ft NAVD88], Extremal Water Surface Elevation [ft NAVD88], Associated Velocity [ft/s], Associated Significant Wave height [ft], Associated Peak Wave Period [s].
- Method 2 - Extremal Significant Wave Heights: Longitude, Latitude, Station, Node Elevation [ft NAVD88], WSE [ft NAVD88], Associated Velocity [ft/s], Extremal Significant Wave height [ft], Associated Period [s].

A.3 Environmental Conditions

A.3.1 100-yr SLR 50-yr Tr Environmental Conditions

- Longitude, Latitude, PGL Station [ft], wave crest elevation [ft NAVD88], surge elevation [ft NAVD88] for proposed bridge (2 main spans, and Proposed Bridge 12 ramps).

A.3.2 100-yr SLR 100-yr Tr Environmental Conditions

- Longitude, Latitude, PGL Station [ft], wave crest elevation [ft NAVD88], surge elevation [ft NAVD88] for each bridge (Existing Bridge, Proposed Bridge 2 main spans, and Proposed Bridge 12 ramps).

A.4 Wave-induced Forces on Existing Bridge

A.4.1 100-yr SLR 100-yr Tr wave forces

- Forces on Superstructure
    - Design Case 1:
        - F_{Vmax}, maximum vertical force [kips/ft]
        - F_s, vertical slamming force [kips/ft]
        - $F_{H\text{-}av}$, horizontal load associated with the max vertical [kips/ft]
        - $M_{T\text{-}av}$, moment associated with Design Case 1 [kips-ft/ft]
    - Design Case 2:
        - F_{Hmax}, maximum horizontal force [kips/ft]
        - $F_{V\text{-}ah}$, vertical load associated with max horizontal [kips/ft]
        - M_{T_ah}, moment associated with design case 2 [kips-ft/ft]
    - Current Force [kips/ft]
- Forces on Substructure (Piles)
    - F_T, total max resultant force on the pile [kips/pile]
    - M_T, total moment around mudline [kips-ft/pile]
    - Zr, moment arm [distance above mudline ft]

B. Example Calculations

B.1 Superstructure Forces Example Calculation

MOTT MACDONALD

Mobile Bridge Storm Surge

Calculation

Element: Span 250 100yr SLR - 100yr Storm		Prepared by: Joshua Todd	Date: 6/8/2018	Discipline: Coastal
Description: Wave Forces on Superstructure		Checked by: Katlin Walling	Date: 6/8/2018	
Calculation No: 1.0	Rev. No. 0	Reviewed by: Victoria Curto	Date: 6/8/2018	Sheet No.: 1 of 30

Wave Loading on Bridge Superstructure

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Mobile Bridge Storm Surge

Calculation

Subject: Span 250 100yr SLR - 100yr Storm		Prepared by: Joshua Todd	Date: 6/8/2018	Discipline: Coastal
Component: Wave Forces on Superstructure		Checked by: Katlin Walling	Date: 6/8/2018	
Calculation No: 1.0	Rev. No. 0	Reviewed by: Victoria Curto	Date: 6/8/2018	Sheet No.: 3 of 30

1 Calculation Scope

Calculate wave loads on the Superstructure at Span 250
using methodology perscribed in the Coastal Engineering Manual (USACE, 2012)

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2 Technical Requirements

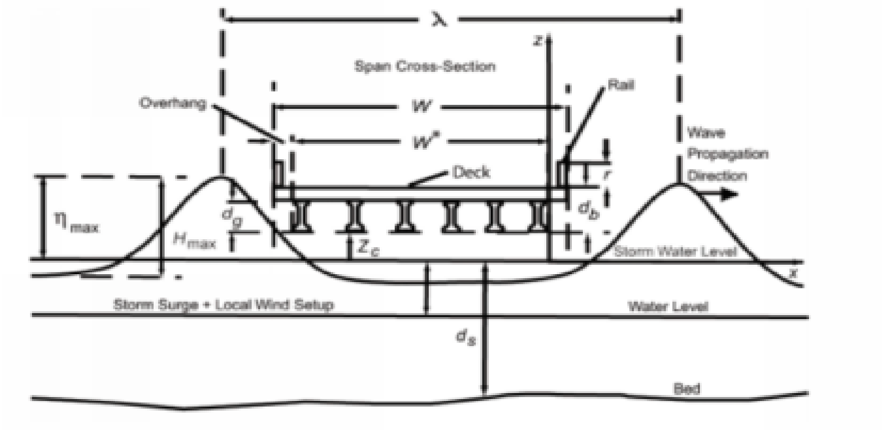
Wave loads are calculated using WSE, Hs, and Tp dervied in Sec. 6 of the Report

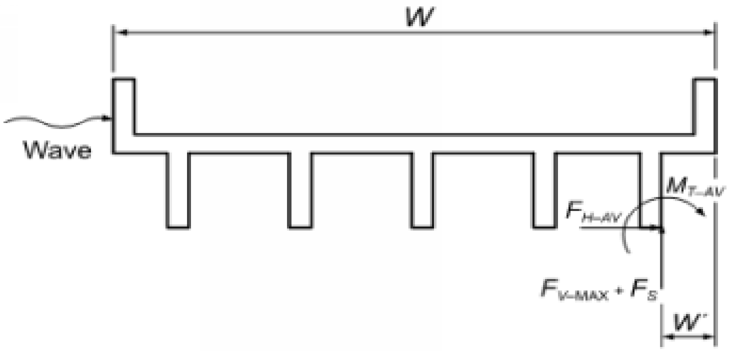
3 Criteria, Codes, and Standards

USACE Coastal Engineering Manual (TR 1.4.2.C.13)

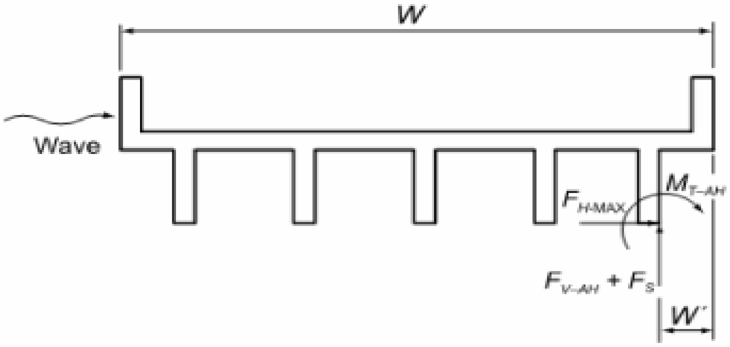
4 Methodology and Assumptions

Description of Geometry and Variables





(a) Case I—$F_{V\text{-MAX}}$ with Associated Forces



(b) Case II—$F_{H\text{-MAX}}$ with Associated Forces

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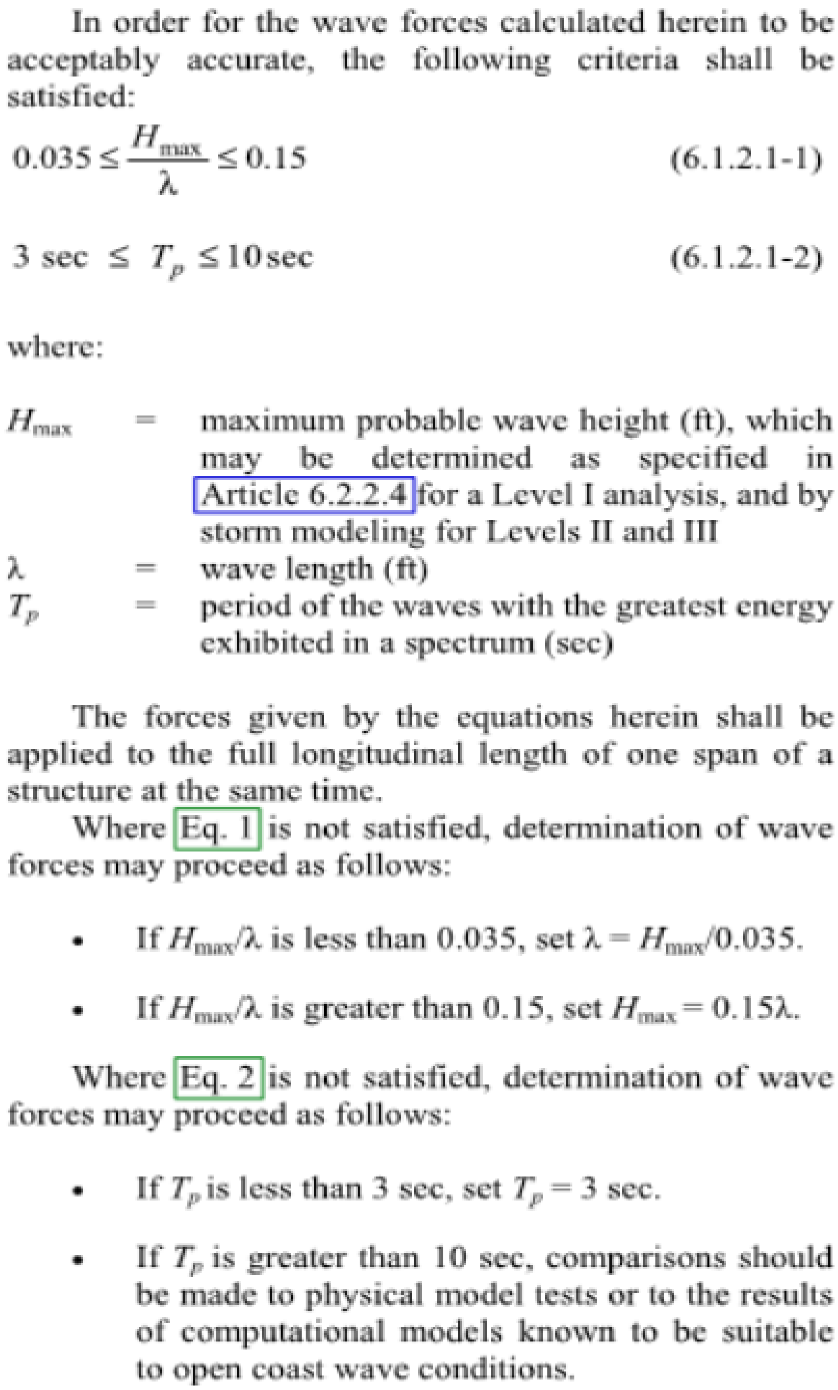


In order for the wave forces calculated herein to be acceptably accurate, the following criteria shall be satisfied:

$$0.035 \leq \frac{H_{max}}{\lambda} \leq 0.15 \qquad (6.1.2.1\text{-}1)$$

$$3 \text{ sec} \leq T_p \leq 10 \text{ sec} \qquad (6.1.2.1\text{-}2)$$

where:

H_{max} = maximum probable wave height (ft), which may be determined as specified in Article 6.2.2.4 for a Level I analysis, and by storm modeling for Levels II and III

λ = wave length (ft)

T_p = period of the waves with the greatest energy exhibited in a spectrum (sec)

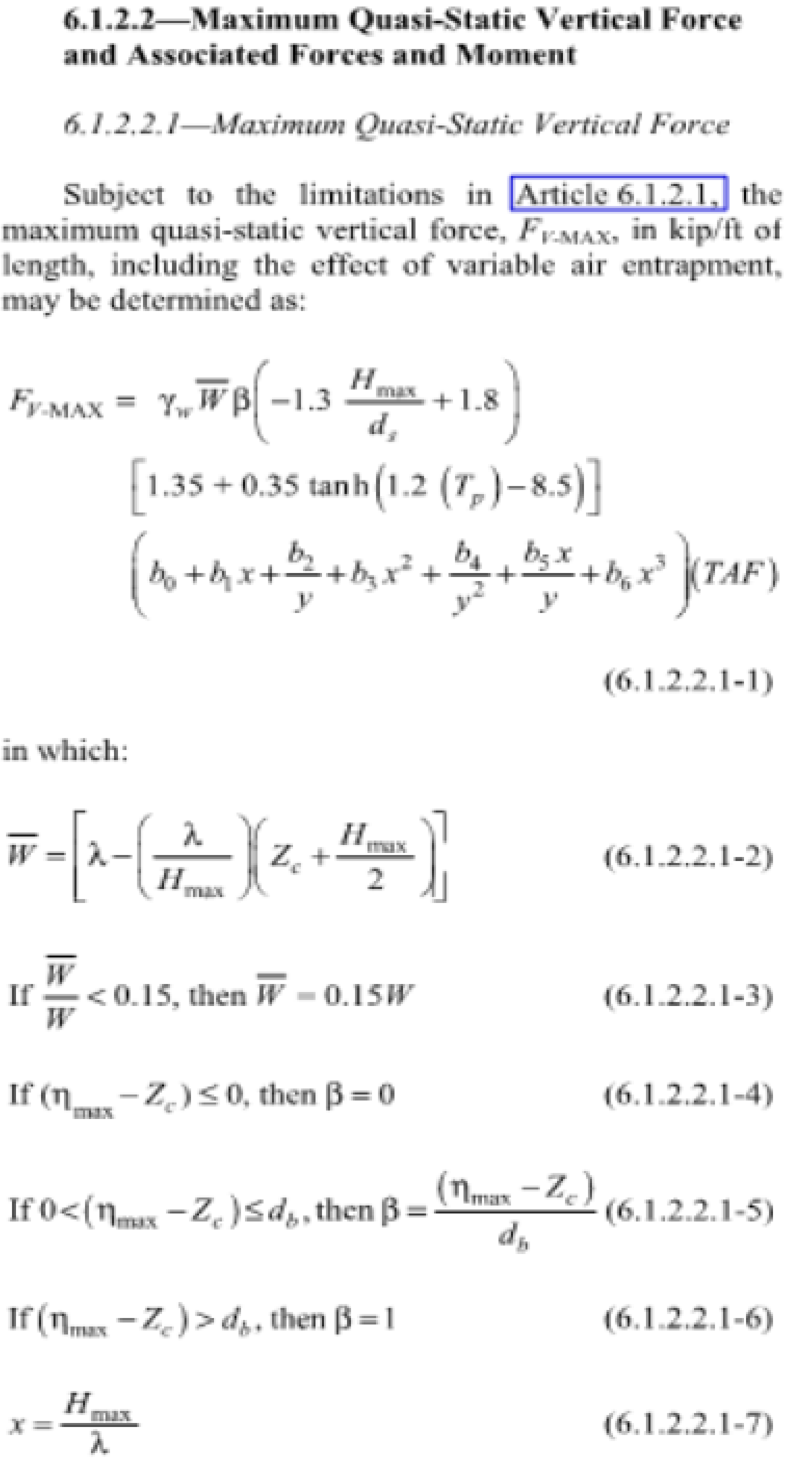
The forces given by the equations herein shall be applied to the full longitudinal length of one span of a structure at the same time.

