Assessing the Vulnerability of Delaware's Coastal Bridges to Hurricane Forces

By Dennis Mertz and Matthew Hayes

A report submitted to the University of Delaware University Transportation Center (UD-UTC)

October, 2009

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Overview

The following thesis was submitted by Matthew Hayes as partial requirements for the degree of Master of Civil Engineering. The thesis was completed under the supervision of Professor Dennis Mertz. The thesis serves as a final report for the University Transportation Center project "Assessing the Vulnerability of Delaware's Coastal Bridges to Hurricane Forces."

ASSESSING THE VULNERABILITY OF DELAWARE'S COASTAL BRIDGES TO HURRICANE FORCES

by

Matthew Brendan Hayes

A thesis submitted to the Faculty of the University of Delaware in partial fulfillment of the requirements for the degree of Master of Civil Engineering

Fall 2008

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ACKNOWLEDGMENTS

I would like to thank my adviser, Professor Dennis Mertz, as well as Jiten Soneji, Joe Krolak, and Professor Max Sheppard for their guidance. Additionally, I am grateful for the support of the University of Delaware University Transportation Center for providing me this opportunity and Brent Cooper for helping me better understand coastal engineering fundamentals.

I must also thank my friends and family, especially my fiancée Erin Baker for her continued help and understanding in all my endeavors.

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ABSTRACT

There exists a need for new guidelines to address the threat of hurricane forces to coastal bridges. Researchers at the University of Florida, Ocean Engineering Associates, Inc., Modjeski and Masters, Inc., Moffatt & Nichol, and the Federal Highway Administration have developed a three-level assessment to determine the vulnerability of coastal bridges to hurricane forces. The original research was performed in the State of Florida and is being tested in the Gulf and Atlantic Coasts. The purpose of this study is to analyze a sample of Delaware's coastal bridges to determine the applicability of the specifications to the Middle Atlantic coast and to determine any risk to Delaware's bridge inventory. Feedback will also be provided to DelDOT on the specifications and the safety of their bridges.

Three bridges in Delaware were chosen to analyze using the specifications. They are the Indian River Inlet Bridge (Bridge 3-156), the Fenwick Island Bridge (Bridge 3-437), and the Old Mill Bridge (Bridge 3-460). They were chosen because of their proximity to the coast, low elevations, and criticality in evacuation or rescue operations during a hurricane.

The results for the study were that the 100-year wave crest elevation, in addition to the design storm water elevation, was not high enough to impact any of the three bridge superstructures. In each case, the minimum 1 ft of required clearance was maintained. The risk to Delaware's coastal bridge inventory from hurricane forces is very low and it was determined that the specifications used are acceptably applicable to Delaware. Additionally, the recommendations to DelDOT are to become familiar with the specifications to use for future bridge design and to also become familiar with recovery techniques if a disaster does occur to a coastal bridge.

Chapter 1

INTRODUCTION

The purpose of the report is to discuss the vulnerability of coastal bridges to hurricane forces and to analyze the risk to Delaware's bridge inventory. This study will use procedures that were developed from extensive hurricane history and data collection in the State of Florida. The methods have been extended for application to the entire Gulf and Atlantic coasts and this study will attempt to validate the application of the procedures to other areas by analyzing Delaware. Also, the report will discuss the history of bridges that have been impacted by hurricanes, the new specifications to analyze coastal forces on bridges, and the threat posed to Delaware. In addition, it will discuss recovery techniques and provide recommendations to the Delaware Department of Transportation (DelDOT).

Problem Statement

There exists a need for new guidelines to address the threat of hurricane forces to coastal bridges after the loss of several important bridges during Hurricane Ivan and Hurricane Katrina. Traditionally, only storm surge has been accounted for when designing coastal bridges and not wave forces. Guidelines need to be determined to assess the vulnerability of existing bridges as well as address problems that may arise when designing new coastal bridges.

Due to the need for guidance on the vulnerability of coastal bridges to hurricane forces, the Florida Department of Transportation has sponsored research at the University of Florida and Ocean Engineering Associates, Inc.. They have developed a three-level vulnerability analysis procedure for Florida's coastal bridges based on experimental wave force equations. In addition, there have been efforts by the Federal Highway Administration, Moffat & Nichol, and Modjeski and Masters, Inc. to adapt the work done in Florida to the entire Gulf and Atlantic coasts. The end goal is to include the work in the American Association of State Highway and Transportation Officials' (AASHTO) specifications. Additionally, other states are performing trial assessments of their bridges to provide feedback on the practicality of the use of the procedures.

Objective

The objective of this study is to perform a trial assessment on several Delaware coastal bridges that may be vulnerable to hurricane forces. Feedback will be provided on the application of the three-level vulnerability assessment procedure to coastal bridges on the Middle Atlantic coast.

Chapter 2

BACKGROUND

The vulnerability of infrastructure to costal storms has become a national concern after several bridges in Florida, Louisiana, and Mississippi were destroyed by hurricane forces. Hurricane Ivan struck Pensacola, Florida on September 16, 2004 and severely damaged the I-10 bridge over Escambia Bay. The hurricane's 120 mph winds damaged the bridge, causing 3,400 feet of the bridge to drop into the bay (Interstate I-10 Bridge). Figure 2.1 shows the bridge during the storm and Figure 2.2 shows the damage caused to the superstructure from the hurricane.

Figure 2.1 I-10 Bridge over Escambia Bay during Hurricane Ivan (Sheppard).

Figure 2.2 I-10 Bridge over Escambia Bay after Hurricane Ivan (Sheppard).

In addition, during Hurricane Katrina, Louisiana lost the I-10 bridge over Lake Pontchartrain and Mississippi lost two US-90 bridges. The replacement of these structures, as well as the I-10 bridge loss during Hurricane Ivan, is likely to cost in excess of $1 billion.

Bridge Design Specifications

The American Association of State Highway and Transportation Official's (AASHTO) *LRFD Bridge Design Specification* does not provide adequate guidance on hurricane wave forces. Considering the recent bridge disasters caused by hurricane forces the need for guidelines is extremely high. The specifications briefly mention, "wave action shall be considered where the development of significant wave forces may occur." Additionally, it is mentioned to refer to the *Shore Protection Manual* of the US Army as well as the *Coastal Engineering Manual*. Neither of these publications clearly dictate how to asses the vulnerability of a bridge or how to safely design a new bridge to be safe from hurricane forces.

Hurricanes

The ability of a hurricane to cause immense damage to the built environment has become an ever increasing concern since Hurricane Katrina. Hurricanes can create damage not only to homes, but to other critical parts of the infrastructure, such as bridges. The main danger to bridges during hurricanes comes from storm surge and waves. The high winds and storm surge can raise the water level and create waves that can impact the bridge superstructure and impart forces on it that it was not designed for.

According to NOAA a hurricane is, "a type of tropical cyclone, which is a generic term for a low pressure system that generally forms in the tropics. The cyclone is accompanied by thunderstorms, and in the Northern Hemisphere, a counterclockwise circulation of winds near the earth's surface." Tropical cyclones are classified as either a tropical depression, tropical storm, or hurricane based on their maximum sustained wind. The maximum sustained wind is a one minute average wind measured at 33 ft above the surface near the center of the storm. Tropical depressions have a maximum sustained wind speed of 38 mph and tropical storms have a maximum sustained wind speed between 39 and 73 mph. Hurricanes have maximum sustained wind speeds of greater than 74 mph and are classified using the Saffir-Simpson Hurricane Scale that ranges from Category 1 to Category 5. Category 1 has the lowest wind speeds, ranging from 74 to 95 mph, and Category 5 has the highest wind speeds, that are 156 mph and above. (Delaware Hurricane 5).

Storm surge is higher than expected water levels along ocean coasts and interior shorelines that is generally the result of a meteorological event. Storm surges can range over 100 miles on a shoreline, but can be greatly changed by bathymetric and topographic characteristics of the coastline (Delaware Hurricane 13). The main factor which leads to storm surge is wind and the resultant frictional stresses it imposes onto the water surface. These surface currents create subsurface currents that when in the onshore direction will begin to pile up water as it is impeded by the decreasing sea floor. Additionally, storm surge is impacted by winds parallel to the coastline, the reduction of atmospheric pressure, and wave setup.

Chapter 3

DELAWARE

Delaware is located on the Atlantic Coast in the Mid-Atlantic region of the United States and is the 49th largest state with approximately 1,982 square miles. Delaware includes 24 miles of open ocean coastline, 50 miles of inland bay shoreline and 87 miles of shoreline along the Delaware Bay estuary (Delaware Hurricane 2). Delaware is located at the northern part of the Delmarva Peninsula and is the second lowest lying state with the majority of the state below 60 ft sea level (NGVD 1929). The population of Delaware was 843,524 in 2005 and there are several popular tourist areas in southern Delaware. For example, the year-round population at Rehoboth Beach is about 1,500 people and in the summer it grows to 25,000 within the city limits (Rehoboth Beach).

Hurricane History

Areas where tropical cyclones form are referred to as tropical cyclone basins. Delaware is located in the Northern Atlantic tropical cyclone basin, which includes the North Atlantic Ocean, the Caribbean Sea, and the Gulf of Mexico. The Northern Atlantic hurricane season begins on June 1 and lasts until November 30. The hurricane season is typically strongest from August to September (Delaware Hurricane 6).

Delaware has never been struck by a tropical cyclone while maintaining hurricane intensity. Since 1791 there have been 101 tropical cyclones that have affected the state with 21 passing over Delaware. Additionally, since 1893, there have been 29 hurricanes that have passed within a 150 mile radius of Rehoboth Beach, DE (Delaware Hurricane 6). Delaware has been impacted much less than other regions along the Northern Atlantic tropical cyclone basin. Table 3.1 shows a list of hurricanes that have come within 150 miles of Rehoboth Beach, DE between 1856 and 2004 and Figure 3.1 shows their tracks (Historical Hurricane Tracks).

Table 3.1 List of hurricanes that have come within 150 miles of Rehoboth Beach, DE from 1893 to 2004 (Historical Hurricane Tracks).