Where Eq. 1 is not satisfied, determination of wave forces may proceed as follows:

- If H_{max}/λ is less than 0.035, set $\lambda = H_{max}/0.035$.
- If H_{max}/λ is greater than 0.15, set $H_{max} = 0.15\lambda$.

Where Eq. 2 is not satisfied, determination of wave forces may proceed as follows:

- If T_p is less than 3 sec, set $T_p = 3$ sec.
- If T_p is greater than 10 sec, comparisons should be made to physical model tests or to the results of computational models known to be suitable to open coast wave conditions.



6.1.2.2—Maximum Quasi-Static Vertical Force and Associated Forces and Moment

6.1.2.2.1—Maximum Quasi-Static Vertical Force

Subject to the limitations in Article 6.1.2.1, the maximum quasi-static vertical force, $F_{V\text{-}MAX}$, in kip/ft of length, including the effect of variable air entrapment, may be determined as:

$$F_{V\text{-}MAX} = \gamma_w \overline{W} \beta\left(-1.3\frac{H_{max}}{d_s}+1.8\right)\left[1.35+0.35\tanh\left(1.2\left(T_p\right)-8.5\right)\right]\left(b_0+b_1x+\frac{b_2}{y}+b_3x^2+\frac{b_4}{y^2}+\frac{b_5x}{y}+b_6x^3\right)(TAF) \quad (6.1.2.2.1\text{-}1)$$

in which:

$$\overline{W} = \left[\lambda-\left(\frac{\lambda}{H_{max}}\right)\left(Z_c+\frac{H_{max}}{2}\right)\right] \quad (6.1.2.2.1\text{-}2)$$

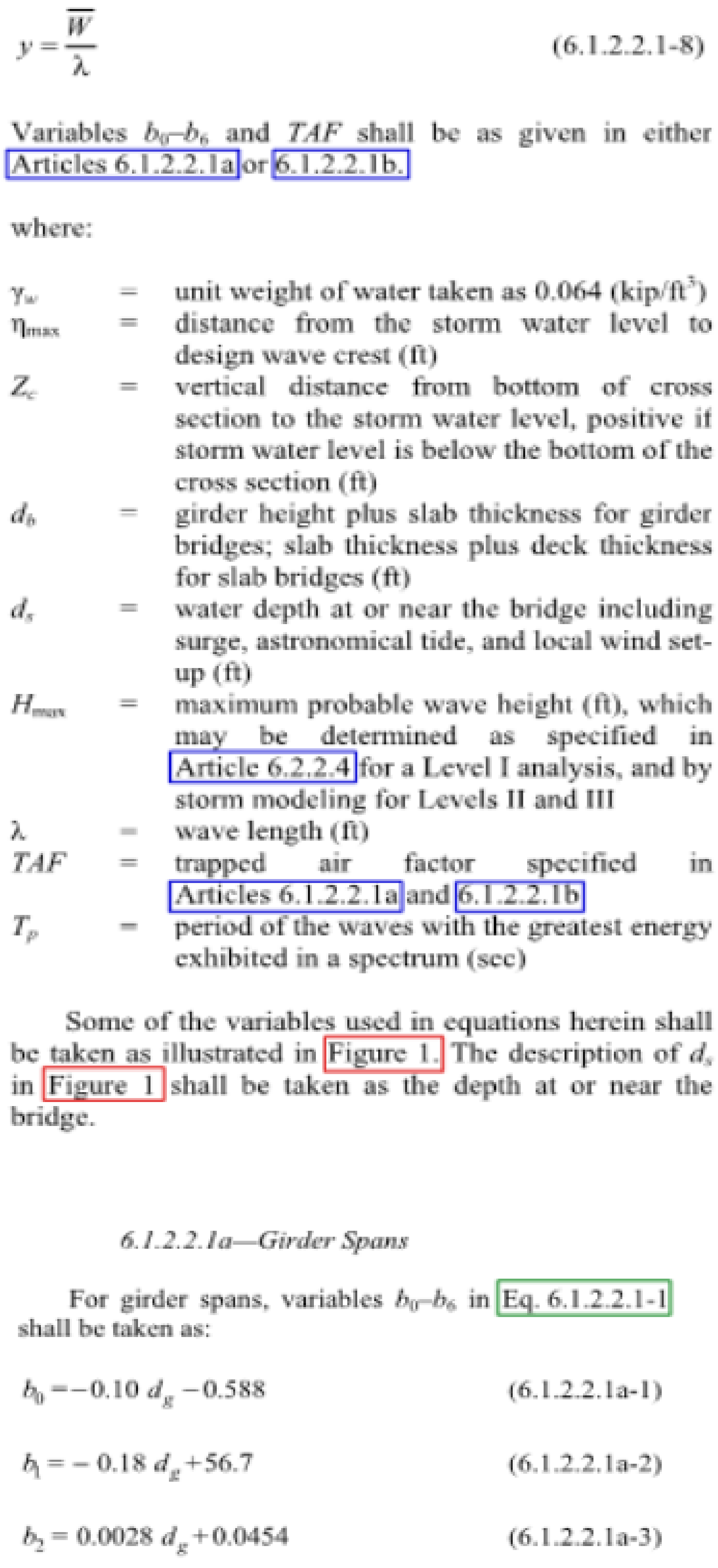
$$\text{If } \frac{\overline{W}}{W} < 0.15, \text{ then } \overline{W} = 0.15W \quad (6.1.2.2.1\text{-}3)$$

$$\text{If } (\eta_{max}-Z_c) \le 0, \text{ then } \beta = 0 \quad (6.1.2.2.1\text{-}4)$$

$$\text{If } 0<(\eta_{max}-Z_c)\le d_b, \text{ then } \beta = \frac{(\eta_{max}-Z_c)}{d_b} \quad (6.1.2.2.1\text{-}5)$$

$$\text{If } (\eta_{max}-Z_c) > d_b, \text{ then } \beta = 1 \quad (6.1.2.2.1\text{-}6)$$

$$x = \frac{H_{max}}{\lambda} \quad (6.1.2.2.1\text{-}7)$$



$$y = \frac{\overline{W}}{\lambda} \qquad (6.1.2.2.1\text{-}8)$$

Variables b_0–b_6 and *TAF* shall be as given in either Articles 6.1.2.2.1a or 6.1.2.2.1b.

where:

γ_w = unit weight of water taken as 0.064 (kip/ft^3)
η_{max} = distance from the storm water level to design wave crest (ft)
Z_c = vertical distance from bottom of cross section to the storm water level, positive if storm water level is below the bottom of the cross section (ft)
d_b = girder height plus slab thickness for girder bridges; slab thickness plus deck thickness for slab bridges (ft)
d_s = water depth at or near the bridge including surge, astronomical tide, and local wind set-up (ft)
H_{max} = maximum probable wave height (ft), which may be determined as specified in Article 6.2.2.4 for a Level I analysis, and by storm modeling for Levels II and III
λ = wave length (ft)
TAF = trapped air factor specified in Articles 6.1.2.2.1a and 6.1.2.2.1b
T_p = period of the waves with the greatest energy exhibited in a spectrum (sec)

Some of the variables used in equations herein shall be taken as illustrated in Figure 1. The description of d_s in Figure 1 shall be taken as the depth at or near the bridge.

6.1.2.2.1a—Girder Spans

For girder spans, variables b_0–b_6 in Eq. 6.1.2.2.1-1 shall be taken as:

$$b_0 = -0.10\,d_g - 0.588 \qquad (6.1.2.2.1a\text{-}1)$$

$$b_1 = -0.18\,d_g + 56.7 \qquad (6.1.2.2.1a\text{-}2)$$

$$b_2 = 0.0028\,d_g + 0.0454 \qquad (6.1.2.2.1a\text{-}3)$$

$$b_3 = 0.2352\, d_g - 193.6 \qquad (6.1.2.2.1a\text{-}4)$$

$$b_4 = -0.00006\, d_g - 0.0003 \qquad (6.1.2.2.1a\text{-}5)$$

$$b_5 = 0.184\, d_g - 0.608 \qquad (6.1.2.2.1a\text{-}6)$$

$$b_6 = 2.1 d_g + 1.56 \qquad (6.1.2.2.1a\text{-}7)$$

The trapped air factor, *TAF*, may be determined as:

$$TAF = A_{AIR}\left(\%AIR\right) + B_{AIR} \le 1.0 \qquad (6.1.2.2.1a\text{-}8)$$

in which:

$$A_{AIR} = 0.0123 - 0.0045 e^{(-Z_c/\eta_{max})} + 0.0014\, ln(W/\lambda)$$

(6.1.2.2.1a-9)

$$B_{AIR} = e^{\left[-2.477 + 1.002 e^{(-Z_c/\eta_{max})} - 0.403\, ln(W/\lambda)\right]} \qquad (6.1.2.2.1a\text{-}10)$$

If $0 < \frac{\eta_{max} - Z_c}{d_g} \le 1$, then $\%AIR$ may be selected

from the range $100\left[1 - \left(\frac{\eta_{max} - Z_c}{d_g}\right)\right]$ to the

maximum amount possible

(6.1.2.2.1a-11)

If $\frac{\eta_{max} - Z_c}{d_g} > 1$, then $\%AIR$ may be selected from the

range 0 to the maximum amount possible

(6.1.2.2.1a-12)

where:

b_0–b_6 = coefficients (ft)

W = total bridge width (ft)

Z_c = vertical distance from bottom of cross section to the storm water level, positive if storm water level is below the bottom of the cross section (ft)

η_{max} = distance from the storm water level to design wave crest (ft)

λ = wave length (ft)

d_g = girder height for girder bridges (ft)

TAF = a factor to adjust the vertical quasi-static force for variable amounts of entrapped air (dim)

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Date: 6/8/2018
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Reviewed by: Victoria Curto
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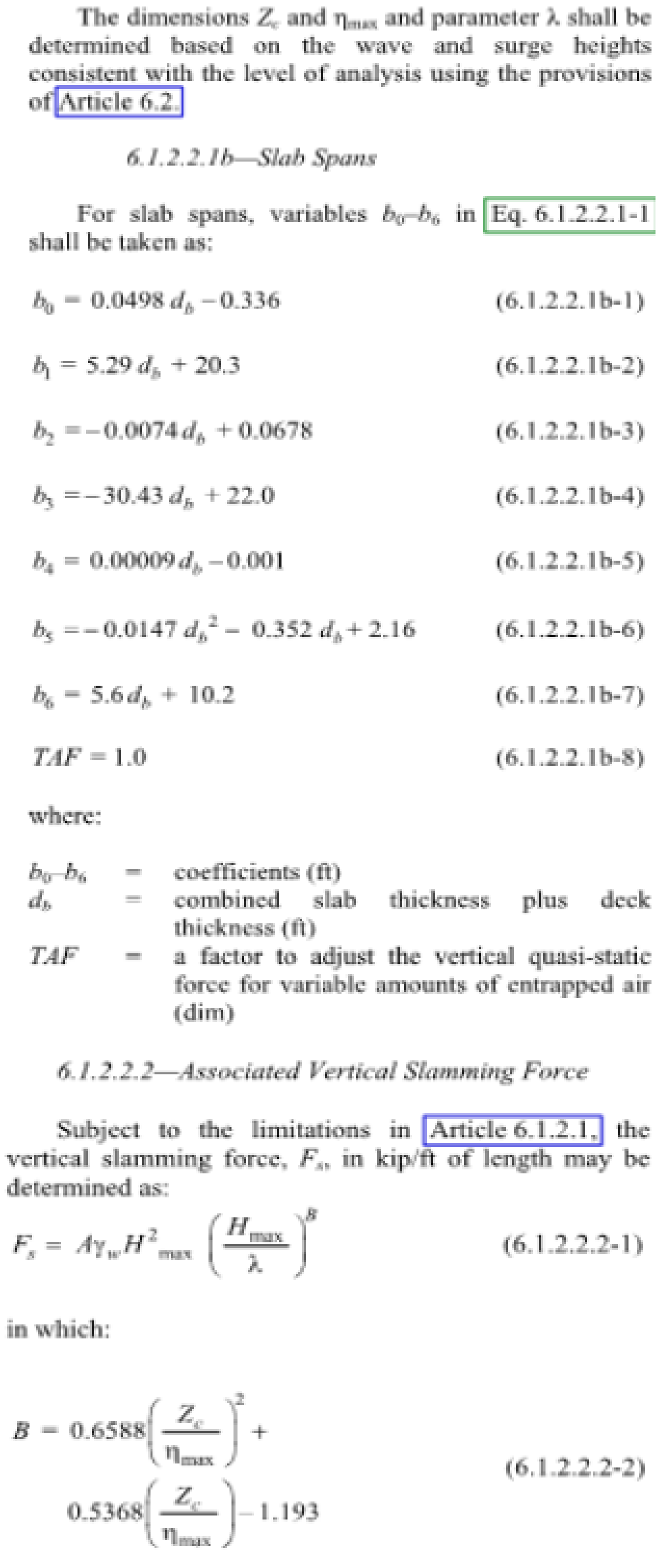

The dimensions Z_c and η_{max} and parameter λ shall be determined based on the wave and surge heights consistent with the level of analysis using the provisions of Article 6.2.