YEAR	MONTH	DAY	STORM NAME	WIND SPEED (mph)	CATEGORY
1893	6	17	NOTNAMED	74.75	H1
1893	8	23	NOTNAMED	109.25	H2
1894	9	29	NOTNAMED	80.5	H1
1894	10	10	NOTNAMED	74.75	H1
1899	8	18	NOTNAMED	103.5	H2
1903	9	16	NOTNAMED	92	H1
1908	5	30	NOTNAMED	74.75	H1
1924	8	26	NOTNAMED	120.75	H3
1933	9	16	NOTNAMED	92	H1
1935	9	6	NOTNAMED	74.75	H1
1936	9	18	NOTNAMED	97.75	H2
1938	9	21	NOTNAMED	115	H3
1944	9	14	NOTNAMED	103.5	H2
1953	8	14	BARBARA	80.5	H1
1954	8	31	CAROL	97.75	H2
1954	9	11	EDNA	115	H3
1955	8	12	CONNIE	74.75	H1
1955	9	20	IONE	74.75	H1
1958	8	29	DAISY	126.5	H3
1960	9	12	DONNA	103.5	H2
1962	8	28	ALMA	74.75	H1

1967	9	16	DORIA	80.5	H1
1976	8	9	BELLE	109.25	H2
1985	9	27	GLORIA	103.5	H2
1986	8	17	CHARLEY	74.75	H1
1991	8	19	BOB	109.25	H2
1993	9	1	EMILY	115	H3
1998	8	28	BONNIE	86.25	H1
1999	9	16	FLOYD	80.5	H1

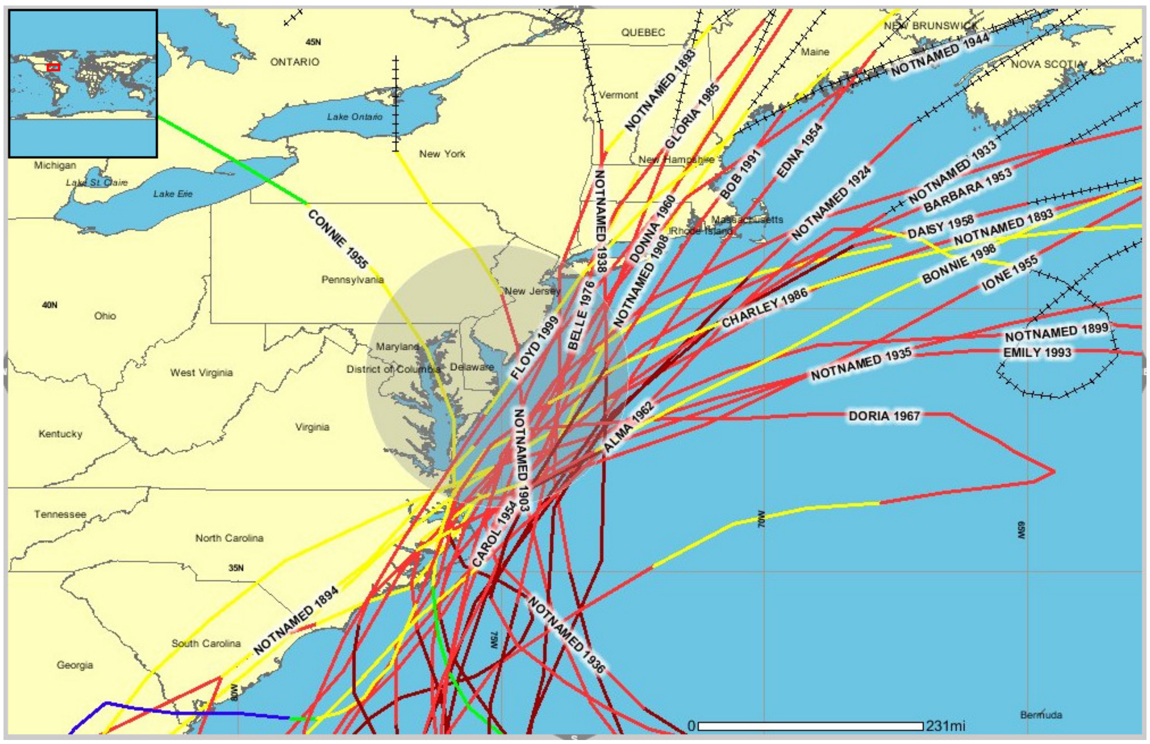

Figure 3.1 Map of hurricane tracks that have come within 150 miles of Rehoboth Beach, DE from 1893 to 2004 (Historical Hurricane Tracks).

Nor'easter

In addition to hurricanes Delaware is vulnerable to impacts by nor'easters. A nor'easter is a low pressure storm that forms along the eastern coast of the United States mainly in the winter months. It produces strong northeasterly winds, flooding, and heavy snowfall and rainfall. Nor'easters may not be able to produce winds as strong as hurricanes, but they typically are much slower moving and can impact an area for a longer time.

The worst storm to affect Delaware was a nor'easter and it occurred on March 6 to 8, 1962. It is referred to as the Ash Wednesday Storm of 1962 and lasted through five consecutive high tides. The storm produced large waves and a storm surge of 7.9 ft above mean sea level at Breakwater Harbor, Delaware, which included high tide. A nor'easter in 1991 was estimated as a 10-yr storm and caused a 4.0 ft storm tide on Little Assawoman Bay in Fenwick Island (Kobayashi).

Chapter 4

ANALYSIS METHOD

According to the *Guide Specifications for Bridges Vulnerable to Coastal Storms,* "the vertical clearance of highway bridges should be sufficient to provide at least 1 ft. of clearance over the 100-year design wave crest elevation, which includes the design storm water elevation." In the following chapters the 100-year storm surge heights at each bridge location and also the 100-year wave crest height above the storm surge will be determined and if the storm surge and waves impact the superstructure, forces on the bridge will be calculated. This chapter will discuss the approach of the analysis and chapters 6, 7, and 8 will analyze each bridge.

Level Analysis

The *Guide Specifications for Bridges Vulnerable to Coastal Storms* use a three-level analysis to analyze forces on the bridge. Level I is the simplest and least accurate method. It is typically the most conservative approach and it uses widely available information on wind speed, surge height, local wind setup, astronomical tides, bridge elevation, water depth, and fetch length. Level II uses improved data determined through simulations of the sea state. Additionally, a Level III approach uses advanced numerical simulation of the sea state, shallow depth modeling, and advanced determination of wave parameters.

For this assessment of Delaware's coastal bridges a Level I analysis will initially be performed and if needed depending on the results a higher level study will be implemented. The specifications state that when a Level I analysis gives results that produce very large demands on structures that may require expensive modification, a Level II or III analysis may reduce the loads. The Level I study is the most conservative approach because it assumes correlation between events that will cause a 100-year storm surge and 100-year wave height. In some cases this will take place, but at most bridge locations the combination of the 100-year events will not occur together. Also, input from a qualified coastal engineer shall be used for a Level I analysis to determine if the analysis was performed correctly and inputs used were appropriate. Level II and III analyses shall be conducted by a qualified coastal engineer.

Input Sources

In order to determine if a bridge is vulnerable to coastal forces site specific meteorological and oceanographic information is needed. To perform the Level I assessment the following existing information is used; 100-year design wind speed, maximum fetch length and orientation to the open coastline, 100-year storm surge elevation and the mechanisms considered in its determination, and bathymetry.

Design wind speeds will be taken from ASCE Standard 7-05 and bathymetry inputs from NOAA maps. The Coastal Engineering Department at the University of Delaware has performed studies on the coasts of Delaware and has provided valuable information concerning storm surges and fetch lengths. In addition, the United States Army Corps of Engineers has performed a *Delaware Hurricane Evacuation Study* that gives insight into how Delaware will be affected by a hurricane.

When researching sources for inputs, there were instances when different reports provided conflicting information. The most difficult input to find was the storm surge heights at the bridge locations. There were several reports that provided information for storm surge along the open coast, but they used different methods to determine the heights and thus provided different values. The most recent report by the US Army Corps of Engineers provided storm surge values based on the category of the hurricane and not for the 100-year storm. The values were very high and did not coincide well with other reports. Additionally, the way the values were presented was vague and not site specific. Other studies that gave more detailed values for storm surge at certain locations were used.

Additionally, storm surge values at the coastline were readily available, but not in areas such as Little Assawoman Bay. According to Professor Kobayashi at the University of Delaware the sea level rise in the ocean and bay is approximately the same. For bridges in Little Assawoman Bay the storm surge is assumed to be similar to the value at the coast.

Also, the majority of input sources use the National Geodetic Vertical Datum of 1929 (NGVD29). Values that were given using North American Vertical Datum 88 (NVD88) were converted to NGVD29 using information from the US Army Corps of Engineers.

Nomenclature

Figure 6.1 shows a diagram that contains part of the nomenclature used in the analysis. More detail on the nomenclature, especially for the wave calculations, can be found in Appendix A.

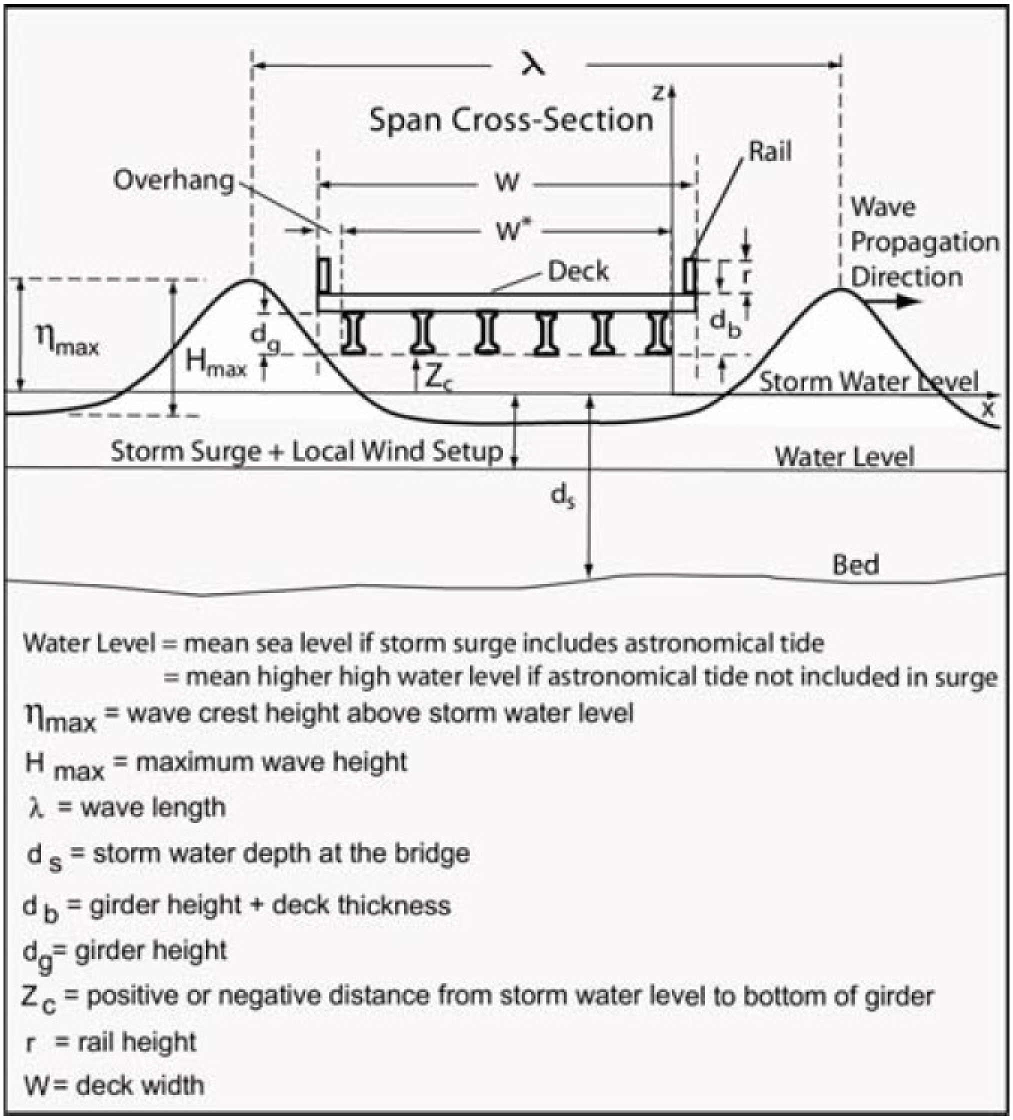

Figure 4.1 Nomenclature for wave and force equations (Guide Specifications).

Chapter 5

BRIDGE SELECTION

An initial step for assessing the vulnerability of coastal bridges to hurricane forces is to decide which bridges to analyze. In order to do this, a meeting was conducted with the chief bridge engineer for the Delaware Department of Transportation, Jiten Soneji. Mr. Soneji's familiarity with the entire Delaware bridge inventory allowed insight into bridges that can potentially be impacted by hurricanes. The initial criteria used to select bridges were it had to have a low clearance above water, be close to the coastline, and be in an area where waves can form. Also, the bridge superstructure type was considered. A bridge consisting of simple spans would be more impacted by hurricane forces than a bridge with a continuous superstructure.