6.1.2.2.1b—Slab Spans

For slab spans, variables b_0–b_6 in Eq. 6.1.2.2.1-1 shall be taken as:

$$b_0 = 0.0498\, d_b - 0.336 \qquad (6.1.2.2.1b\text{-}1)$$

$$b_1 = 5.29\, d_b + 20.3 \qquad (6.1.2.2.1b\text{-}2)$$

$$b_2 = -0.0074\, d_b + 0.0678 \qquad (6.1.2.2.1b\text{-}3)$$

$$b_3 = -30.43\, d_b + 22.0 \qquad (6.1.2.2.1b\text{-}4)$$

$$b_4 = 0.00009\, d_b - 0.001 \qquad (6.1.2.2.1b\text{-}5)$$

$$b_5 = -0.0147\, d_b^{\,2} - 0.352\, d_b + 2.16 \qquad (6.1.2.2.1b\text{-}6)$$

$$b_6 = 5.6\, d_b + 10.2 \qquad (6.1.2.2.1b\text{-}7)$$

$$TAF = 1.0 \qquad (6.1.2.2.1b\text{-}8)$$

where:

b_0–b_6 = coefficients (ft)
d_b = combined slab thickness plus deck thickness (ft)
TAF = a factor to adjust the vertical quasi-static force for variable amounts of entrapped air (dim)

6.1.2.2.2—Associated Vertical Slamming Force

Subject to the limitations in Article 6.1.2.1, the vertical slamming force, F_s, in kip/ft of length may be determined as:

$$F_s = A\gamma_w H^2_{max}\left(\frac{H_{max}}{\lambda}\right)^B \qquad (6.1.2.2.2\text{-}1)$$

in which:

$$B = 0.6588\left(\frac{Z_c}{\eta_{max}}\right)^2 + 0.5368\left(\frac{Z_c}{\eta_{max}}\right) - 1.193 \qquad (6.1.2.2.2\text{-}2)$$

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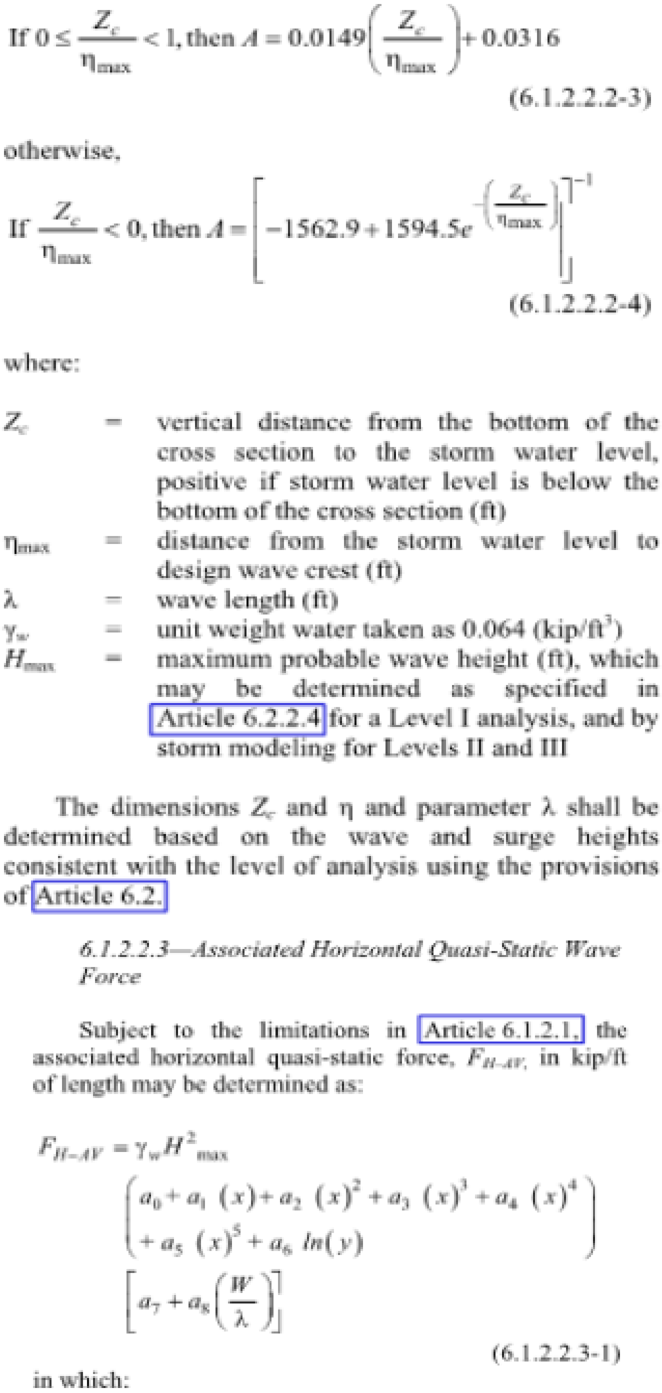

If $0 \le \frac{Z_c}{\eta_{max}} < 1$, then $A = 0.0149\left(\frac{Z_c}{\eta_{max}}\right) + 0.0316$

(6.1.2.2.2-3)

otherwise,

If $\frac{Z_c}{\eta_{max}} < 0$, then $A = \left[-1562.9 + 1594.5e^{-\left(\frac{Z_c}{\eta_{max}}\right)}\right]^{-1}$

(6.1.2.2.2-4)

where:

Z_c = vertical distance from the bottom of the cross section to the storm water level, positive if storm water level is below the bottom of the cross section (ft)

η_{max} = distance from the storm water level to design wave crest (ft)

λ = wave length (ft)

γ_w = unit weight water taken as 0.064 (kip/ft^3)

H_{max} = maximum probable wave height (ft), which may be determined as specified in Article 6.2.2.4 for a Level I analysis, and by storm modeling for Levels II and III

The dimensions Z_c and η and parameter λ shall be determined based on the wave and surge heights consistent with the level of analysis using the provisions of Article 6.2.

6.1.2.2.3—Associated Horizontal Quasi-Static Wave Force

Subject to the limitations in Article 6.1.2.1, the associated horizontal quasi-static force, F_{H-AV}, in kip/ft of length may be determined as:

$$F_{H-AV} = \gamma_w H^2{}_{max} \begin{pmatrix} a_0 + a_1\ (x) + a_2\ (x)^2 + a_3\ (x)^3 + a_4\ (x)^4 \\ + a_5\ (x)^5 + a_6\ ln(y) \end{pmatrix} \left[a_7 + a_8\left(\frac{W}{\lambda}\right)\right]$$

(6.1.2.2.3-1)

in which:

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Mobile Bridge Storm Surge

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Span 250 100yr SLR - 100yr Storm		Joshua Todd	6/8/2018	Coastal
Component:		Checked by:	Date:	
Wave Forces on Superstructure		Katlin Walling	6/8/2018	
Calculation No:	Rev. No.	Reviewed by:	Date:	Sheet No.:
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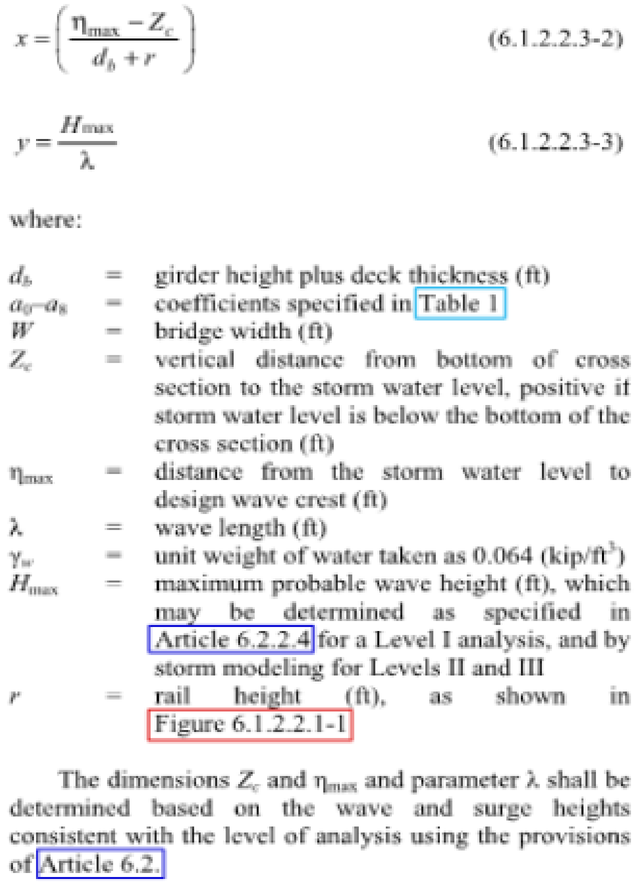

$$x = \left(\frac{\eta_{max} - Z_c}{d_b + r} \right) \qquad (6.1.2.2.3\text{-}2)$$

$$y = \frac{H_{max}}{\lambda} \qquad (6.1.2.2.3\text{-}3)$$

where:

d_b = girder height plus deck thickness (ft)
a_0–a_8 = coefficients specified in Table 1
W = bridge width (ft)
Z_c = vertical distance from bottom of cross section to the storm water level, positive if storm water level is below the bottom of the cross section (ft)
η_{max} = distance from the storm water level to design wave crest (ft)
λ = wave length (ft)
γ_w = unit weight of water taken as 0.064 (kip/ft^3)
H_{max} = maximum probable wave height (ft), which may be determined as specified in Article 6.2.2.4 for a Level I analysis, and by storm modeling for Levels II and III
r = rail height (ft), as shown in Figure 6.1.2.2.1-1

The dimensions Z_c and η_{max} and parameter λ shall be determined based on the wave and surge heights consistent with the level of analysis using the provisions of Article 6.2.

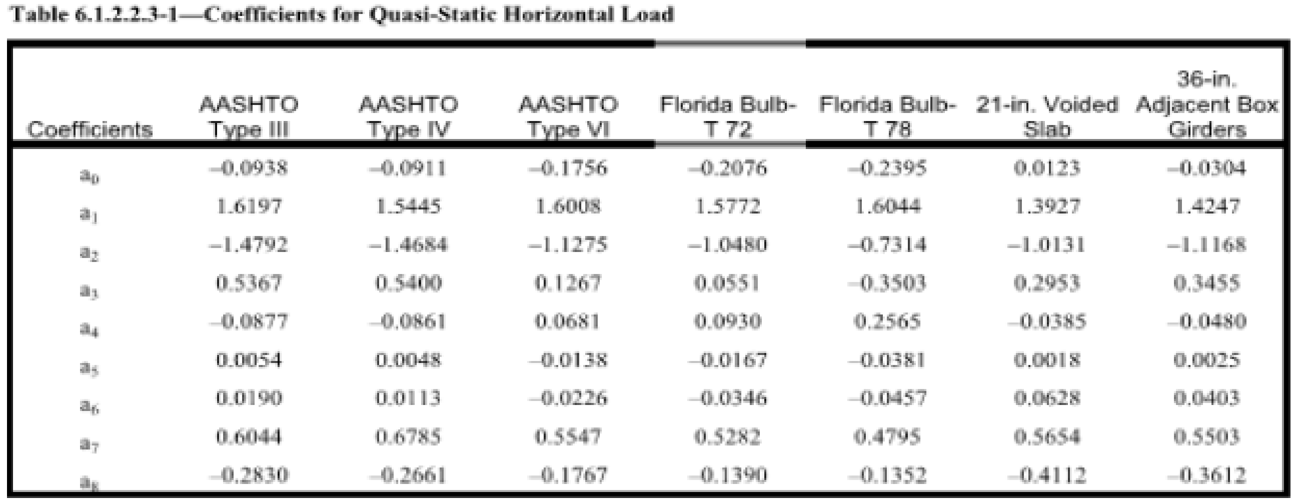

Table 6.1.2.2.3-1—Coefficients for Quasi-Static Horizontal Load

Coefficients	AASHTO Type III	AASHTO Type IV	AASHTO Type VI	Florida Bulb-T 72	Florida Bulb-T 78	21-in. Voided Slab	36-in. Adjacent Box Girders
a_0	−0.0938	−0.0911	−0.1756	−0.2076	−0.2395	0.0123	−0.0304
a_1	1.6197	1.5445	1.6008	1.5772	1.6044	1.3927	1.4247
a_2	−1.4792	−1.4684	−1.1275	−1.0480	−0.7314	−1.0131	−1.1168
a_3	0.5367	0.5400	0.1267	0.0551	−0.3503	0.2953	0.3455
a_4	−0.0877	−0.0861	0.0681	0.0930	0.2565	−0.0385	−0.0480
a_5	0.0054	0.0048	−0.0138	−0.0167	−0.0381	0.0018	0.0025
a_6	0.0190	0.0113	−0.0226	−0.0346	−0.0457	0.0628	0.0403
a_7	0.6044	0.6785	0.5547	0.5282	0.4795	0.5654	0.5503
a_8	−0.2830	−0.2661	−0.1767	−0.1390	−0.1352	−0.4112	−0.3612