Initially, 10 bridges were selected and then narrowed down using storm surge maps and more detailed information about the bathymetry and surrounding coastline. Ideally bridges would have been selected where a high storm surge could occur, there is a low clearance above water, and there is sufficient fetch length to develop waves. The majority of the bridges selected were not ideal candidates and after further review only three were selected.

Indian River Inlet Bridge

The Indian River Inlet Bridge is located between Rehoboth Beach and Bethany Beach and carries SR 1 over the Indian River Inlet. The Indian River Inlet is the only inlet along Delaware's ocean coast and it allows tidal exchange between the ocean, Indian River and Rehoboth Bays (Delaware Hurricane 4). Due to the high number of tourists that visit southern Delaware and the consistent increases in population in the surrounding areas, the bridge is extremely important. Additionally, the Indian River Inlet Bridge has issues with scour near the central piers that may be exacerbated during an intense storm. Figure 5.1 shows the location of the bridge and Figure 5.2 shows a picture of the bridge.

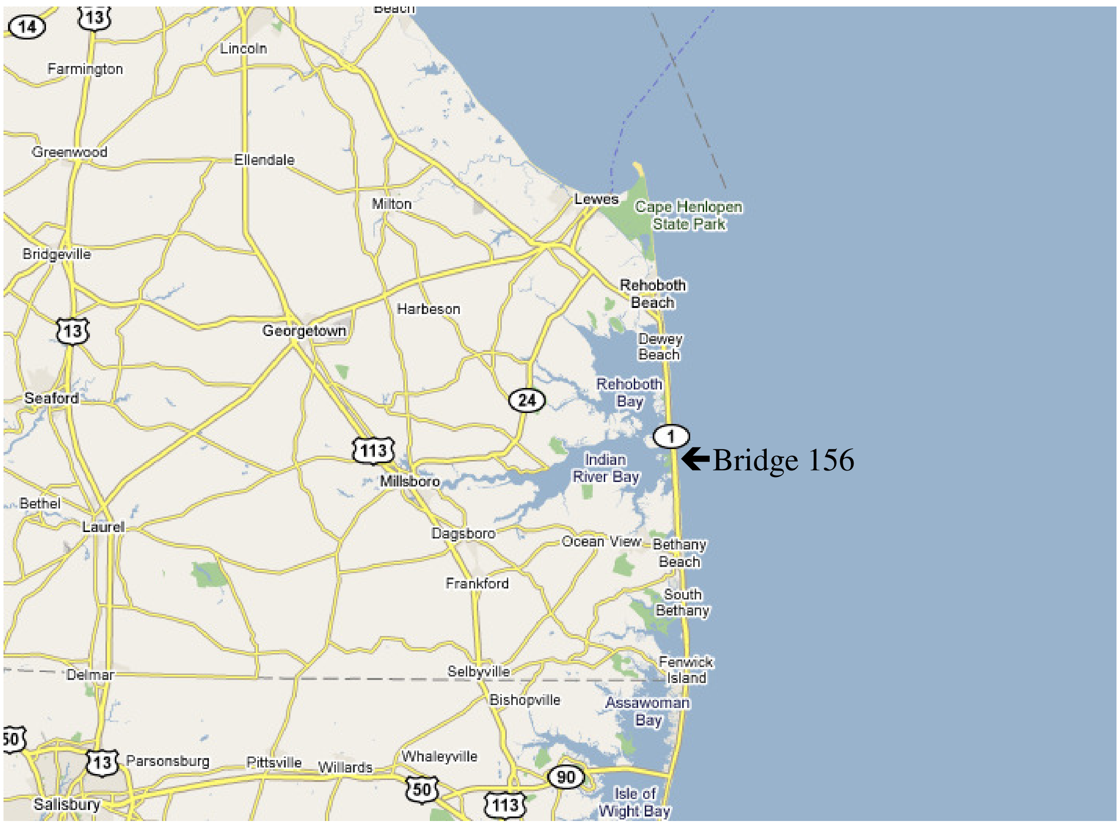

Figure 5.1 Location of Indian River Inlet Bridge.

Figure 5.2 Picture of Indian River Inlet Bridge looking east (Photo: Hayes 2008).

The Indian River Inlet Bridge, which is bridge 3-156 of the DelDOT bridge inventory, is a continuous steel haunched plate-girder bridge. It has five spans that total 874 ft, with the largest center span being 250 ft. There are four piers, two that are in the inlet and two on the shore. The bridge has separate northbound and southbound structures, each 36.5 ft wide that contain five steel girders spaced at 7ft 4 in. The girder depth at the center of the bridge is 7 ft 2 in and 8 ft 5 in at the piers that are in the inlet. The girder elevation is 35 ft above NGVD29 at the center of the bridge and 33.67 ft above NGVD29 at the piers in the inlet. Figure 5.3 shows a plan view and Figure 5.4 shows an elevation view of the Indian River Inlet Bridge.

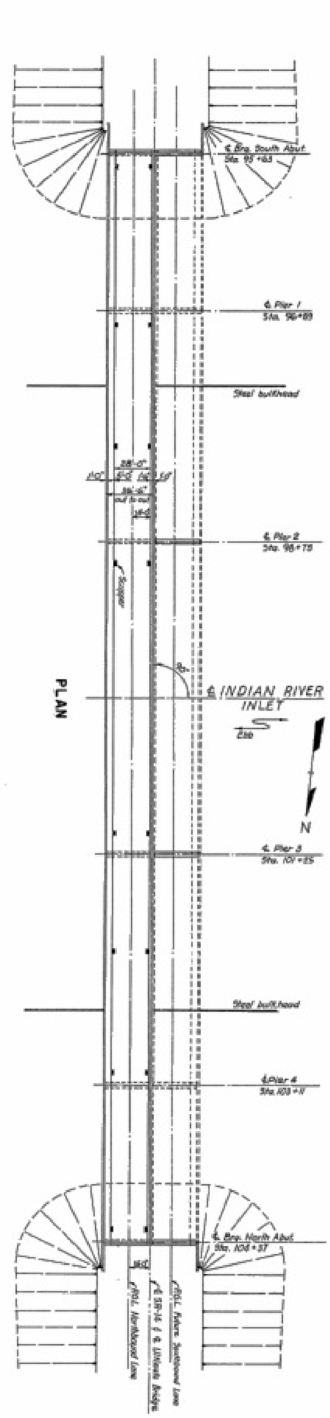

Figure 5.3 Plan view of Indian River Inlet Bridge.

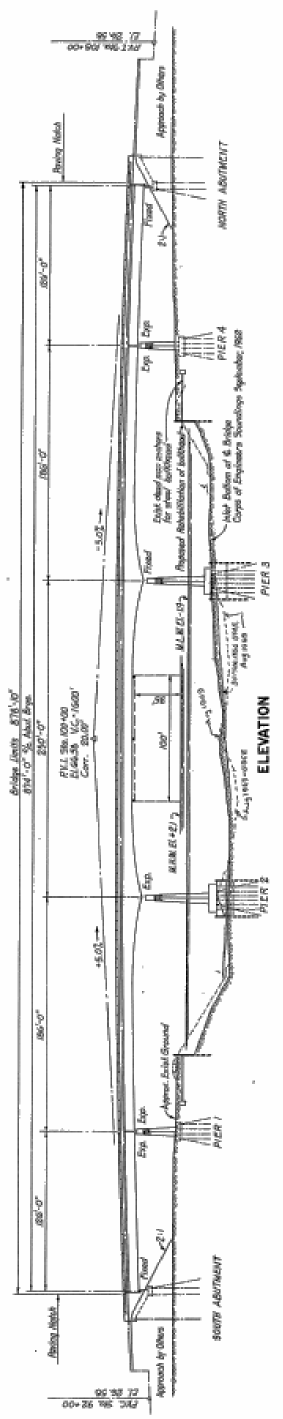

Figure 5.4 Elevation view of Indian River Inlet Bridge.

Fenwick Island Bridge

The Fenwick Island Bridge, bridge 3-437 in the DelDOT bridge inventory, carries SR 54 over a narrow strait that connects Little Assawoman Bay and Assawoman Bay in the southern part of Delaware. Assawoman Bay is a lagoon that is located between Ocean City, Maryland and the mainland of Delmarva. Little Assawoman Bay is located to the north of Assawoman Bay and is connected by the narrow strait that the Fenwick Island Bridge crosses. Figure 5.5 shows a location of the bridge and Figure 5.6 shows a picture of the bridge.

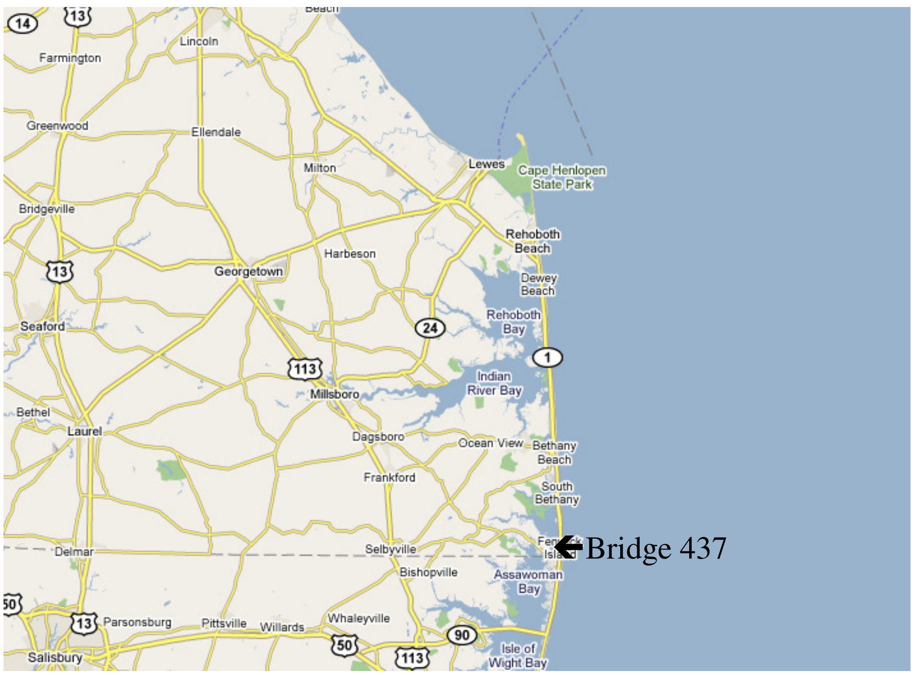

Figure 5.5 Location of Fenwick Island Bridge.

Figure 5.6 Picture of Fenwick Island Bridge looking north (Photo: Hayes 2008).

The Fenwick Island Bridge may be impacted by hurricane forces because of its proximity to the coast and low clearance above water. Although the bridge is not exposed to the open coast there is still the possibility of high storm surge and wave formation. The Fenwick Island Bridge is an important structure because in Delaware Fenwick Island can only be accessed by using the Indian River Inlet Bridge or the Fenwick Island Bridge. As previously mentioned, the Indian River Inlet Bridge may be susceptible to hurricane forces and has issues with scour near the central piers that may be worsened during an intense storm.

The bridge is 439 ft 11 in long, has 11 simply supported concrete spans with a typical span length of 40 ft, 10 piers in the water, and has a 30 ft wide superstructure carrying two lanes of traffic. The spans are pretensioned adjacent concrete box girders that are 21 in deep. Also, the elevation of the lowest span is 12.02 ft above NGVD29. Figure 5.7 shows a plan view and Figure 5.8 shows an elevation view of the bridge.