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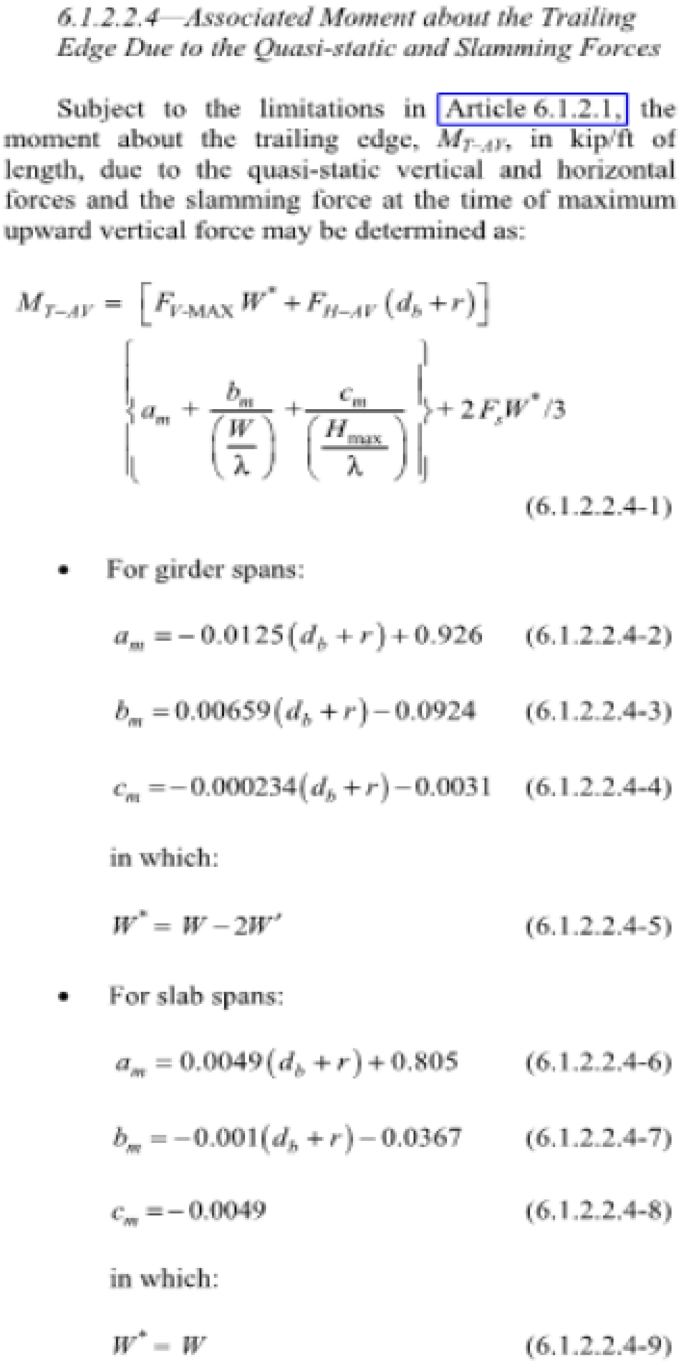


6.1.2.2.4—Associated Moment about the Trailing Edge Due to the Quasi-static and Slamming Forces

Subject to the limitations in Article 6.1.2.1, the moment about the trailing edge, M_{T-AV}, in kip/ft of length, due to the quasi-static vertical and horizontal forces and the slamming force at the time of maximum upward vertical force may be determined as:

$$M_{T-AV} = \left[F_{V\text{-MAX}} W^{*} + F_{H-AV}\left(d_b + r\right)\right] \left\{a_m + \frac{b_m}{\left(\frac{W}{\lambda}\right)} + \frac{c_m}{\left(\frac{H_{\max}}{\lambda}\right)}\right\} + 2F_s W^{*}/3 \quad (6.1.2.2.4\text{-}1)$$

- For girder spans:

$$a_m = -0.0125\left(d_b + r\right) + 0.926 \quad (6.1.2.2.4\text{-}2)$$

$$b_m = 0.00659\left(d_b + r\right) - 0.0924 \quad (6.1.2.2.4\text{-}3)$$

$$c_m = -0.000234\left(d_b + r\right) - 0.0031 \quad (6.1.2.2.4\text{-}4)$$

in which:

$$W^{*} = W - 2W' \quad (6.1.2.2.4\text{-}5)$$

- For slab spans:

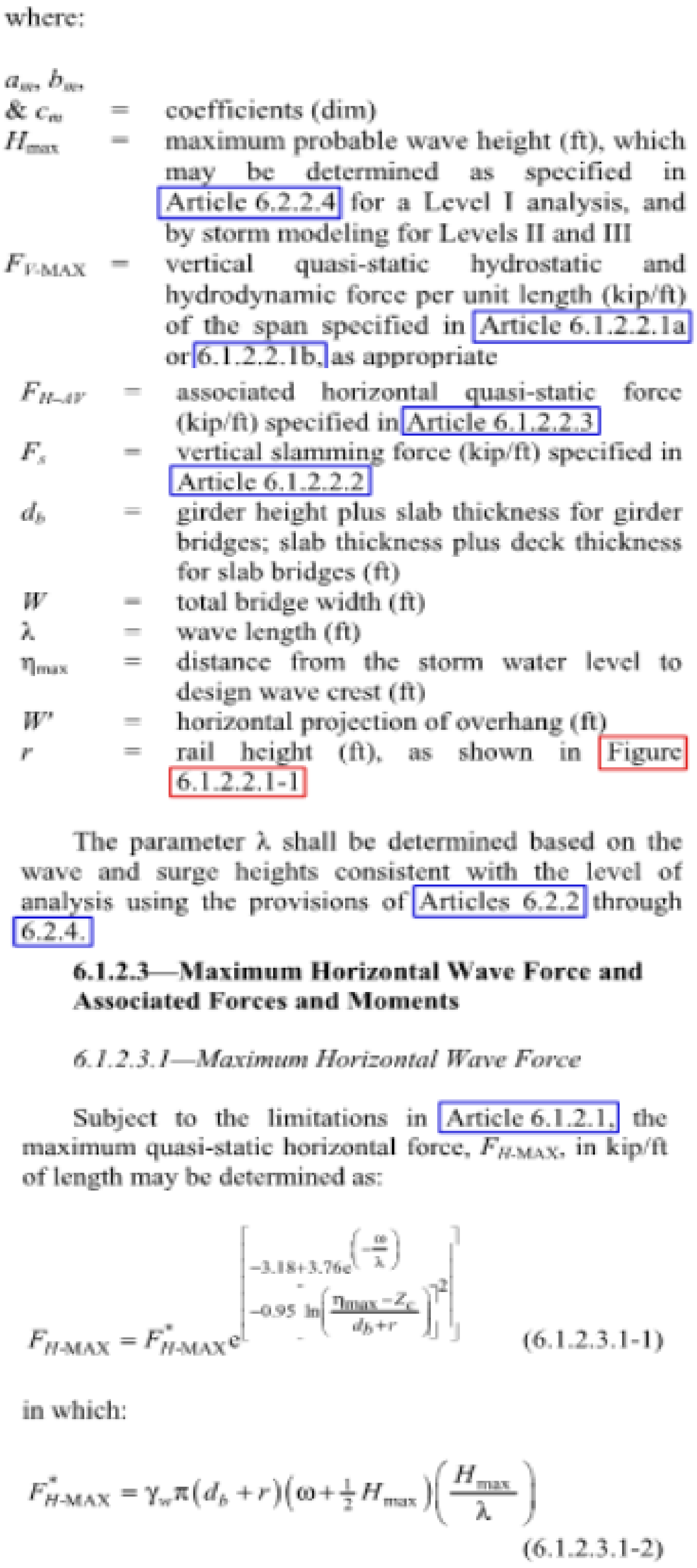
$$a_m = 0.0049\left(d_b + r\right) + 0.805 \quad (6.1.2.2.4\text{-}6)$$

$$b_m = -0.001\left(d_b + r\right) - 0.0367 \quad (6.1.2.2.4\text{-}7)$$

$$c_m = -0.0049 \quad (6.1.2.2.4\text{-}8)$$

in which:

$$W^{*} = W \quad (6.1.2.2.4\text{-}9)$$



where:

a_w, b_w, & c_w = coefficients (dim)

H_{max} = maximum probable wave height (ft), which may be determined as specified in Article 6.2.2.4 for a Level I analysis, and by storm modeling for Levels II and III

$F_{V\text{-}MAX}$ = vertical quasi-static hydrostatic and hydrodynamic force per unit length (kip/ft) of the span specified in Article 6.1.2.2.1a or 6.1.2.2.1b, as appropriate

$F_{H\text{-}AV}$ = associated horizontal quasi-static force (kip/ft) specified in Article 6.1.2.2.3

F_s = vertical slamming force (kip/ft) specified in Article 6.1.2.2.2

d_b = girder height plus slab thickness for girder bridges; slab thickness plus deck thickness for slab bridges (ft)

W = total bridge width (ft)

λ = wave length (ft)

η_{max} = distance from the storm water level to design wave crest (ft)

W' = horizontal projection of overhang (ft)

r = rail height (ft), as shown in Figure 6.1.2.2.1-1

The parameter λ shall be determined based on the wave and surge heights consistent with the level of analysis using the provisions of Articles 6.2.2 through 6.2.4.

6.1.2.3—Maximum Horizontal Wave Force and Associated Forces and Moments

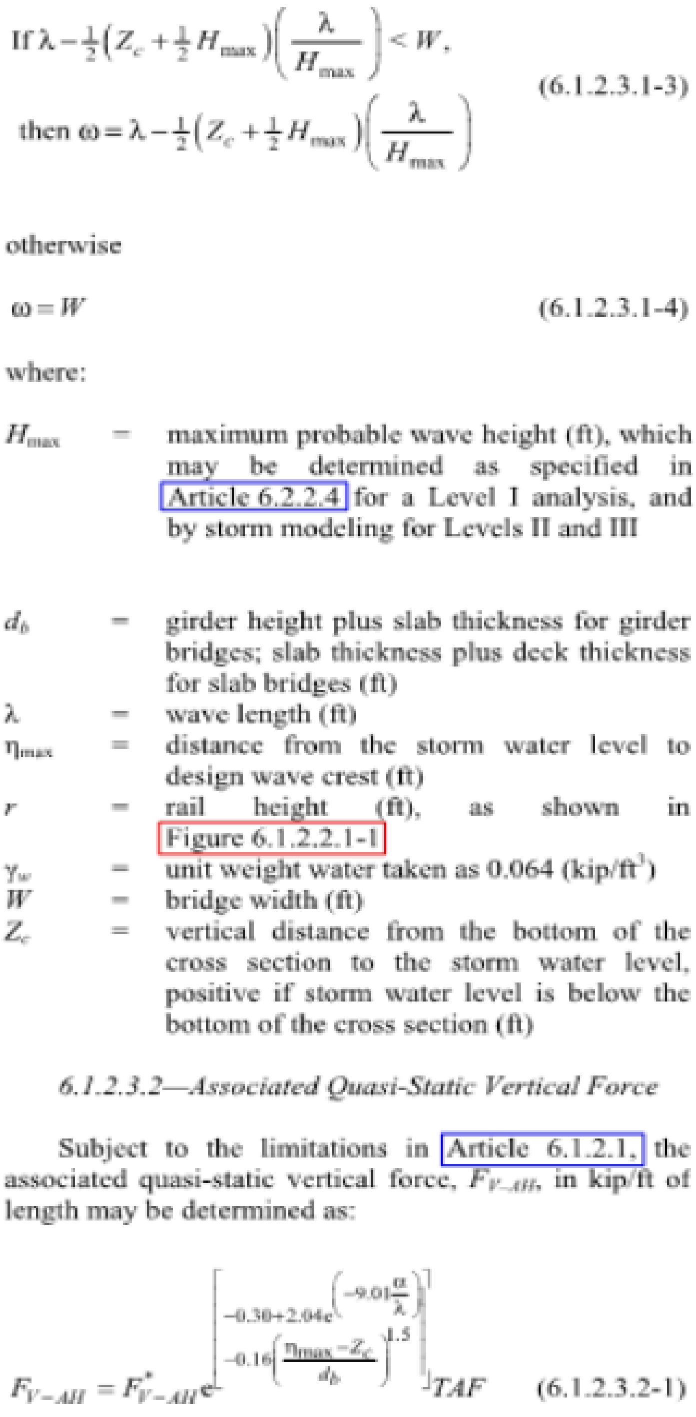
6.1.2.3.1—Maximum Horizontal Wave Force

Subject to the limitations in Article 6.1.2.1, the maximum quasi-static horizontal force, $F_{H\text{-}MAX}$, in kip/ft of length may be determined as:

$$F_{H\text{-}MAX} = F^{*}_{H\text{-}MAX} e^{\left[-3.18+3.76e^{\left(-\frac{\omega}{\lambda}\right)} - 0.95\left[\ln\left(\frac{\eta_{max}-Z_c}{d_b+r}\right)\right]^2\right]} \qquad (6.1.2.3.1\text{-}1)$$

in which:

$$F^{*}_{H\text{-}MAX} = \gamma_w \pi (d_b + r)\left(\omega + \tfrac{1}{2}H_{max}\right)\left(\frac{H_{max}}{\lambda}\right) \qquad (6.1.2.3.1\text{-}2)$$



If $\lambda - \frac{1}{2}\left(Z_c + \frac{1}{2}H_{max}\right)\left(\frac{\lambda}{H_{max}}\right) < W$,

then $\omega = \lambda - \frac{1}{2}\left(Z_c + \frac{1}{2}H_{max}\right)\left(\frac{\lambda}{H_{max}}\right)$ (6.1.2.3.1-3)

otherwise

$\omega = W$ (6.1.2.3.1-4)

where:

H_{max} = maximum probable wave height (ft), which may be determined as specified in Article 6.2.2.4 for a Level I analysis, and by storm modeling for Levels II and III

d_b = girder height plus slab thickness for girder bridges; slab thickness plus deck thickness for slab bridges (ft)

λ = wave length (ft)

η_{max} = distance from the storm water level to design wave crest (ft)

r = rail height (ft), as shown in Figure 6.1.2.2.1-1

γ_w = unit weight water taken as 0.064 (kip/ft^3)