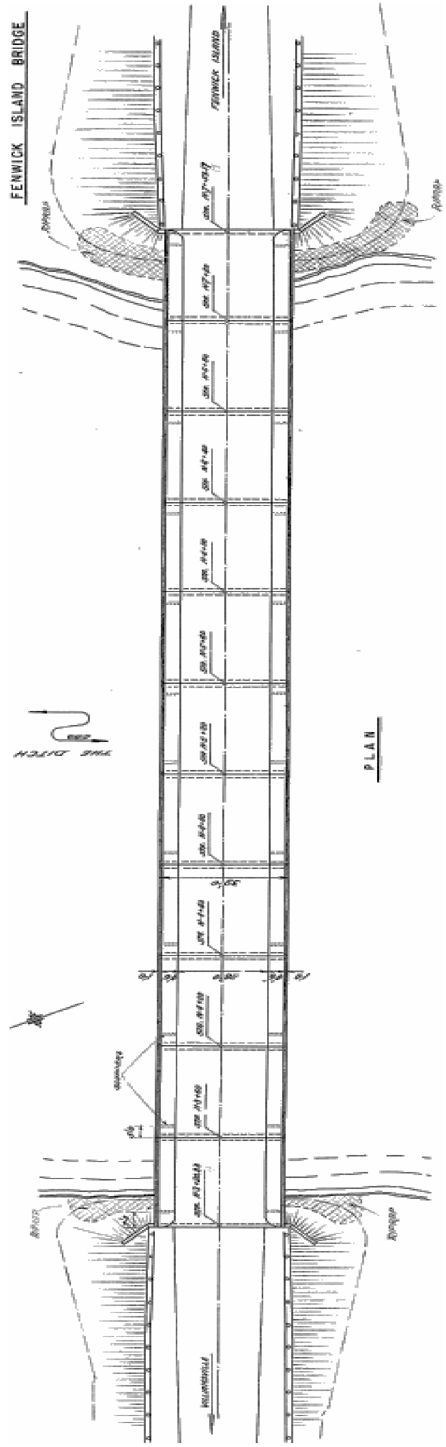

Figure 5.7 Plan view of Fenwick Island Bridge.

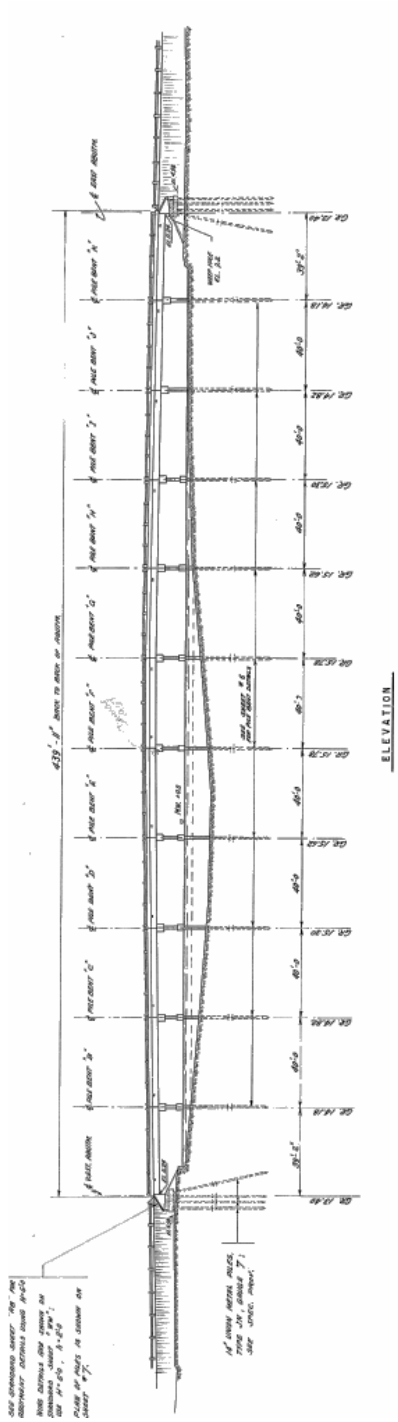

Figure 5.8 Elevation view of Fenwick Island Bridge.

Old Mill Bridge Over Dirickson Creek

Old Mill Bridge over Dirickson Creek, bridge 3-460 in the DelDOT bridge inventory, carries SR 381 over a portion of Dirickson Creek that is connected to Little Assawoman Bay. This bridge may be vulnerable to hurricane forces because of its low clearance above water. Also, this bridge is not as close to the coast as the two previously mentioned bridges, there is still the possibility of storm surge and wave formation in this location. Figure 5.9 shows the location of the bridge and Figure 5.10 shows a picture of the bridge.

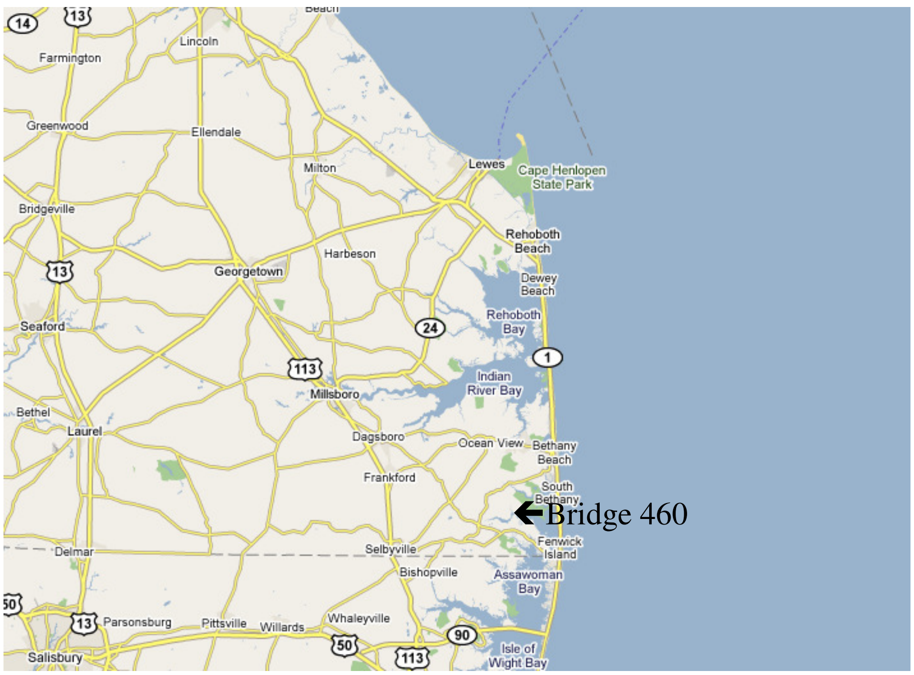

Figure 5.9 Location of Old Mill Bridge.

Figure 5.10 Picture of Old Mill Bridge looking north (Photo: Hayes 2008).

Old Mill Bridge consists of a single 37 ft 37 in span, has a width of 37 ft 1.5 in, and carries two lanes of traffic. The superstructure is comprised of 21 in deep prestressed-concrete adjacent box girders. As can be seen in Figure 5.10 the clearance above water is low and is 7 ft above NGVD29. Figure 5.11 shows a plan view and Figure 5.12 shows an elevation view of the bridge.

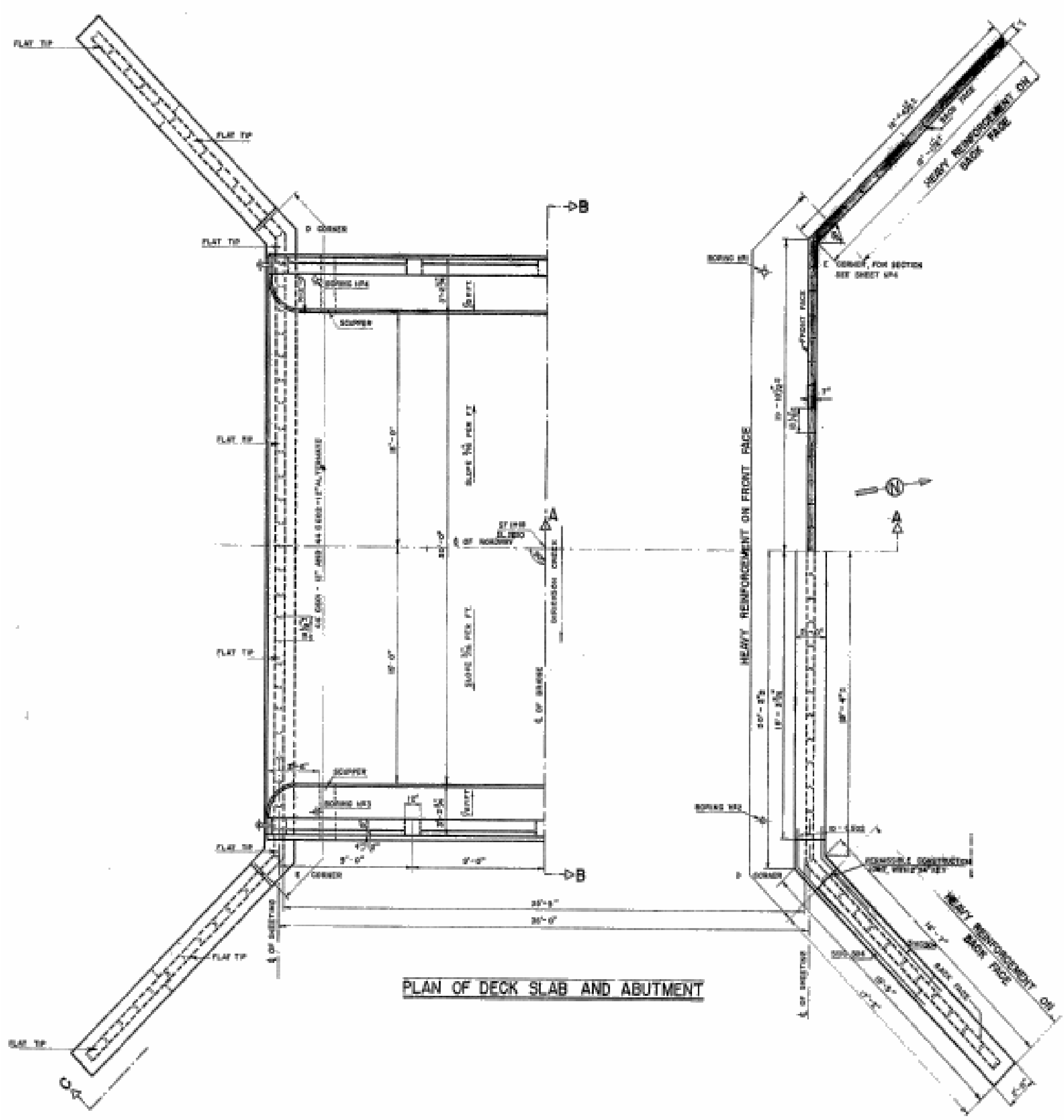

Figure 5.11 Plan view of Old Mill Bridge.

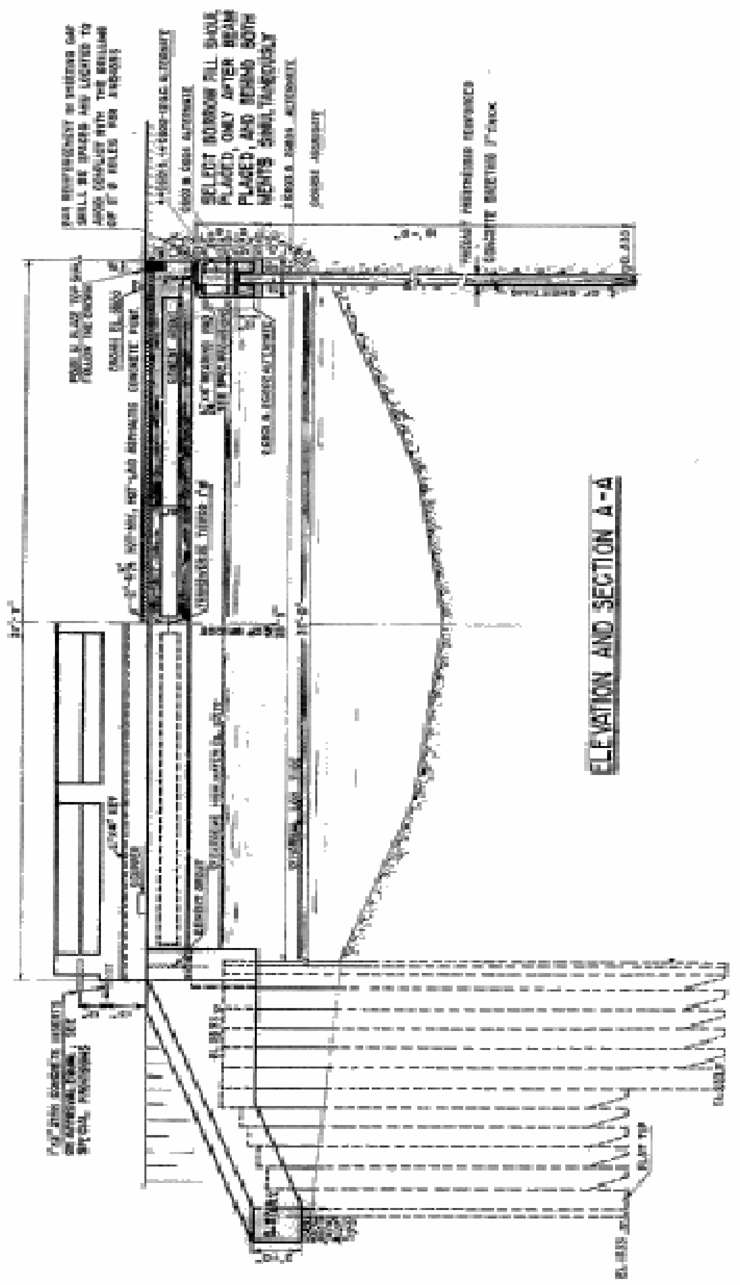

Figure 5.12 Elevation view of Old Mill Bridge.

Chapter 6

INDIAN RIVER INLET BRIDGE

This section is going to analyze the Indian River Inlet Bridge to determine if a storm surge and wave combination can impact the superstructure and cause damage during a hurricane. This bridge is in a location that will experience high storm surge and large waves during a 100-year event. It also has a high clearance above the inlet and is a continuous structure, which will make it harder for a wave to reach the bridge, and if it does the bridge may be able to better distribute the force.

Inputs

Table 6.1 shows the site specific meteorological and oceanographic inputs. They were obtained from various previously performed and widely available studies.

Table 6.1 Indian River Inlet Bridge inputs.

Input	Source	Value
Storm Surge (not including tide)	USACE	6.37 ft
50-Year Wind Speed	ASCE 7	115 mph
Water Depth for Normal High Tide at Bridge Location	DelDOT	50 ft
Fetch Length	Duke University	50530 ft
Average Water Depth over Fetch Length (including high tide and storm surge)	USACE	60 ft

Design Wave

This section is going to detail how to determine the design wave at the Indian River Inlet Bridge for a 100-year event. The equations can be found in section 6.2.2.4 in the *Guide Specifications for Bridges Vulnerable to Coastal Storms*.

The first step is to transform the 50-year design speed to a 100-year event and convert to feet per second.

$$U_{100-year} = 1.07 * U_{50-Year} \qquad \text{Eq. 1}$$

$$U_{100-Year} = 1.07 * 115 \frac{mi}{hr} * \frac{5280 \frac{ft}{mi}}{3600 \frac{\sec}{hr}} \qquad \text{Eq. 1}$$

$$U_{100-Year} = 180.47 \frac{ft}{\sec} \qquad \text{Eq. 1}$$

The next step is to determine the wind-stress factor, U^*_t, using the surface wind speed calculated in Eq. 1.

$$U_t^* = 0.539 * U_{100-Year}^{1.23} \qquad \text{Eq. 2}$$

$$U_t^* = 0.539 * 180.47^{1.23}_{\frac{ft}{\sec}} \qquad \text{Eq. 2}$$

$$U_t^* = 321.35 \frac{ft}{\sec} \qquad \text{Eq. 2}$$

Next determine the wave period, T_p.

$$T_p = 7.54 \tanh\left[0.833\left(\frac{gd}{U_T^{*2}}\right)^{3/8}\right] \tanh\left\{\frac{0.0379\left(\frac{gF}{U_t^{*2}}\right)^{1/3}}{\tanh\left[0.833\left(\frac{gd}{U_t^{*2}}\right)^{3/8}\right]}\right\}\left(\frac{U_t^*}{g}\right) \qquad \text{Eq. 3}$$

$$T_p = 7.54 \tanh\left[0.833\left(\frac{32.2\frac{ft}{\sec^2}*60\,ft}{\left(321.35\frac{ft}{\sec}\right)^2}\right)^{3/8}\right]\tanh\left\{\frac{0.0379\left(\frac{32.2\frac{ft}{\sec^2}*50530\,ft}{\left(321.35\frac{ft}{\sec}\right)^2}\right)^{1/3}}{\tanh\left[0.833\left(\frac{32.2\frac{ft}{\sec^2}*60\,ft}{\left(321.35\frac{ft}{\sec}\right)^2}\right)^{3/8}\right]}\right\}$$

$$\left(\frac{321.35\frac{ft}{\sec}}{32.2\frac{ft}{\sec^2}}\right)$$

Eq. 3

$$T_p = 6.58\sec$$ Eq. 3

Determine the time duration required to develop a fetch limited wave.

$$t = 537\left(\frac{gT_p}{U_t^*}\right)^{7/3}\left(\frac{U_t^*}{g}\right)$$ Eq. 4

$$t = 537\left(\frac{32.2\frac{ft}{\sec^2}*6.58\sec}{321.35\frac{ft}{\sec}}\right)^{7/3}\left(\frac{321.35\frac{ft}{\sec}}{32.2\frac{ft}{\sec^2}}\right)$$ Eq. 4

$$t = 2025.46\sec$$ Eq. 4

Adjust the surface wind speed, U_t, from its base duration of 3 seconds from ASCE 7-05 to a one-hour average wind speed and then from a one-hour duration to duration t. The value of t should converge and the associated value of T_p calculated.

$$U_{1-Hour} = \frac{U_t}{1.277 + 0.296\tanh\left(0.9\log\left(\frac{45}{t}\right)\right)}$$ Eq. 5

$$U_{1-Hour} = \frac{180.5\frac{ft}{\sec}}{1.277 + 0.296\tanh\left(0.9\log\left(\frac{45}{3\sec}\right)\right)}$$ Eq. 5

$$U_{1-Hour} = 119.57\sec$$ Eq. 5

First Iteration:

$$U_t = U_{1-Hour} * \left(1.277 + 0.296\tanh\left(0.9\log\left(\frac{45}{t}\right)\right)\right)$$ Eq. 5

$$U_t = 119.57\sec * \left(1.277 + 0.296\tanh\left(0.9\log\left(\frac{45}{2025.46\sec}\right)\right)\right)$$ Eq. 5

$$U_t = 120.73\frac{ft}{\sec}$$ Eq. 5

Second Iteration:

$$U_t^* = 0.539 * U_t^{1.23}$$ Eq. 2

$$U_t^* = 0.539 * 120.73^{1.23}_{\frac{ft}{\sec}}$$ Eq. 2

$$U_t^* = 195.98\frac{ft}{\sec}$$ Eq. 2

$$T_p = 7.54\tanh\left[0.833\left(\frac{gd}{U_T^{*2}}\right)^{3/8}\right]\tanh\left\{\frac{0.0379\left(\frac{gF}{U_t^{*2}}\right)^{1/3}}{\tanh\left[0.833\left(\frac{gd}{U_t^{*2}}\right)^{3/8}\right]}\right\}\left(\frac{U_t^*}{g}\right)$$ Eq. 3

$$T_p = 7.54 \tanh\left[0.833\left(\frac{32.2\frac{ft}{\sec^2} * 60\,ft}{\left(195.98\frac{ft}{\sec}\right)^2}\right)^{3/8}\right] \tanh\left\{\frac{0.0379\left(\frac{32.2\frac{ft}{\sec^2} * 50530\,ft}{\left(195.98\frac{ft}{\sec}\right)^2}\right)^{1/3}}{\tanh\left[0.833\left(\frac{32.2\frac{ft}{\sec^2} * 60\,ft}{\left(195.98\frac{ft}{\sec}\right)^2}\right)^{3/8}\right]}\right\}$$

$$\left(\frac{195.98\frac{ft}{\sec}}{32.2\frac{ft}{\sec^2}}\right)$$

Eq. 3

$T_p = 5.60\sec$ Eq. 3

$t = 537\left(\frac{gT_p}{U_t^*}\right)^{7/3}\left(\frac{U_t^*}{g}\right)$ Eq. 4

$$t = 537\left(\frac{32.2\frac{ft}{\sec^2} * 5.60\sec}{195.98\frac{ft}{\sec}}\right)^{7/3}\left(\frac{195.98\frac{ft}{\sec}}{32.2\frac{ft}{\sec^2}}\right)$$ Eq. 4

$t = 2691.29\sec$ Eq. 4

Third Iteration:

$$U_t = U_{1-Hour} * \left(1.277 + 0.296\tanh\left(0.9\log\left(\frac{45}{t}\right)\right)\right)$$ Eq. 5

$$U_t = 119.57\sec * \left(1.277 + 0.296\tanh\left(0.9\log\left(\frac{45}{2691.29\sec}\right)\right)\right)$$ Eq. 5

$U_t = 120.07\frac{ft}{\sec}$ Eq. 5

$U_t^* = 0.539 * U_t^{1.23}$ Eq. 2

$U_t^* = 0.539 * 120.07^{1.23}_{\frac{ft}{\sec}}$ Eq. 2

$U_t^* = 194.67 \frac{ft}{\sec}$ Eq. 2

$$T_p = 7.54 \tanh\left[0.833\left(\frac{gd}{U_T^{*2}}\right)^{3/8}\right] \tanh\left\{\frac{0.0379\left(\frac{gF}{U_t^{*2}}\right)^{1/3}}{\tanh\left[0.833\left(\frac{gd}{U_t^{*2}}\right)^{3/8}\right]}\right\}\left(\frac{U_t^*}{g}\right) \quad \text{Eq. 3}$$

$$T_p = 7.54 \tanh\left[0.833\left(\frac{32.2\frac{ft}{\sec^2} * 60\, ft}{\left(194.67\frac{ft}{\sec}\right)^2}\right)^{3/8}\right] \tanh\left\{\frac{0.0379\left(\frac{32.2\frac{ft}{\sec^2} * 50530\, ft}{\left(194.67\frac{ft}{\sec}\right)^2}\right)^{1/3}}{\tanh\left[0.833\left(\frac{32.2\frac{ft}{\sec^2} * 60\, ft}{\left(194.67\frac{ft}{\sec}\right)^2}\right)^{3/8}\right]}\right\}\left(\frac{194.67\frac{ft}{\sec}}{32.2\frac{ft}{\sec^2}}\right)$$