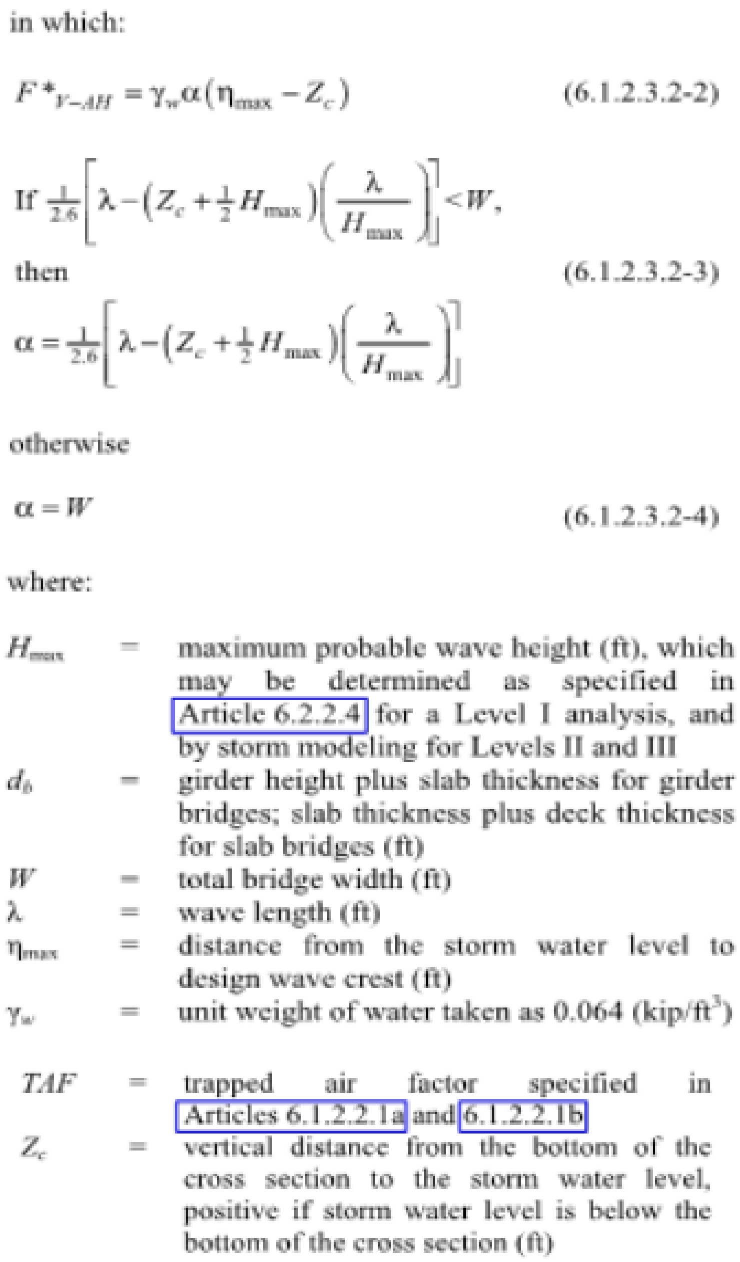
W = bridge width (ft)

Z_c = vertical distance from the bottom of the cross section to the storm water level, positive if storm water level is below the bottom of the cross section (ft)

6.1.2.3.2—Associated Quasi-Static Vertical Force

Subject to the limitations in Article 6.1.2.1, the associated quasi-static vertical force, F_{V-AH}, in kip/ft of length may be determined as:

$$F_{V-AH} = F^*_{V-AH}\, e^{\left[-0.30+2.04e^{\left(-9.01\frac{\omega}{\lambda}\right)} -0.16\left(\frac{\eta_{max}-Z_c}{d_b}\right)^{1.5}\right]} TAF \qquad (6.1.2.3.2\text{-}1)$$



in which:

$$F^*_{V-AH} = \gamma_w \alpha(\eta_{max} - Z_c) \qquad (6.1.2.3.2\text{-}2)$$

If $\frac{1}{2.6}\left[\lambda - \left(Z_c + \frac{1}{2}H_{max}\right)\left(\frac{\lambda}{H_{max}}\right)\right] < W$,

then (6.1.2.3.2-3)

$$\alpha = \frac{1}{2.6}\left[\lambda - \left(Z_c + \frac{1}{2}H_{max}\right)\left(\frac{\lambda}{H_{max}}\right)\right]$$

otherwise

$$\alpha = W \qquad (6.1.2.3.2\text{-}4)$$

where:

H_{max} = maximum probable wave height (ft), which may be determined as specified in Article 6.2.2.4 for a Level I analysis, and by storm modeling for Levels II and III

d_b = girder height plus slab thickness for girder bridges; slab thickness plus deck thickness for slab bridges (ft)

W = total bridge width (ft)

λ = wave length (ft)

η_{max} = distance from the storm water level to design wave crest (ft)

γ_w = unit weight of water taken as 0.064 (kip/ft^3)

TAF = trapped air factor specified in Articles 6.1.2.2.1a and 6.1.2.2.1b

Z_c = vertical distance from the bottom of the cross section to the storm water level, positive if storm water level is below the bottom of the cross section (ft)

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Mobile Bridge Storm Surge

Calculation

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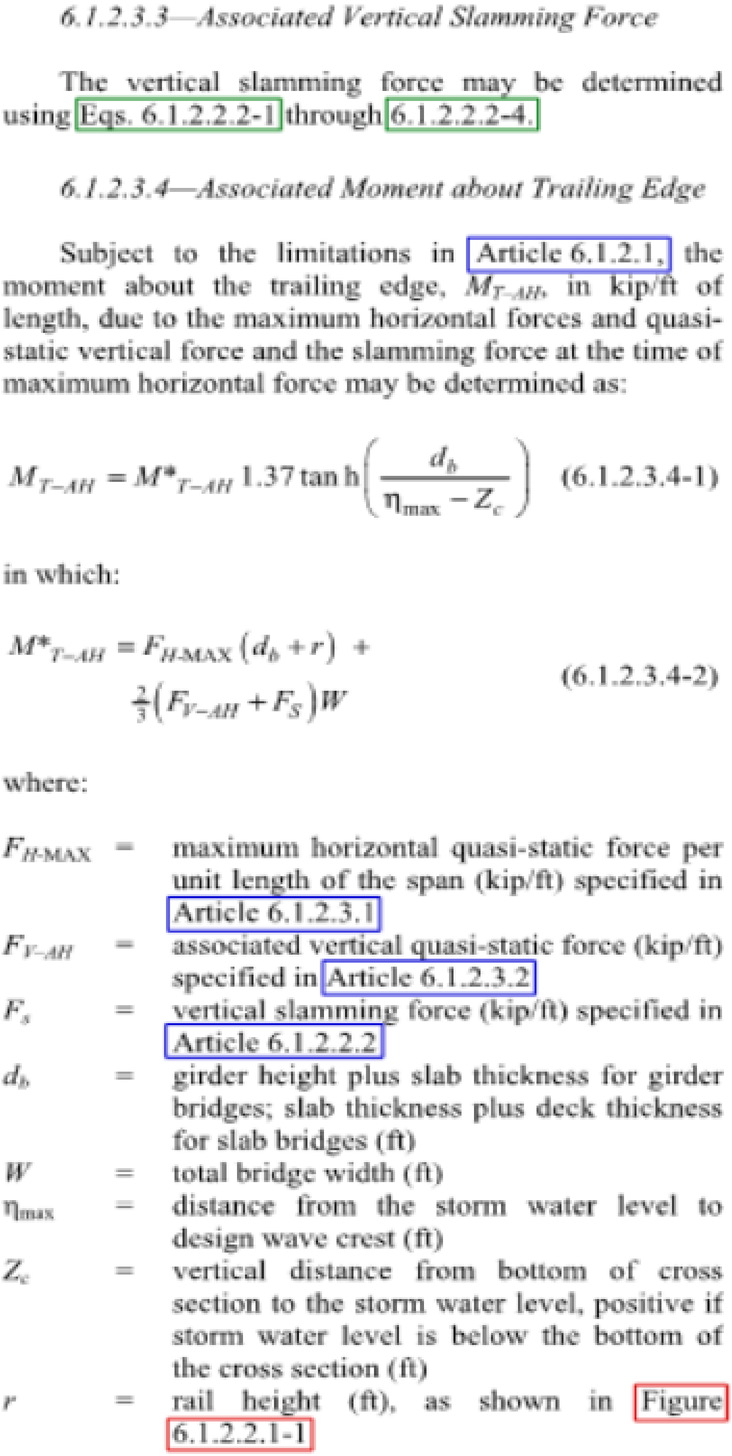

6.1.2.3.3—Associated Vertical Slamming Force

The vertical slamming force may be determined using Eqs. 6.1.2.2.2-1 through 6.1.2.2.2-4.

6.1.2.3.4—Associated Moment about Trailing Edge

Subject to the limitations in Article 6.1.2.1, the moment about the trailing edge, M_{T-AH}, in kip/ft of length, due to the maximum horizontal forces and quasi-static vertical force and the slamming force at the time of maximum horizontal force may be determined as:

$$M_{T-AH} = M^*_{T-AH}\,1.37\tan\mathrm{h}\left(\frac{d_b}{\eta_{\max} - Z_c}\right) \qquad (6.1.2.3.4\text{-}1)$$

in which:

$$M^*_{T-AH} = F_{H\text{-MAX}}\left(d_b + r\right) + \tfrac{2}{3}\left(F_{V-AH} + F_S\right)W \qquad (6.1.2.3.4\text{-}2)$$

where:

$F_{H\text{-MAX}}$ = maximum horizontal quasi-static force per unit length of the span (kip/ft) specified in Article 6.1.2.3.1

F_{V-AH} = associated vertical quasi-static force (kip/ft) specified in Article 6.1.2.3.2

F_s = vertical slamming force (kip/ft) specified in Article 6.1.2.2.2

d_b = girder height plus slab thickness for girder bridges; slab thickness plus deck thickness for slab bridges (ft)

W = total bridge width (ft)

$\eta_{\max}$ = distance from the storm water level to design wave crest (ft)

Z_c = vertical distance from bottom of cross section to the storm water level, positive if storm water level is below the bottom of the cross section (ft)

r = rail height (ft), as shown in Figure 6.1.2.2.1-1

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Mobile Bridge Storm Surge

Calculation

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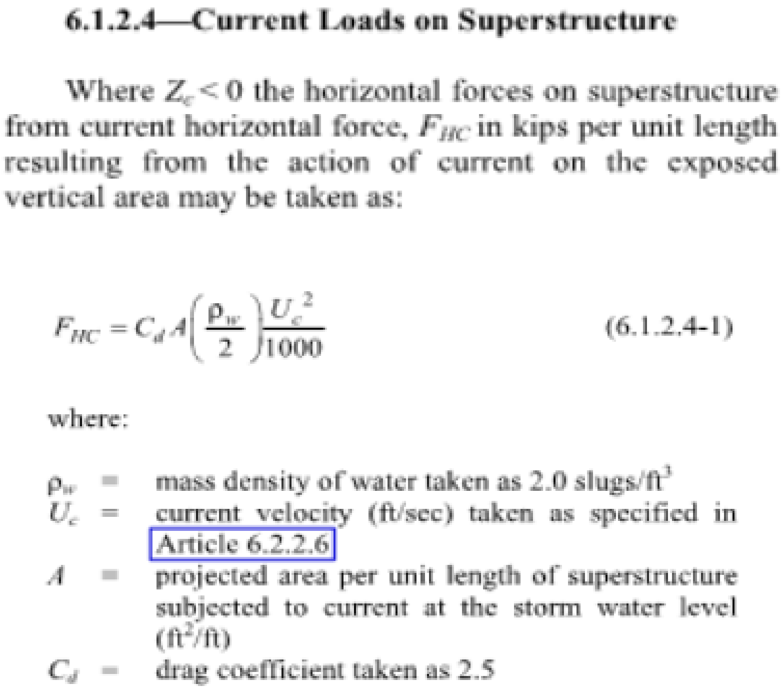

6.1.2.4—Current Loads on Superstructure

Where $Z_c < 0$ the horizontal forces on superstructure from current horizontal force, F_{HC} in kips per unit length resulting from the action of current on the exposed vertical area may be taken as:

$$F_{HC} = C_d A\left(\frac{\rho_w}{2}\right)\frac{U_c^{\,2}}{1000} \qquad (6.1.2.4\text{-}1)$$

where:

ρ_w = mass density of water taken as 2.0 slugs/ft^3

U_c = current velocity (ft/sec) taken as specified in Article 6.2.2.6

A = projected area per unit length of superstructure subjected to current at the storm water level (ft^2/ft)

C_d = drag coefficient taken as 2.5

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Mobile Bridge Storm Surge

Calculation

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5 Computations and Results

Input Parameters

Inputs from As-Builts

Width of Bridge Deck	$W_{given} \equiv 42.92ft$
Horizontal projection of Overhang	$W_{prime} \equiv 2.54ft$
Girder height	$d_g \equiv 3.75ft$
Bridge Type (1 for Girder spans, 2 for Slab spans)	$B_{type} \equiv 1$
Rail height	$r \equiv 3.83ft$
Deck Thickness	$d_{thickness} \equiv 0.63ft$
Deck Elevation in Feet NAVD88	$d_{elev} \equiv 22.01ft$
Water depth from NAVD88 (ft)	$depth \equiv 2.15ft$
Bottom of Girder NAVD88 (ft)	$BoG \equiv 17.64ft$

Geometric values calculated from as-builts

$$d_b \equiv d_{thickness} + d_g = 4.4ft$$

Values from MM Level 3 Analysis

Note: Values below were calculated using various numerical models and empirical analyses

Max wave Height as determined by Coastal Engineer	$H_{maxgiven} \equiv 10.95ft$
Peak Wave Period	$T_{pgiven} \equiv 5.55sec$
Distance from storm water level to wave crest	$\eta_{max} \equiv 8.23ft$
Wavelength	$\lambda_{given} \equiv 149.827ft$
Design Current Velocity from Modeling	$U_c \equiv 2.72\frac{ft}{s}$
Design Water Surface Elevation (ft NAVD88)	$WSE \equiv 15.4ft$