Eq. 3

$T_p = 5.59 \sec$ Eq. 3

$t = 537\left(\frac{gT_p}{U_t^*}\right)^{7/3}\left(\frac{U_t^*}{g}\right)$ Eq. 4

$$t = 537\left(\frac{32.2\frac{ft}{\sec^2}*5.59\sec}{194.67\frac{ft}{\sec}}\right)^{7/3}\left(\frac{194.67\frac{ft}{\sec}}{32.2\frac{ft}{\sec^2}}\right) \qquad \text{Eq. 4}$$

$$t = 2701.62\sec \qquad \text{Eq. 4}$$

Fourth Iteration:

$$U_t = U_{1-Hour} * \left(1.277 + 0.296\tanh\left(0.9\log\left(\frac{45}{t}\right)\right)\right) \qquad \text{Eq. 5}$$

$$U_t = 119.57\sec*\left(1.277 + 0.296\tanh\left(0.9\log\left(\frac{45}{2701.62\sec}\right)\right)\right) \qquad \text{Eq. 5}$$

$$U_t = 120.07\frac{ft}{\sec} \qquad \text{Eq. 5}$$

$$U_t^* = 0.539 * U_t^{1.23} \qquad \text{Eq. 2}$$

$$U_t^* = 0.539 * 120.07^{1.23}_{\frac{ft}{\sec}} \qquad \text{Eq. 2}$$

$$U_t^* = 194.66\frac{ft}{\sec} \qquad \text{Eq. 2}$$

$$T_p = 7.54\tanh\left[0.833\left(\frac{gd}{U_T^{*2}}\right)^{3/8}\right]\tanh\left\{\frac{0.0379\left(\frac{gF}{U_t^{*2}}\right)^{1/3}}{\tanh\left[0.833\left(\frac{gd}{U_t^{*2}}\right)^{3/8}\right]}\right\}\left(\frac{U_t^*}{g}\right) \qquad \text{Eq. 3}$$

$$T_p = 7.54\tanh\left[0.833\left(\frac{32.2\frac{ft}{\sec^2}*60\,ft}{\left(194.66\frac{ft}{\sec}\right)^2}\right)^{3/8}\right]\tanh\left\{\frac{0.0379\left(\frac{32.2\frac{ft}{\sec^2}*50530\,ft}{\left(194.66\frac{ft}{\sec}\right)^2}\right)^{1/3}}{\tanh\left[0.833\left(\frac{32.2\frac{ft}{\sec^2}*60\,ft}{\left(194.66\frac{ft}{\sec}\right)^2}\right)^{3/8}\right]}\right\}$$

$$\left(\frac{194.66\frac{ft}{\sec}}{32.2\frac{ft}{\sec^2}}\right)$$

Eq. 3

$T_p = 5.59\sec$ Eq. 3

$t = 537\left(\frac{gT_p}{U_t^*}\right)^{7/3}\left(\frac{U_t^*}{g}\right)$ Eq. 4

$$t = 537\left(\frac{32.2\frac{ft}{\sec^2}*5.59\sec}{194.66\frac{ft}{\sec}}\right)^{7/3}\left(\frac{194.66\frac{ft}{\sec}}{32.2\frac{ft}{\sec^2}}\right)$$ Eq. 4

$t = 2701.75\sec$ Eq. 4

The duration of U_t has converged to 2701.75 sec and now the remaining wave characteristics can be determined. The following calculations will determine the significant wave height, maximum wave height, and the wave length.

Determine the significant wave height, H_s.

$$H_s = 0.283\tanh\left[0.53\left(\frac{gd}{U_t^{*2}}\right)^{3/4}\right]\tanh\left\{\frac{0.00565\left(\frac{gF}{U_t^{*2}}\right)^{1/2}}{\tanh\left[0.53\left(\frac{gd}{U_t^{*2}}\right)^{3/4}\right]}\right\}\left(\frac{U_t^{*2}}{g}\right) \qquad \text{Eq. 6}$$

$$H_s = 0.283\tanh\left[0.53\left(\frac{32.2\frac{ft}{\sec^2}*60\,ft}{\left(194.66\frac{ft}{\sec}\right)^2}\right)^{3/4}\right]\tanh\left\{\frac{0.00565\left(\frac{32.2\frac{ft}{\sec^2}*50530\,ft}{\left(194.66\frac{ft}{\sec}\right)^2}\right)^{1/2}}{\tanh\left[0.53\left(\frac{32.2\frac{ft}{\sec^2}*60\,ft}{\left(194.66\frac{ft}{\sec}\right)^2}\right)^{3/4}\right]}\right\}\left(\frac{\left(194.66\frac{ft}{\sec}\right)^2}{32.2\frac{ft}{\sec^2}}\right)$$

Eq. 6

$H_s = 10.84\,ft$ Eq. 6

Determine the wave length, λ.

$$\lambda = \frac{gT_p^2}{2\pi}\sqrt{\tanh\left(\frac{4\pi^2}{T_p^2}\frac{d_s}{g}\right)} \qquad \text{Eq. 7}$$

$$\lambda = \frac{32.2\frac{ft}{\sec^2}*(5.59\sec)^2}{2\pi}\sqrt{\tanh\left(\frac{4\pi^2}{(5.59\sec)^2}\frac{56.37\,ft}{32.2\frac{ft}{\sec^2}}\right)} \qquad \text{Eq. 7}$$

$\lambda = 158.11\,ft$ Eq. 7

Determine the maximum wave height.

$$H_{\max} = 1.80 H_s \qquad \text{Eq. 8}$$

$$H_{\max} = 1.80 * 10.84 ft \qquad \text{Eq. 8}$$

$$H_{\max} = 19.51 ft \qquad \text{Eq. 8}$$

The maximum wave height, H_{max}, should be limited for depth and for steepness using the lesser of the results of Eq. 9 and Eq. 10.

$$H_{\max} \le 0.65 d_s \qquad \text{Eq. 9}$$

$$H_{\max} \le 0.65 * 56.37 ft \qquad \text{Eq. 9}$$

$$H_{\max} \le 36.64 ft \qquad \text{Eq. 9}$$

$$H_{\max} \le \frac{\lambda}{7.0} \qquad \text{Eq. 10}$$

$$H_{\max} \le \frac{158.11 ft}{7.0} \qquad \text{Eq. 10}$$

$$H_{\max} = 22.59 ft \qquad \text{Eq. 10}$$

Based on the above limitations, H_{max}, equals 19.51 ft. Next determine the assumed maximum distance from the storm water level to the design wave crest, η_{max}.

$$\eta_{\max} = 0.70 H_{\max} \qquad \text{Eq. 11}$$

$$\eta_{\max} = 0.70 * 19.51 ft \qquad \text{Eq. 11}$$

$$\eta_{\max} = 13.65 ft \qquad \text{Eq. 11}$$

In order for the wave forces to be acceptably accurate determine is the following criteria is satisfied.

$$0.035 \le \frac{H_{\max}}{\lambda} \le 0.15 \qquad \text{Eq. 12}$$

$0.035 \leq \frac{19.51 ft}{158.11 ft} \leq 0.15$ Eq. 12

$0.035 \leq 0.12 \leq 0.15$ Eq.12

$3\sec \leq T \leq 10\sec$ Eq. 13

$3\sec \leq 5.59\sec \leq 10\sec$ Eq. 13

The criteria are met.

Superstructure Clearance

To determine if the wave will contact the superstructure the wave crest height above storm water level, η_{max}, must be greater than the distance from the storm water level to the bottom of the girder, z_c. Figure 6.1 shows more detail on this scenario. For the Indian River Inlet Bridge z_c = 25.27 ft and η_{max} = 13.65 ft. In this case the wave misses the lowest part of the superstructure by 11.62 ft. This satisfies the requirement to have at least 1 ft of clearance over 100-year design wave crest elevation and no calculations will need to be performed to determine any forces on the bridge.

Chapter 7

FENWICK ISLAND BRIDGE

This section is going to analyze the Fenwick Island Bridge to determine if a storm surge and wave combination can impact the superstructure and cause damage during a hurricane. This bridge is in a location that will experience moderate storm surge, but not large waves during a 100-year event. It also has a low clearance above water and is a simply supported structure, which will make it easier for a wave to reach the bridge, and if it does the bridge will not be able to distribute the force well.

Inputs

Table 7.1 shows the site specific meteorological and oceanographic inputs. They were obtained from various previously performed and widely available studies.

Table 7.1 Fenwick Island Bridge inputs.

Input	**Source**	**Value**
Storm Surge (not including tide)	USACE	7.66 ft
50-Year Wind Speed	ASCE 7	115 mph
Water Depth for Normal High Tide at Bridge Location	NOAA	4 ft
Fetch Length	Google Earth	7000 ft
Average Water Depth over Fetch Length (including high tide and storm surge)	USACE	11.66 ft

Design Wave

This section is going to detail how to determine the design wave at the Fenwick Island Bridge for a 100-year event. The equations can be found in section 6.2.2.4 in the *Guide Specifications for Bridges Vulnerable to Coastal Storms*.

The first step is to transform the 50-year design speed to a 100-year event and convert to feet per second.

$$U_{100-year} = 1.07 * U_{50-Year} \qquad \text{Eq. 1}$$

$$U_{100-Year} = 1.07 * 115 \frac{mi}{hr} * \frac{5280 \frac{ft}{mi}}{3600 \frac{\sec}{hr}} \qquad \text{Eq. 1}$$

$$U_{100-Year} = 180.47 \frac{ft}{\sec} \qquad \text{Eq. 1}$$

The next step is to determine the wind-stress factor, $U*_t$, using the surface wind speed calculated in Eq. 1.

$$U_t^* = 0.539 * U_{100-Year}^{1.23} \qquad \text{Eq. 2}$$

$$U_t^* = 0.539 * 180.47^{1.23}_{\frac{ft}{\sec}} \qquad \text{Eq. 2}$$

$$U_t^* = 321.35 \frac{ft}{\sec} \qquad \text{Eq. 2}$$

Next determine the wave period, T_p.

$$T_p = 7.54 \tanh\left[0.833\left(\frac{gd}{U_T^{*2}}\right)^{3/8}\right] \tanh\left\{\frac{0.0379\left(\frac{gF}{U_t^{*2}}\right)^{1/3}}{\tanh\left[0.833\left(\frac{gd}{U_t^{*2}}\right)^{3/8}\right]}\right\}\left(\frac{U_t^*}{g}\right) \qquad \text{Eq. 3}$$

$$T_p = 7.54\tanh\left[0.833\left(\frac{32.2\frac{ft}{\sec^2}*11.66ft}{\left(321.35\frac{ft}{\sec}\right)^2}\right)^{3/8}\right]\tanh\left\{\frac{0.0379\left(\frac{32.2\frac{ft}{\sec^2}*7000ft}{\left(321.35\frac{ft}{\sec}\right)^2}\right)^{1/3}}{\tanh\left[0.833\left(\frac{32.2\frac{ft}{\sec^2}*11.66ft}{\left(321.35\frac{ft}{\sec}\right)^2}\right)^{3/8}\right]}\right\}\left(\frac{321.35\frac{ft}{\sec}}{32.2\frac{ft}{\sec^2}}\right)$$

Eq. 3

$T_p = 3.43\sec$ Eq. 3

Determine the time duration required to develop a fetch limited wave.

$t = 537\left(\frac{gT_p}{U_t^*}\right)^{7/3}\left(\frac{U_t^*}{g}\right)$ Eq. 4

$$t = 537\left(\frac{32.2\frac{ft}{\sec^2}*3.43\sec}{321.35\frac{ft}{\sec}}\right)^{7/3}\left(\frac{321.35\frac{ft}{\sec}}{32.2\frac{ft}{\sec^2}}\right)$$ Eq. 4

$t = 443.98\sec$ Eq. 4

Adjust the surface wind speed, U_t, from its base duration of 3 seconds from ASCE 7-05 to a one-hour average wind speed and then from a one-hour duration to duration t. The value of t should converge and the associated value of T_p calculated.

$$U_{1-Hour} = \frac{U_t}{1.277 + 0.296\tanh\left(0.9\log\left(\frac{45}{t}\right)\right)} \qquad \text{Eq. 5}$$

$$U_{1-Hour} = \frac{180.5\frac{ft}{\sec}}{1.277 + 0.296\tanh\left(0.9\log\left(\frac{45}{3\sec}\right)\right)} \qquad \text{Eq. 5}$$

$$U_{1-Hour} = 119.57\sec \qquad \text{Eq. 5}$$

First Iteration:

$$U_t = U_{1-Hour} * \left(1.277 + 0.296\tanh\left(0.9\log\left(\frac{45}{t}\right)\right)\right) \qquad \text{Eq. 5}$$

$$U_t = 119.57\sec * \left(1.277 + 0.296\tanh\left(0.9\log\left(\frac{45}{443.98\sec}\right)\right)\right) \qquad \text{Eq. 5}$$

$$U_t = 127.43\frac{ft}{\sec} \qquad \text{Eq. 5}$$

Second Iteration:

$$U_t^* = 0.539 * U_t^{1.23} \qquad \text{Eq. 2}$$

$$U_t^* = 0.539 * 127.43^{1.23}_{\frac{ft}{\sec}} \qquad \text{Eq. 2}$$

$$U_t^* = 209.44\frac{ft}{\sec} \qquad \text{Eq. 2}$$

$$T_p = 7.54\tanh\left[0.833\left(\frac{gd}{U_T^{*2}}\right)^{3/8}\right]\tanh\left\{\frac{0.0379\left(\frac{gF}{U_t^{*2}}\right)^{1/3}}{\tanh\left[0.833\left(\frac{gd}{U_t^{*2}}\right)^{3/8}\right]}\right\}\left(\frac{U_t^*}{g}\right) \qquad \text{Eq. 3}$$

$$T_p = 7.54\tanh\left[0.833\left(\frac{32.2\frac{ft}{\sec^2}*11.66\,ft}{\left(209.44\frac{ft}{\sec}\right)^2}\right)^{3/8}\right]\tanh\left\{\frac{0.0379\left(\frac{32.2\frac{ft}{\sec^2}*7000\,ft}{\left(209.44\frac{ft}{\sec}\right)^2}\right)^{1/3}}{\tanh\left[0.833\left(\frac{32.2\frac{ft}{\sec^2}*11.66\,ft}{\left(209.44\frac{ft}{\sec}\right)^2}\right)^{3/8}\right]}\right\}$$

$$\left(\frac{209.44\frac{ft}{\sec}}{32.2\frac{ft}{\sec^2}}\right)$$

Eq. 3

$T_p = 2.99\sec$ Eq. 3

$t = 537\left(\frac{gT_p}{U_t^*}\right)^{7/3}\left(\frac{U_t^*}{g}\right)$ Eq. 4

$$t = 537\left(\frac{32.2\frac{ft}{\sec^2}*2.99\sec}{209.44\frac{ft}{\sec}}\right)^{7/3}\left(\frac{209.44\frac{ft}{\sec}}{32.2\frac{ft}{\sec^2}}\right)$$ Eq. 4

$t = 568.79\sec$ Eq. 4

Third Iteration:

$$U_t = U_{1-Hour} * \left(1.277 + 0.296\tanh\left(0.9\log\left(\frac{45}{t}\right)\right)\right)$$ Eq. 5

$$U_t = 119.57\sec*\left(1.277 + 0.296\tanh\left(0.9\log\left(\frac{45}{568.79\sec}\right)\right)\right)$$ Eq. 5

$U_t = 125.86\frac{ft}{\sec}$ Eq. 5

$$U_t^* = 0.539 * U_t^{1.23} \qquad \text{Eq. 2}$$

$$U_t^* = 0.539 * 125.86^{1.23}_{\frac{ft}{\sec}} \qquad \text{Eq. 2}$$

$$U_t^* = 206.27 \frac{ft}{\sec} \qquad \text{Eq. 2}$$

$$T_p = 7.54 \tanh\left[0.833\left(\frac{gd}{U_T^{*2}}\right)^{3/8}\right] \tanh\left\{\frac{0.0379\left(\frac{gF}{U_t^{*2}}\right)^{1/3}}{\tanh\left[0.833\left(\frac{gd}{U_t^{*2}}\right)^{3/8}\right]}\right\}\left(\frac{U_t^*}{g}\right) \qquad \text{Eq. 3}$$

$$T_p = 7.54 \tanh\left[0.833\left(\frac{32.2\frac{ft}{\sec^2} * 11.66\,ft}{\left(206.27\frac{ft}{\sec}\right)^2}\right)^{3/8}\right] \tanh\left\{\frac{0.0379\left(\frac{32.2\frac{ft}{\sec^2} * 7000\,ft}{\left(206.27\frac{ft}{\sec}\right)^2}\right)^{1/3}}{\tanh\left[0.833\left(\frac{32.2\frac{ft}{\sec^2} * 11.66\,ft}{\left(206.27\frac{ft}{\sec}\right)^2}\right)^{3/8}\right]}\right\}$$

$$\left(\frac{206.27\frac{ft}{\sec}}{32.2\frac{ft}{\sec^2}}\right)$$

Eq. 3

$$T_p = 2.97 \sec \qquad \text{Eq. 3}$$

$$t = 537\left(\frac{gT_p}{U_t^*}\right)^{7/3}\left(\frac{U_t^*}{g}\right) \qquad \text{Eq. 4}$$

$$t = 537\left(\frac{32.2\frac{ft}{\sec^2} * 2.97\sec}{206.27\frac{ft}{\sec}}\right)^{7/3}\left(\frac{206.27\frac{ft}{\sec}}{32.2\frac{ft}{\sec^2}}\right) \qquad \text{Eq. 4}$$

$$t = 573.82\sec \qquad \text{Eq. 4}$$

Fourth Iteration:

$$U_t = U_{1-Hour} * \left(1.277 + 0.296 \tanh\left(0.9\log\left(\frac{45}{t}\right)\right)\right) \qquad \text{Eq. 5}$$

$$U_t = 119.57\sec * \left(1.277 + 0.296\tanh\left(0.9\log\left(\frac{45}{573.82\sec}\right)\right)\right) \qquad \text{Eq. 5}$$

$$U_t = 125.81\frac{ft}{\sec} \qquad \text{Eq. 5}$$

$$U_t^* = 0.539 * U_t^{1.23} \qquad \text{Eq. 2}$$

$$U_t^* = 0.539 * 125.81^{1.23}_{\frac{ft}{\sec}} \qquad \text{Eq. 2}$$

$$U_t^* = 206.17\frac{ft}{\sec} \qquad \text{Eq. 2}$$

$$T_p = 7.54\tanh\left[0.833\left(\frac{gd}{U_T^{*2}}\right)^{3/8}\right]\tanh\left\{\frac{0.0379\left(\frac{gF}{U_t^{*2}}\right)^{1/3}}{\tanh\left[0.833\left(\frac{gd}{U_t^{*2}}\right)^{3/8}\right]}\right\}\left(\frac{U_t^*}{g}\right) \qquad \text{Eq. 3}$$

$$T_p = 7.54\tanh\left[0.833\left(\frac{32.2\frac{ft}{\sec^2}*11.66\,ft}{\left(206.17\frac{ft}{\sec}\right)^2}\right)^{3/8}\right]\tanh\left\{\frac{0.0379\left(\frac{32.2\frac{ft}{\sec^2}*7000\,ft}{\left(206.17\frac{ft}{\sec}\right)^2}\right)^{1/3}}{\tanh\left[0.833\left(\frac{32.2\frac{ft}{\sec^2}*11.66\,ft}{\left(206.17\frac{ft}{\sec}\right)^2}\right)^{3/8}\right]}\right\}$$

$$\left(\frac{206.17\frac{ft}{\sec}}{32.2\frac{ft}{\sec^2}}\right)$$

Eq. 3

$T_p = 2.97\sec$ Eq. 3

$t = 537\left(\frac{gT_p}{U_t^*}\right)^{7/3}\left(\frac{U_t^*}{g}\right)$ Eq. 4

$$t = 537\left(\frac{32.2\frac{ft}{\sec^2}*2.97\sec}{206.17\frac{ft}{\sec}}\right)^{7/3}\left(\frac{206.17\frac{ft}{\sec}}{32.2\frac{ft}{\sec^2}}\right)$$ Eq. 4

$t = 573.98\sec$ Eq. 4

The duration of U_t has converged to 573.98 sec and now the remaining wave characteristics can be determined. The following calculations will determine the significant wave height, maximum wave height, and the wave length.

Determine the significant wave height, H_s.

$$H_s = 0.283\tanh\left[0.53\left(\frac{gd}{U_t^{*2}}\right)^{3/4}\right]\tanh\left\{\frac{0.00565\left(\frac{gF}{U_t^{*2}}\right)^{1/2}}{\tanh\left[0.53\left(\frac{gd}{U_t^{*2}}\right)^{3/4}\right]}\right\}\left(\frac{U_t^{*2}}{g}\right) \qquad \text{Eq. 6}$$

$$H_s = 0.283\tanh\left[0.53\left(\frac{32.2\frac{ft}{\sec^2}*11.66ft}{\left(206.17\frac{ft}{\sec}\right)^2}\right)^{3/4}\right]\tanh\left\{\frac{0.00565\left(\frac{32.2\frac{ft}{\sec^2}*7000ft}{\left(206.17\frac{ft}{\sec}\right)^2}\right)^{1/2}}{\tanh\left[0.53\left(\frac{32.2\frac{ft}{\sec^2}*11.66ft}{\left(206.17\frac{ft}{\sec}\right)^2}\right)^{3/4}\right]}\right\}$$

$$\left(\frac{\left(206.17\frac{ft}{\sec}\right)^2}{32.2\frac{ft}{\sec^2}}\right)$$

Eq. 6

$H_s = 3.95 ft$ Eq. 6

Determine the wave length, λ.

$$\lambda = \frac{gT_p^2}{2\pi}\sqrt{\tanh\left(\frac{4\pi^2}{T_p^2}\frac{d_s}{g}\right)} \qquad \text{Eq. 7}$$

$$\lambda = \frac{32.2\frac{ft}{\sec^2}*(2.97\sec)^2}{2\pi}\sqrt{\tanh\left(\frac{4\pi^2}{(2.97\sec)^2}\frac{11.66ft}{32.2\frac{ft}{\sec^2}}\right)} \qquad \text{Eq. 7}$$

$\lambda = 43.54 ft$ Eq. 7

Determine the maximum wave height.

$$H_{\max} = 1.80 H_s \qquad \text{Eq. 8}$$

$$H_{\max} = 1.80 * 3.95 ft \qquad \text{Eq. 8}$$

$$H_{\max} = 7.11 ft \qquad \text{Eq. 8}$$

The maximum wave height, H_{max}, should be limited for depth and for steepness using the lesser of the results of Eq. 9 and Eq. 10.

$$H_{\max} \le 0.65 d_s \qquad \text{Eq. 9}$$

$$H_{\max} \le 0.65 * 11.66 ft \qquad \text{Eq. 9}$$

$$H_{\max} \le 7.58 ft \qquad \text{Eq. 9}$$

$$H_{\max} \le \frac{\lambda}{7.0} \qquad \text{Eq. 10}$$

$$H_{\max} \le \frac{43.54 ft}{7.0} \qquad \text{Eq. 10}$$

$$H_{\max} = 6.22 ft \qquad \text{Eq. 10}$$

Based on the above limitations, H_{max}, equals 6.22 ft. Next determine the assumed maximum distance from the storm water level to the design wave crest, η_{max}.

$$\eta_{\max} = 0.70 H_{\max} \qquad \text{Eq. 11}$$

$$\eta_{\max} = 0.70 * 6.22 ft \qquad \text{Eq. 11}$$

$$\eta_{\max} = 4.35 ft \qquad \text{Eq. 11}$$

In order for the wave forces to be acceptably accurate determine is the following criteria is satisfied.

$$0.035 \le \frac{H_{\max}}{\lambda} \le 0.15 \qquad \text{Eq. 12}$$

$$0.035 \leq \frac{6.22\,ft}{43.54\,ft} \leq 0.15 \qquad \text{Eq. 12}$$

$$0.035 \leq 0.14 \leq 0.15 \qquad \text{Eq.12}$$

$$3\sec \leq T \leq 10\sec \qquad \text{Eq. 13}$$

$$3\sec \leq 2.97\sec \leq 10\sec \qquad \text{Eq. 13}$$

The criteria from Eq. 12 are met and from Eq. 13 are not. This is due to the short fetch length that can form in Little Assawoman Bay.