Constants

unit weight of water $\gamma_W \equiv 64 \frac{lbf}{ft^3}$

acceleration due to gravity $g \equiv 32.174 \frac{ft}{s^2}$

Values to be calculated from combination of As-builts and values from Level 3 Analysis

$d_S \equiv WSE + depth = 17.55ft$

$Z_C \equiv BoG - WSE = 2.2ft$

Calculations

check Hmax/λ

$$ratio_{Hmax\_\lambda} \equiv \frac{H_{max\,given}}{\lambda_{given}} = 0.1$$

$$\lambda \equiv if\left(ratio_{Hmax\_\lambda} < 0.035, \frac{H_{maxgiven}}{0.035}, \lambda_{given}\right) = 149.8ft$$

$$H_{max} \equiv if\left[\left(ratio_{Hmax\_\lambda} > 0.15\right), 0.15 \cdot \lambda_{given}, H_{max\,given}\right] = 11ft$$

$$T_p \equiv if\left(T_{pgiven} < 3s, 3s, T_{pgiven}\right) = 5.6s$$

Maximum Quasi-Static Vertical Force and Associated Force and Moment

Maximum Quasi-Static Vertical Force (Fvmax)

$$W_{barinit} \equiv \left[\left[\lambda - \left(\frac{\lambda}{H_{max}}\right)\cdot\left[Z_c + \left(\frac{H_{max}}{2}\right)\right]\right]\right] = 44.3\,\text{ft}$$

$$W_{bar} \equiv \text{if}\left[\left(\frac{W_{barinit}}{W_{given}}\right) < 0.15, 0.15\cdot W_{given}, W_{barinit}\right] = 44.3\,\text{ft}$$

$$\beta \equiv \text{if}\left[\left(\eta_{max} - Z_c\right) \le 0, 0, \left[\text{if}\left[\left(\eta_{max} - Z_c\right) > d_b, 1, \left[\frac{\left(\eta_{max} - Z_c\right)}{d_b}\right]\right]\right]\right] = 1$$

$$x \equiv \frac{H_{max}}{\lambda} = 0.1$$

$$y \equiv \frac{W_{bar}}{\lambda} = 0.3$$

Girder Spans

$$b_{0g} \equiv \left(-0.1\cdot d_g\right) - 0.588\text{ft} = -1\,\text{ft}$$

$$b_{1g} \equiv \left(-0.18\cdot d_g\right) + 56.7\text{ft} = 56\,\text{ft}$$

$$b_{2g} \equiv \left(0.0028\cdot d_g\right) + 0.0454\text{ft} = 0.1\,\text{ft}$$

$$b_{3g} \equiv \left(0.2352\cdot d_g\right) - 193.6\text{ft} = -192.7\text{ft}$$

$$b_{4g} \equiv \left(-0.00006\cdot d_g\right) - 0.0003\text{ft} = -5.2\times 10^{-4}\,\text{ft}$$

$$b_{5g} \equiv \left(0.184\cdot d_g\right) - 0.608\text{ft} = 0.1\,\text{ft}$$

$$b_{6g} \equiv \left(2.1\cdot d_g\right) + 1.56\text{ft} = 9.4\,\text{ft}$$

$$A_{air} \equiv 0.0123 - \left[0.0045\cdot e^{\left(\frac{-Z_c}{\eta_{max}}\right)}\right] + \left(0.0014\cdot\ln\left(\frac{W_{given}}{\lambda}\right)\right) = 7.1\times 10^{-3}$$

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Mobile Bridge Storm Surge

Calculation

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$$B_{air} \equiv e^{\left[-2.477+\left[1.002\cdot e^{\left(\frac{-z_c}{\eta_{max}}\right)}\right]-\left(0.403\cdot \ln\left(\frac{W_{given}}{\lambda}\right)\right)\right]} = 0.3$$

$$ratio_{air\_wavecrest} \equiv \frac{\left(\eta_{max} - z_c\right)}{d_g} = 1.6$$

minimum %AIR entrapped in bridge (Note: for when Ratio above is between 0 and 1

$$min_{airperc} \equiv 100\cdot\left[1 - \left(ratio_{air\_wavecrest}\right)\right] = -59.7$$

if ratio above is greater than or eqal to 1 then %AIR entrapped can be any value between 0 and 100%

$$perc_{air\_entrap} \equiv 100$$

Note: 100% was assumed for conservative estimate

$$TAF_g \equiv \left(A_{air}\cdot perc_{air\_entrap}\right) + B_{air} = 1$$

TAF must be less than or equal to 1

Slab Spans

$$b_{0s} \equiv \left(0.0498\cdot d_b\right) - 0.336ft = -0.1\,ft$$

$$b_{1s} \equiv \left(5.29\cdot d_b\right) + 20.3ft = 43.5\,ft$$

$$b_{2s} \equiv \left(-0.0074\cdot d_b\right) + 0.0678ft = 0\,ft$$

$$b_{3s} \equiv \left(-30.43\cdot d_b\right) + 22ft = -111.3ft$$

$$b_{4s} \equiv \left(0.00009\cdot d_b\right) - 0.001ft = -6.1\times 10^{-4}ft$$

$$b_{5s} \equiv \left[-0.0147\left[\frac{1}{ft}\cdot\left(d_b^{\,2}\right)\right]\right] - \left(0.352\cdot d_b\right) + 2.16ft = 0.3\,ft$$

$$b_{6s} \equiv \left(5.6\cdot d_b\right) + 10.2ft = 34.7ft$$

$$TAF_s \equiv 1$$

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Mobile Bridge Storm Surge

Calculation

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Use the following values based on bridge type

$$b_0 \equiv \mathrm{if}\left(B_{type} = 1, b_{0g}, b_{0s}\right) = -1\,\mathrm{ft}$$

$$b_1 \equiv \mathrm{if}\left(B_{type} = 1, b_{1g}, b_{1s}\right) = 56\,\mathrm{ft}$$

$$b_2 \equiv \mathrm{if}\left(B_{type} = 1, b_{2g}, b_{2s}\right) = 0.1\,\mathrm{ft}$$

$$b_3 \equiv \mathrm{if}\left(B_{type} = 1, b_{3g}, b_{3s}\right) = -192.7\,\mathrm{ft}$$

$$b_4 \equiv \mathrm{if}\left(B_{type} = 1, b_{4g}, b_{4s}\right) = -5.2 \times 10^{-4}\,\mathrm{ft}$$

$$b_5 \equiv \mathrm{if}\left(B_{type} = 1, b_{4g}, b_{4s}\right) = -5.2 \times 10^{-4}\,\mathrm{ft}$$

$$b_6 \equiv \mathrm{if}\left(B_{type} = 1, b_{6g}, b_{6s}\right) = 9.4\,\mathrm{ft}$$

$$TAF \equiv \mathrm{if}\left(B_{type} = 1, TAF_g, TAF_s\right) = 1$$

$$c_1 \equiv -1.3 \cdot \left(\frac{H_{max}}{d_s}\right) + 1.8 = 1$$

$$c_2 \equiv 1.35 + \left[0.35 \cdot \tanh\left[1.2 \cdot \left[T_p \cdot \left(\frac{1}{sec}\right)\right] - 8.5\right]\right] = 1$$

$$c_3 \equiv b_0 + \left(b_1 \cdot x\right) + \left(\frac{b_2}{y}\right) + \left[b_3 \cdot \left(x^2\right)\right] + \left[\frac{b_4}{\left(y^2\right)}\right] + \left[\frac{\left(b_5 \cdot x\right)}{y}\right] + \left[b_6 \cdot \left(x^3\right)\right] = 2.3\,\mathrm{ft}$$

$$F_{V\_max} \equiv \gamma_w \cdot W_{bar} \cdot \beta \cdot \left(c_1\right) \cdot \left(c_2\right) \cdot \left(c_3\right) \cdot TAF = 6.6 \cdot \frac{kip}{ft}$$

Max quasi-static vertical force $\boxed{F_{V\_max} = 6.6 \cdot \frac{kip}{ft}}$

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Mobile Bridge Storm Surge

Calculation

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Associated Vertical Slamming Force (Fs)

$$B \equiv \left[0.6588\left[\left(\frac{Z_c}{\eta_{max}}\right)^2\right]\right] + \left[0.5368\left(\frac{Z_c}{\eta_{max}}\right)\right] - 1.193 = -1$$

$$\text{If } 0 \le \frac{Z_c}{\eta_{\max}} < 1, \text{then } A = 0.0149\left(\frac{Z_c}{\eta_{\max}}\right) + 0.0316$$

(6.1.2.2.2-3)

otherwise,

$$\text{If } \frac{Z_c}{\eta_{\max}} < 0, \text{then } A = \left[-1562.9 + 1594.5e^{-\left(\frac{Z_c}{\eta_{\max}}\right)}\right]^{-1}$$

(6.1.2.2.2-4)

$$\text{ratio}_{Zc\_\eta max} \equiv \frac{Z_c}{\eta_{max}} = 0.3$$

$$A_{0to1} \equiv \left[0.0149\cdot\left(\frac{Z_c}{\eta_{max}}\right) + 0.0316\right] = 0$$

$$A_{otherwise} \equiv \left[-1562.9 + \left[1594.5\cdot e^{-\left(\frac{Z_c}{\eta_{max}}\right)}\right]\right]^{-1} = -2.9\times 10^{-3}$$

$$A_{sf} \equiv \text{if}\left(\text{ratio}_{Zc\_\eta max} < 0, A_{otherwise}, A_{0to1}\right) = 0$$

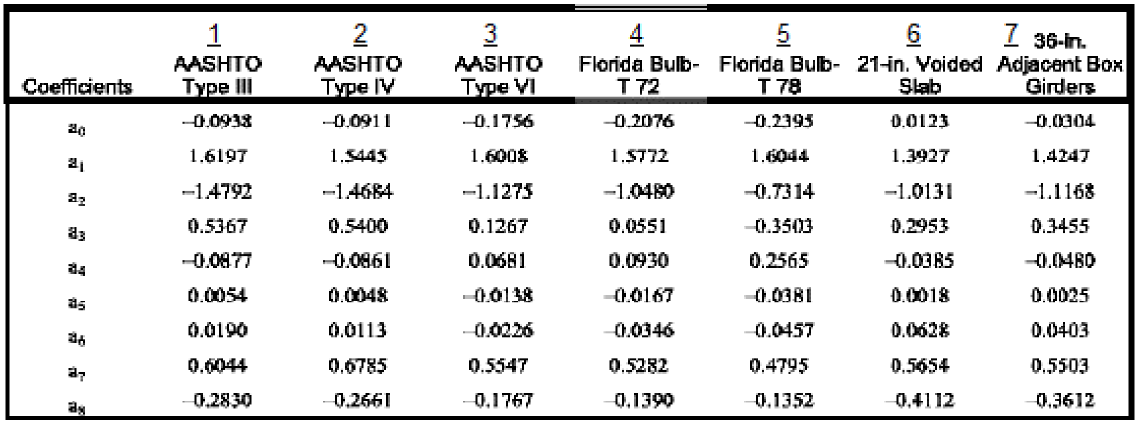
$$F_s \equiv A_{sf}\cdot\gamma_w\cdot\left(H_{max}^{\ 2}\right)\left[\left(\frac{H_{max}}{\lambda}\right)^B\right] = 3.7\cdot\frac{kip}{ft}$$

Vertical slamming force $\boxed{F_s = 3.7\cdot\frac{kip}{ft}}$

Associated Horizontal Quasi-Static Wave Force (FH-AV)

Select girder type (1-7) from table below $\quad$ girder ≡ 1

Table 6.1.2.2.3-1—Coefficients for Quasi-Static Horizontal Load



Coefficients	1 AASHTO Type III	2 AASHTO Type IV	3 AASHTO Type VI	4 Florida Bulb-T 72	5 Florida Bulb-T 78	6 21-in. Voided Slab	7 36-in. Adjacent Box Girders
a_0	−0.0938	−0.0911	−0.1756	−0.2076	−0.2395	0.0123	−0.0304
a_1	1.6197	1.5445	1.6008	1.5772	1.6044	1.3927	1.4247
a_2	−1.4792	−1.4684	−1.1275	−1.0480	−0.7314	−1.0131	−1.1168
a_3	0.5367	0.5400	0.1267	0.0551	−0.3503	0.2953	0.3455
a_4	−0.0877	−0.0861	0.0681	0.0930	0.2565	−0.0385	−0.0480
a_5	0.0054	0.0048	−0.0138	−0.0167	−0.0381	0.0018	0.0025
a_6	0.0190	0.0113	−0.0226	−0.0346	−0.0457	0.0628	0.0403
a_7	0.6044	0.6785	0.5547	0.5282	0.4795	0.5654	0.5503
a_8	−0.2830	−0.2661	−0.1767	−0.1390	−0.1352	−0.4112	−0.3612

▶ CoefficientsBased on Bride Girder Type

from chart above enter a0 through a8 below

$\mathrm{coeffa}_0 = -0.1$

$a_1 = 1.6 \qquad a_5 = 5.4 \times 10^{-3}$

$a_2 = -1.5 \qquad a_6 = 0$

$a_3 = 0.5 \qquad a_7 = 0.6$

$a_4 = -0.1 \qquad a_8 = -0.3$

$$x_H \equiv \frac{(\eta_{max} - Z_c)}{(d_b + r)} = 0.7$$

$$y_H \equiv \frac{H_{max}}{\lambda} = 0.1$$

$$c_4 \equiv \left[\begin{array}{l}\mathrm{coeffa}_0 + (a_1 \cdot x_H) + \left[a_2 \cdot \left(x_H^{\,2}\right)\right] + \left[a_3 \cdot \left(x_H^{\,3}\right)\right] + a_4 \cdot \left(x_H^{\,4}\right) + \left[a_5 \cdot \left(x_H^{\,5}\right)\right] \ldots \\ + \left(a_6 \cdot \ln(y_H)\right)\end{array}\right] = 0.4$$

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Mobile Bridge Storm Surge

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$$F_{H\_AV} \equiv \gamma_w \cdot \left(H_{max}{}^2\right) \cdot c_4 \cdot \left[a_7 + \left[a_8 \cdot \left(\frac{W_{given}}{\lambda}\right)\right]\right] = 1.7 \cdot \frac{kip}{ft}$$

Horizontal quasi-static wave force $\boxed{F_{H\_AV} = 1.7 \cdot \frac{kip}{ft}}$

Associated Moment about the trailing edge due to the Quasi-static and slamming forces (MTAV)

For girder spans

$$a_{mgirder} \equiv \left[-0.0125 \cdot \left(d_b + r\right) \cdot \left(\frac{1}{ft}\right)\right] + 0.926 = 0.8$$

$$b_{mgirder} \equiv \left[0.00659 \cdot \left(d_b + r\right) \cdot \left(\frac{1}{ft}\right)\right] - 0.0924 = -0$$

$$c_{mgirder} \equiv \left[-0.000234 \cdot \left(d_b + r\right) \cdot \left(\frac{1}{ft}\right)\right] - 0.0031 = -5 \times 10^{-3}$$

$$W_{stargirder} \equiv W_{given} - \left(2 \cdot W_{prime}\right) = 37.8\,ft$$

For slab spans

$$a_{mslab} \equiv \left[0.0049 \cdot \left(d_b + r\right) \cdot \left(\frac{1}{ft}\right)\right] + 0.805 = 0.8$$

$$b_{mslab} \equiv \left[-0.001 \cdot \left(d_b + r\right) \cdot \left(\frac{1}{ft}\right)\right] - 0.0367 = -0$$

$$c_{mslab} \equiv -0.0049$$

$$W_{starslab} \equiv W_{given} = 42.9\,ft$$

Use following Coefficients based on Span type

$$a_m \equiv \text{if}\left(B_{type} = 1, a_{mgirder}, a_{mslab}\right) = 0.8$$

$$b_m \equiv \text{if}\left(B_{type} = 1, b_{mgirder}, b_{mslab}\right) = -0$$

$$c_m \equiv \text{if}\left(B_{type} = 1, c_{mgirder}, c_{mslab}\right) = -5 \times 10^{-3}$$

$$W_{star} \equiv \text{if}\left(B_{type} = 1, W_{stargirder}, W_{starslab}\right) = 37.8\,ft$$

$$c_5 \equiv \left(F_{V\_max} \cdot W_{star}\right) + \left[F_{H\_AV} \cdot \left(d_b + r\right)\right] = 2.6 \times 10^5\ \mathrm{lbf}$$

$$c_6 \equiv a_m + \left[\frac{b_m}{\left(\frac{W_{given}}{\lambda}\right)}\right] + \left[\frac{c_m}{\left(\frac{H_{max}}{\lambda}\right)}\right] = 0.6$$

$$c_7 \equiv \frac{\left(2 \cdot F_s \cdot W_{star}\right)}{3} = 9.4 \times 10^4\ \mathrm{lbf}$$

$$M_{T\_AV} \equiv \left(c_5 \cdot c_6\right) + c_7 = 257.8 \cdot \mathrm{kip}$$

Moment due to quasi-static force and associated slamming force

$$\boxed{M_{T\_AV} = 257.8 \cdot \mathrm{kip}}$$

Maximum Horizontal Wave Force and Associated Forces and Moments

Maximum Horizontal Wave Force (FHmax)

$$\text{If } \lambda - \tfrac{1}{2}\left(Z_c + \tfrac{1}{2} H_{max}\right)\left(\frac{\lambda}{H_{max}}\right) < W,$$

$$\text{then } \omega = \lambda - \tfrac{1}{2}\left(Z_c + \tfrac{1}{2} H_{max}\right)\left(\frac{\lambda}{H_{max}}\right) \qquad (6.1.2.3.1\text{-}3)$$

otherwise

$$\omega = W \qquad (6.1.2.3.1\text{-}4)$$

$$\mathrm{check1} \equiv \lambda - \left[0.5 \cdot \left[Z_c + \left(0.5 \cdot H_{max}\right)\right] \cdot \left(\frac{\lambda}{H_{max}}\right)\right] = 97\ \mathrm{ft}$$

$$\omega \equiv \mathrm{if}\left(\mathrm{check1} < W_{given}, \mathrm{check1}, W_{given}\right) = 42.9\ \mathrm{ft}$$

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Mobile Bridge Storm Surge

Calculation

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$$F_{star\_H\_max} \equiv \gamma_w \cdot \pi \cdot (d_b + r) \cdot [\omega + (0.5 \cdot H_{max})] \cdot \left(\frac{H_{max}}{\lambda}\right) = 5.8 \cdot \frac{kip}{ft}$$

$$F_{H\_max} \equiv F_{star\_H\_max} \cdot e^{\left[-3.18+\left[3.76 \cdot e^{\left(\frac{-\omega}{\lambda}\right)}\right]-\left[0.95 \cdot \left[\left[\ln\left[\frac{(\eta_{max}-Z_c)}{(d_b+r)}\right]\right]^2\right]\right]\right]} = 3.7 \cdot \frac{kip}{ft}$$

Maximum horizontal wave force $\boxed{F_{H\_max} = 3.7 \cdot \frac{kip}{ft}}$

Associated Quasi-Static Verical Force

$$\text{If } \frac{1}{2.6}\left[\lambda - \left(Z_c + \tfrac{1}{2}H_{\max}\right)\left(\frac{\lambda}{H_{\max}}\right)\right] < W,$$

then (6.1.2.3.2-3)

$$\alpha = \frac{1}{2.6}\left[\lambda - \left(Z_c + \tfrac{1}{2}H_{\max}\right)\left(\frac{\lambda}{H_{\max}}\right)\right]$$

otherwise

$$\alpha = W \qquad (6.1.2.3.2\text{-}4)$$

$$check2 \equiv \left(\frac{1}{2.6}\right) \cdot \left[\lambda - [Z_c + (0.5 \cdot H_{max})] \cdot \left(\frac{\lambda}{H_{max}}\right)\right] = 17 ft$$

$$\alpha \equiv if(check2 < W_{given}, check2, W_{given}) = 17 ft$$

$$F_{star\_V\_AH} \equiv \gamma_w \cdot \alpha \cdot (\eta_{max} - Z_c) = 2.1 \times 10^5 \frac{lb}{s^2}$$

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Mobile Bridge Storm Surge

Calculation

Subject: Span 250 100yr SLR - 100yr Storm	Prepared by: Joshua Todd	Date: 6/8/2018	Discipline: Coastal
Component: Wave Forces on Superstructure	Checked by: Katlin Walling	Date: 6/8/2018	
Calculation No: 1.0 / Rev. No. 0	Reviewed by: Victoria Curto	Date: 6/8/2018	Sheet No.: 29 of 30

$$F_{V\_AH} \equiv F_{star\_V\_AH} \cdot e^{\left[-0.30+\left[2.04 \cdot e^{\left[-9.01 \cdot \left(\frac{\alpha}{\lambda}\right)\right]}\right]-\left[0.16 \cdot \left[\left[\frac{(\eta_{max}-Z_c)}{d_b}\right]^{1.5}\right]\right]\right]} \cdot TAF = 7.9 \cdot \frac{kip}{ft}$$

Quasi-static vertical force $\boxed{F_{V\_AH} = 7.9 \cdot \frac{kip}{ft}}$

Associated Moment about Trailing Edge (M_{T_AH})

$$M_{star\_T\_AH} \equiv \left[F_{H\_max} \cdot (d_b + r)\right] + \left[\left(\frac{2}{3}\right) \cdot (F_{V\_AH} + F_s) \cdot W_{given}\right] = 362.3 \cdot kip$$

$$M_{T\_AH} \equiv M_{star\_T\_AH} \cdot 1.37 \cdot \tanh\left[\frac{d_b}{(\eta_{max} - Z_c)}\right] = 309.7 \cdot kip$$

Moment about trailing edge $\boxed{M_{T\_AH} = 309.7 \cdot kip}$

Current Loads on Superstructure

$$Area \equiv \left[(d_b + r) - (|Z_c|)\right] = 6\,ft$$

$$C_d \equiv 2.5$$

$$\rho_w \equiv 2\,\frac{slug}{ft^3}$$

$$F_{HCstart} \equiv C_d \cdot Area \cdot \left(\frac{\rho_w}{2}\right)\left(\left(U_c^{\,2}\right)\right) = 3.6 \times 10^3\,\frac{lb}{s^2}$$

Note: lb/s^2 is not the actual units units are lbf/ft

$$F_{HC} \equiv \mathrm{if}(Z_c > 0, 0, F_{HCstart}) = 0 \cdot \frac{kip}{ft}$$

Current load $F_{HC} = 0 \cdot \frac{kip}{ft}$

MOTT MACDONALD

Mobile Bridge Storm Surge

Calculation

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Component: Wave Forces on Superstructure		Checked by: Katlin Walling	Date: 6/8/2018	
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6 Conclusions

Summary of Results

Design Case 1

$$F_{V\_max} = 6.6 \cdot \frac{kip}{ft}$$

$$F_{H\_AV} = 1.7 \cdot \frac{kip}{ft}$$

$$M_{T\_AV} = 257.8 \cdot kip$$

$$F_{s} = 3.7 \cdot \frac{kip}{ft}$$

$$F_{HC} = 0 \cdot \frac{kip}{ft}$$

Design Case 2

$$F_{H\_max} = 3.7 \cdot \frac{kip}{ft}$$

$$F_{V\_AH} = 7.9 \cdot \frac{kip}{ft}$$

$$M_{T\_AH} = 309.7 \cdot kip$$

$$F_{s} = 3.7 \cdot \frac{kip}{ft}$$

$$F_{HC} = 0 \cdot \frac{kip}{ft}$$

B.2 Superstructure (Pile) Forces Example Calculation

MOTT MACDONALD

Tensaw Bridge Storm Surge

Calculation

Element: Bent 250		Prepared by: Joshua Todd	Date: 6/8/2018	Discipline: Coastal
Description: Wave Forces on Bridge Piles 100yr - 100yr SLR WSE Results		Checked by: Katlin Walling	Date: 6/8/2018	
Calculation No: 1.0	Rev. No. 0	Reviewed by: Victoria Curto	Date: 6/8/2018	Sheet No.: 1 of 10

Wave Forces on Bridge Piles

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Tensaw Bridge Storm Surge

Calculation

Subject: Bent 250		Prepared by: Joshua Todd	Date: 6/8/2018	Discipline: Coastal
Component: Wave Forces on Bridge Piles 100yr - 100yr SLR WSE Results		Checked by: Katlin Walling	Date: 6/8/2018	
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Tensaw Bridge Storm Surge

Calculation

Subject:		Prepared by:	Date:	Discipline:
Bent 250		Joshua Todd	6/8/2018	Coastal
Component:		Checked by:	Date:	
Wave Forces on Bridge Piles 100yr - 100yr SLR WSE Results		Katlin Walling	6/8/2018	
Calculation No:	Rev. No.	Reviewed by:	Date:	Sheet No.:
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1 Calculation Scope

Calculate wave forces on the piles at Bent 250 of the Existing Bridge
using methodology perscribed in the Coastal Engineering Manual (USACE, 2012)

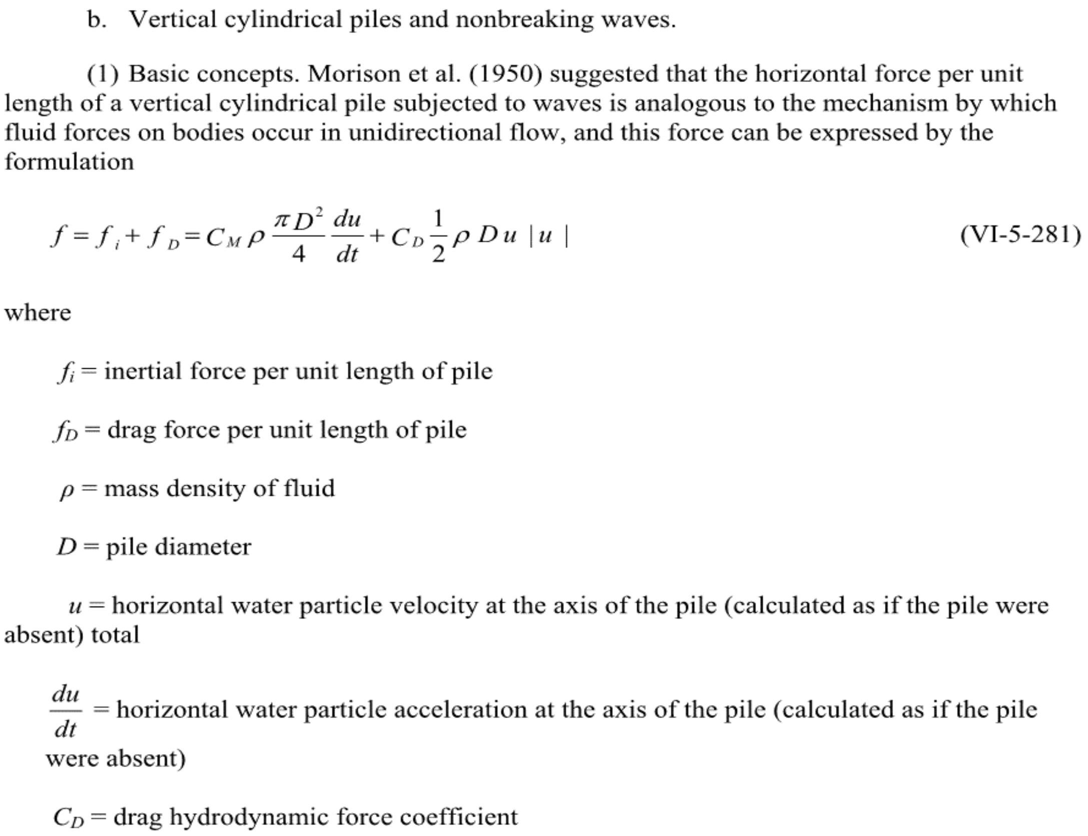
2 Technical Requirements

Wave forces are calculated using WSE, Hs, and Tp dervied in Sec. 6 of the Report.
For this case the calculations are done for a wave phase of 0-seconds

3 Criteria, Codes, and Standards

USACE Coastal Engineering Manual (TR 1.4.2.C.13)

4 Methodology and Assumptions



b. Vertical cylindrical piles and nonbreaking waves.

(1) Basic concepts. Morison et al. (1950) suggested that the horizontal force per unit length of a vertical cylindrical pile subjected to waves is analogous to the mechanism by which fluid forces on bodies occur in unidirectional flow, and this force can be expressed by the formulation

$$f = f_i + f_D = C_M \rho \frac{\pi D^2}{4} \frac{du}{dt} + C_D \frac{1}{2} \rho D u \,|u| \qquad \text{(VI-5-281)}$$

where

f_i = inertial force per unit length of pile

f_D = drag force per unit length of pile

ρ = mass density of fluid

D = pile diameter

u = horizontal water particle velocity at the axis of the pile (calculated as if the pile were absent) total

$\frac{du}{dt}$ = horizontal water particle acceleration at the axis of the pile (calculated as if the pile were absent)

C_D = drag hydrodynamic force coefficient

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Tensaw Bridge Storm Surge

Calculation

Subject:		Prepared by:	Date:	Discipline:
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Wave Forces on Bridge Piles 100yr - 100yr SLR WSE Results		Katlin Walling	6/8/2018	
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5 Computations and Results

INPUT DATA

Given from Original Design of Bridge

Wave Parameters

H_s (significant wave height) [ft]	4.6110.95
T_p (peak wave period) [sec]	5.6
d_s (still water depth) [ft]	17.6
ρ_w (water specfic gravity) [lbf/ft^3]	64
Angle of wave attack (0 or 45 deg)	0.00

CALCULATED PARAMETERS

H_{design} [ft]	10.95
L (wavelength) [ft]	149.8
ω (2π/T) [s^{-1}]	1.13
k (2π/L) [ft^{-1}]	0.04
H_{design}/d	0.62

Pile Parameters

C_D (Drag Coefficient)	0.75	Projected Width of Pile [ft]	4.500
C_M (Inertia Coefficient)	1.8	Cross-sectional Area of Pile [ft^2]	15.904
Shape of pile	circular	Length over which force is calculated [ft]	20.3
D (Diameter of pile) [ft]	4.50	dz [ft]	0.4

Time [sec] (vary from 0 to T_p)	0.00
Distance along the wavelength [ft]	0
Phase (rad)	0.00

Time [sec] largest total force occurs = 0.00

OUTPUTS Using Linear Wave Theory

F_D (Drag Force)	8515	lbf
F_I (Inertia Force)	0	lbf
F_{Total} (Total Force)	8515	lbf
F_{total} location	12.55	ft

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Tensaw Bridge Storm Surge

Calculation

Subject:	Prepared by:	Date:	Discipline:
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(z+d) distance above bottom [ft	u [ft/sec]	∂u/∂t [ft/sec2	FD (lbf)	FI (lbf)	FTotal (lbf)
0.0	13.334	0.000	247.13	0.00	247.13
0.4	13.338	0.000	247.27	0.00	247.27
0.8	13.347	0.000	247.60	0.00	247.60
1.2	13.364	0.000	248.23	0.00	248.23
1.7	13.389	0.000	249.16	0.00	249.16
2.1	13.419	0.000	250.28	0.00	250.28
2.5	13.457	0.000	251.69	0.00	251.69
2.9	13.503	0.000	253.42	0.00	253.42
3.3	13.553	0.000	255.33	0.00	255.33
3.7	13.612	0.000	257.56	0.00	257.56
4.1	13.676	0.000	259.98	0.00	259.98
4.6	13.751	0.000	262.85	0.00	262.85
5.0	13.831	0.000	265.92	0.00	265.92
5.4	13.920	0.000	269.32	0.00	269.32
5.8	14.016	0.000	273.06	0.00	273.06
6.2	14.120	0.000	277.14	0.00	277.14
6.6	14.233	0.000	281.58	0.00	281.58
7.0	14.354	0.000	286.37	0.00	286.37
7.4	14.483	0.000	291.54	0.00	291.54
7.9	14.623	0.000	297.22	0.00	297.22
8.3	14.771	0.000	303.28	0.00	303.28
8.7	14.928	0.000	309.75	0.00	309.75
9.1	15.093	0.000	316.63	0.00	316.63
9.5	15.269	0.000	324.08	0.00	324.08
9.9	15.457	0.000	332.10	0.00	332.10
10.3	15.657	0.000	340.72	0.00	340.72
10.8	15.864	0.000	349.82	0.00	349.82
11.2	16.087	0.000	359.69	0.00	359.69
11.6	16.320	0.000	370.23	0.00	370.23
12.0	16.566	0.000	381.45	0.00	381.45
12.4	16.826	0.000	393.54	0.00	393.54
12.8	17.098	0.000	406.36	0.00	406.36
13.2	17.388	0.000	420.27	0.00	420.27
13.7	17.693	0.000	435.14	0.00	435.14
14.1	18.013	0.000	451.02	0.00	451.02
14.5	18.351	0.000	468.11	0.00	468.11
14.9	18.708	0.000	486.46	0.00	486.46
15.3	19.082	0.000	506.14	0.00	506.14
15.7	19.478	0.000	527.37	0.00	527.37
16.1	19.899	0.000	550.41	0.00	550.41
16.5	20.342	0.000	575.16	0.00	575.16
17.0	20.809	0.000	601.89	0.00	601.89
17.4	21.301	0.000	630.71	0.00	630.71
17.8	21.825	0.000	662.10	0.00	662.10
18.2	22.381	0.000	696.22	0.00	696.22
18.6	22.971	0.000	733.42	0.00	733.42
19.0	23.595	0.000	773.86	0.00	773.86
19.4	24.262	0.000	818.18	0.00	818.18
19.9	24.973	0.000	866.84	0.00	866.84
20.3	25.732	0.000	920.33	0.00	920.33

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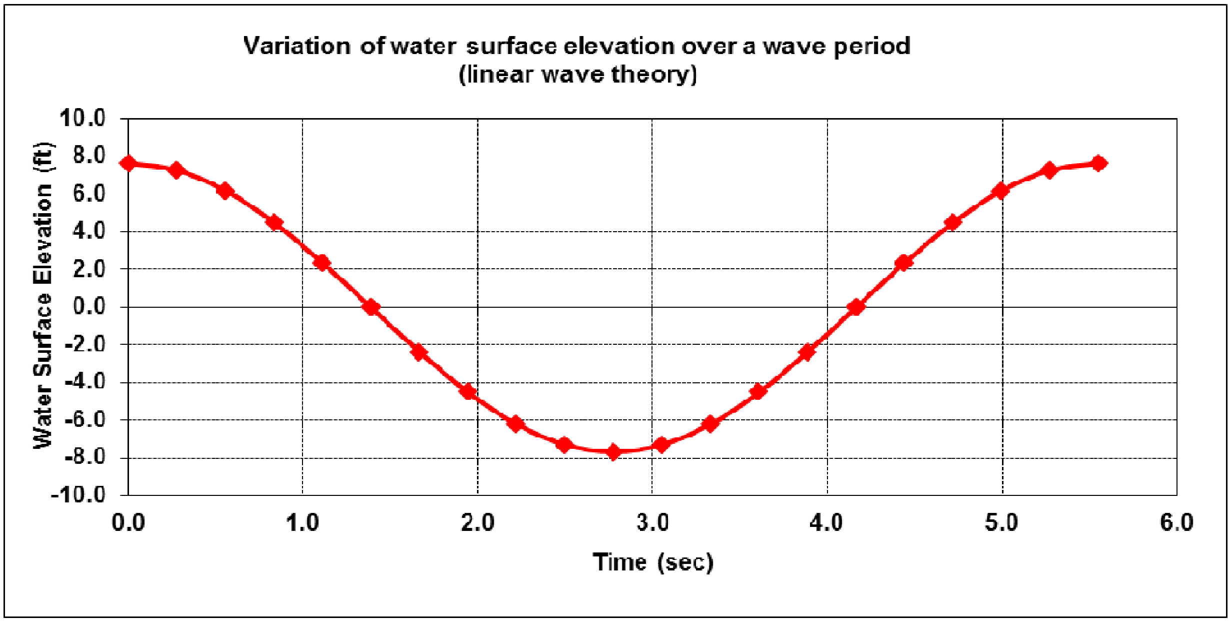

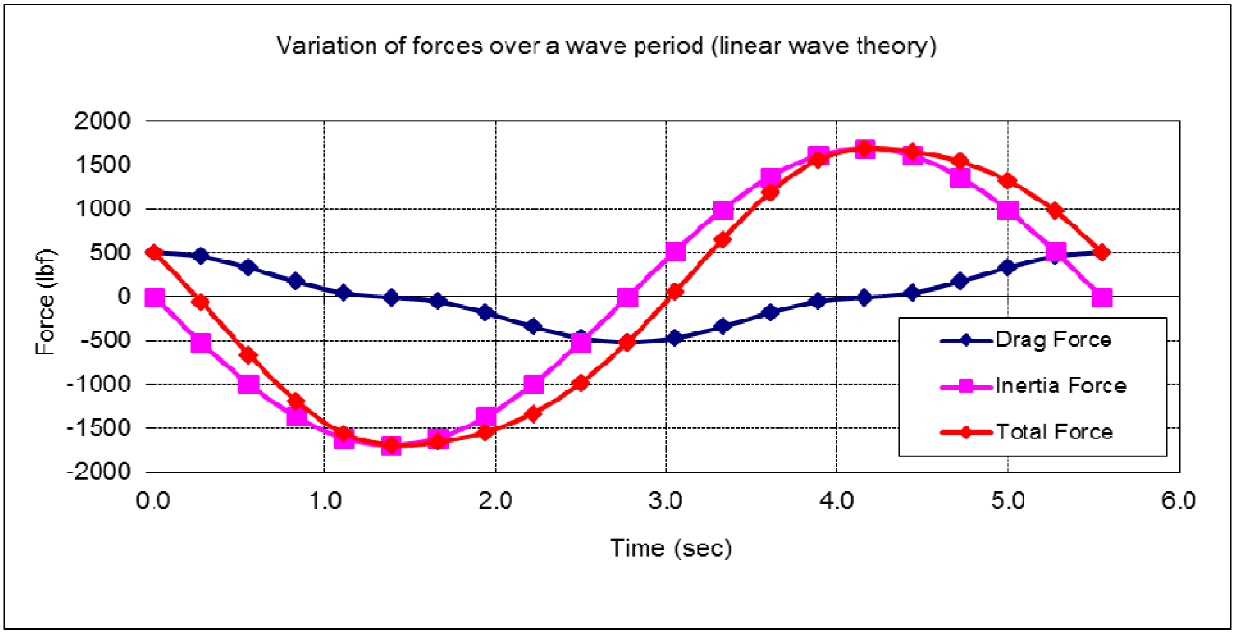

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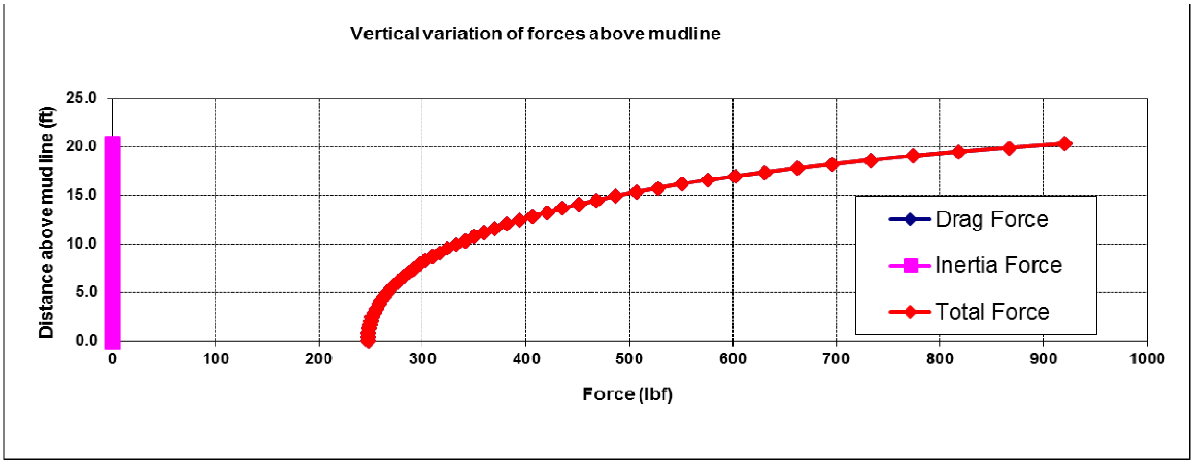

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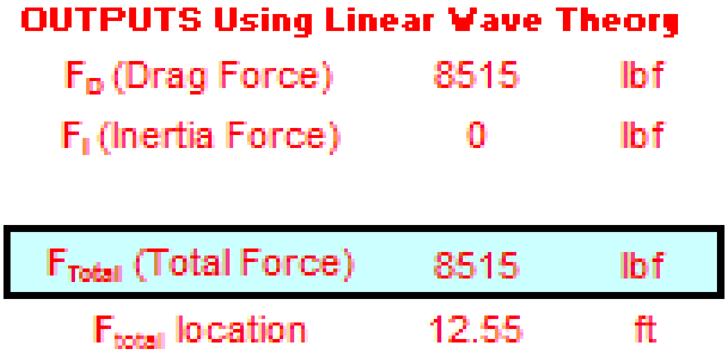
6 Conclusions



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