Superstructure Clearance

To determine if the wave will contact the superstructure the wave crest height above storm water level, η_{max}, must be greater than the distance from the storm water level to the bottom of the girder, z_c. Figure 6.1 shows more detail on this scenario. For the Fenwick Island Bridge z_c = 5.8 ft and η_{max} = 4.35 ft. In this case the wave misses the lowest part of the superstructure by 1.45 ft. This satisfies the requirement to have at least 1 ft of clearance over 100-year design wave crest elevation and no calculations will need to be performed to determine any forces on the bridge.

Chapter 8

OLD MILL BRIDGE

This section is going to analyze the Old Mill Bridge to determine if a storm surge and wave combination can impact the superstructure and cause damage during a hurricane. This bridge is in a location that will experience low storm surge and waves during a 100-year event. It also has a very low clearance above water and is a simply supported structure, which will make it easier for a wave to reach the bridge, and if it does the bridge will not be able to distribute the force well.

Inputs

Table 8.1 shows the site specific meteorological and oceanographic inputs. They were obtained from various previously performed and widely available studies.

Table 8.1 Old Mill Bridge inputs.

Input	Source	Value
Storm Surge (not including tide)	USACE	3 ft
50-Year Wind Speed	ASCE 7	115 mph
Water Depth for Normal High Tide at Bridge Location	NOAA	4 ft
Fetch Length	Google Earth	4000 ft
Average Water Depth over Fetch Length (including high tide and storm surge)	USACE	7 ft

Design Wave

This section is going to detail how to determine the design wave at the Old Mill Bridge for a 100-year event. The equations can be found in section 6.2.2.4 in the *Guide Specifications for Bridges Vulnerable to Coastal Storms*.

The first step is to transform the 50-year design speed to a 100-year event and convert to feet per second.

$$U_{100-year} = 1.07 * U_{50-Year} \qquad \text{Eq. 1}$$

$$U_{100-Year} = 1.07 * 115\frac{mi}{hr} * \frac{5280\frac{ft}{mi}}{3600\frac{\sec}{hr}} \qquad \text{Eq. 1}$$

$$U_{100-Year} = 180.47\frac{ft}{\sec} \qquad \text{Eq. 1}$$

The next step is to determine the wind-stress factor, U^*_t, using the surface wind speed calculated in Eq. 1.

$$U_t^* = 0.539 * U_{100-Year}^{1.23} \qquad \text{Eq. 2}$$

$$U_t^* = 0.539 * 180.47_{\frac{ft}{\sec}}^{1.23} \qquad \text{Eq. 2}$$

$$U_t^* = 321.35\frac{ft}{\sec} \qquad \text{Eq. 2}$$

Next determine the wave period, T_p.

$$T_p = 7.54\tanh\left[0.833\left(\frac{gd}{U_T^{*2}}\right)^{3/8}\right]\tanh\left\{\frac{0.0379\left(\frac{gF}{U_t^{*2}}\right)^{1/3}}{\tanh\left[0.833\left(\frac{gd}{U_t^{*2}}\right)^{3/8}\right]}\right\}\left(\frac{U_t^*}{g}\right) \qquad \text{Eq. 3}$$

$$T_p = 7.54 \tanh\left[0.833\left(\frac{32.2\frac{ft}{\sec^2}*7.0ft}{\left(321.35\frac{ft}{\sec}\right)^2}\right)^{3/8}\right]\tanh\left\{\frac{0.0379\left(\frac{32.2\frac{ft}{\sec^2}*4000ft}{\left(321.35\frac{ft}{\sec}\right)^2}\right)^{1/3}}{\tanh\left[0.833\left(\frac{32.2\frac{ft}{\sec^2}*7.0ft}{\left(321.35\frac{ft}{\sec}\right)^2}\right)^{3/8}\right]}\right\}$$

$$\left(\frac{321.35\frac{ft}{\sec}}{32.2\frac{ft}{\sec^2}}\right)$$

Eq. 3

$$T_p = 2.85\sec$$ Eq. 3

Determine the time duration required to develop a fetch limited wave.

$$t = 537\left(\frac{gT_p}{U_t^*}\right)^{7/3}\left(\frac{U_t^*}{g}\right)$$ Eq. 4

$$t = 537\left(\frac{32.2\frac{ft}{\sec^2}*2.85\sec}{321.35\frac{ft}{\sec}}\right)^{7/3}\left(\frac{321.35\frac{ft}{\sec}}{32.2\frac{ft}{\sec^2}}\right)$$ Eq. 4

$$t = 287.05\sec$$ Eq. 4

Adjust the surface wind speed, U_t, from its base duration of 3 seconds from ASCE 7-05 to a one-hour average wind speed and then from a one-hour duration to duration t. The value of t should converge and the associated value of T_p calculated.

$$U_{1-Hour} = \frac{U_t}{1.277 + 0.296\tanh\left(0.9\log\left(\frac{45}{t}\right)\right)} \qquad \text{Eq. 5}$$

$$U_{1-Hour} = \frac{180.5\frac{ft}{\sec}}{1.277 + 0.296\tanh\left(0.9\log\left(\frac{45}{3\sec}\right)\right)} \qquad \text{Eq. 5}$$

$$U_{1-Hour} = 119.57\sec \qquad \text{Eq. 5}$$

First Iteration:

$$U_t = U_{1-Hour} * \left(1.277 + 0.296\tanh\left(0.9\log\left(\frac{45}{t}\right)\right)\right) \qquad \text{Eq. 5}$$

$$U_t = 119.57\sec * \left(1.277 + 0.296\tanh\left(0.9\log\left(\frac{45}{287.15\sec}\right)\right)\right) \qquad \text{Eq. 5}$$

$$U_t = 130.76\frac{ft}{\sec} \qquad \text{Eq. 5}$$

Second Iteration:

$$U_t^* = 0.539 * U_t^{1.23} \qquad \text{Eq. 2}$$

$$U_t^* = 0.539 * 130.76^{1.23}_{\frac{ft}{\sec}} \qquad \text{Eq. 2}$$

$$U_t^* = 216.19\frac{ft}{\sec} \qquad \text{Eq. 2}$$

$$T_p = 7.54\tanh\left[0.833\left(\frac{gd}{U_T^{*2}}\right)^{3/8}\right]\tanh\left\{\frac{0.0379\left(\frac{gF}{U_t^{*2}}\right)^{1/3}}{\tanh\left[0.833\left(\frac{gd}{U_t^{*2}}\right)^{3/8}\right]}\right\}\left(\frac{U_t^*}{g}\right) \qquad \text{Eq. 3}$$

$$T_p = 7.54\tanh\left[0.833\left(\frac{32.2\frac{ft}{\sec^2}*7.0\,ft}{\left(216.19\frac{ft}{\sec}\right)^2}\right)^{3/8}\right]\tanh\left\{\frac{0.0379\left(\frac{32.2\frac{ft}{\sec^2}*4000\,ft}{\left(216.19\frac{ft}{\sec}\right)^2}\right)^{1/3}}{\tanh\left[0.833\left(\frac{32.2\frac{ft}{\sec^2}*7.0\,ft}{\left(216.19\frac{ft}{\sec}\right)^2}\right)^{3/8}\right]}\right\}$$

$$\left(\frac{216.19\frac{ft}{\sec}}{32.2\frac{ft}{\sec^2}}\right)$$

Eq. 3

$T_p = 2.50\sec$ Eq. 3

$t = 537\left(\frac{gT_p}{U_t^*}\right)^{7/3}\left(\frac{U_t^*}{g}\right)$ Eq. 4

$$t = 537\left(\frac{32.2\frac{ft}{\sec^2}*2.50\sec}{216.19\frac{ft}{\sec}}\right)^{7/3}\left(\frac{216.19\frac{ft}{\sec}}{32.2\frac{ft}{\sec^2}}\right) \quad \text{Eq. 4}$$

$t = 361.19\sec$ Eq. 4

Third Iteration:

$$U_t = U_{1-Hour} * \left(1.277 + 0.296\tanh\left(0.9\log\left(\frac{45}{t}\right)\right)\right) \quad \text{Eq. 5}$$

$$U_t = 119.57\sec*\left(1.277 + 0.296\tanh\left(0.9\log\left(\frac{45}{361.19\sec}\right)\right)\right) \quad \text{Eq. 5}$$

$U_t = 128.91\frac{ft}{\sec}$ Eq. 5

$$U_t^* = 0.539 * U_t^{1.23} \qquad \text{Eq. 2}$$

$$U_t^* = 0.539 * 128.91^{1.23}_{\frac{ft}{\sec}} \qquad \text{Eq. 2}$$

$$U_t^* = 212.44 \frac{ft}{\sec} \qquad \text{Eq. 2}$$

$$T_p = 7.54 \tanh\left[0.833\left(\frac{gd}{U_T^{*2}}\right)^{3/8}\right] \tanh\left\{\frac{0.0379\left(\frac{gF}{U_t^{*2}}\right)^{1/3}}{\tanh\left[0.833\left(\frac{gd}{U_t^{*2}}\right)^{3/8}\right]}\right\}\left(\frac{U_t^*}{g}\right) \qquad \text{Eq. 3}$$

$$T_p = 7.54 \tanh\left[0.833\left(\frac{32.2\frac{ft}{\sec^2} * 7.0\,ft}{\left(212.44\frac{ft}{\sec}\right)^2}\right)^{3/8}\right] \tanh\left\{\frac{0.0379\left(\frac{32.2\frac{ft}{\sec^2} * 4000\,ft}{\left(212.44\frac{ft}{\sec}\right)^2}\right)^{1/3}}{\tanh\left[0.833\left(\frac{32.2\frac{ft}{\sec^2} * 7.0\,ft}{\left(212.44\frac{ft}{\sec}\right)^2}\right)^{3/8}\right]}\right\}$$

$$\left(\frac{212.44\frac{ft}{\sec}}{32.2\frac{ft}{\sec^2}}\right)$$

Eq. 3

$$T_p = 2.49 \sec \qquad \text{Eq. 3}$$

$$t = 537\left(\frac{gT_p}{U_t^*}\right)^{7/3}\left(\frac{U_t^*}{g}\right) \qquad \text{Eq. 4}$$

$$t = 537\left(\frac{32.2\frac{ft}{\sec^2} * 2.49\sec}{212.44\frac{ft}{\sec}}\right)^{7/3}\left(\frac{212.44\frac{ft}{\sec}}{32.2\frac{ft}{\sec^2}}\right) \qquad \text{Eq. 4}$$

$$t = 364.87\sec \qquad \text{Eq. 4}$$

Fourth Iteration:

$$U_t = U_{1-Hour} * \left(1.277 + 0.296\tanh\left(0.9\log\left(\frac{45}{t}\right)\right)\right) \qquad \text{Eq. 5}$$

$$U_t = 119.57\sec * \left(1.277 + 0.296\tanh\left(0.9\log\left(\frac{45}{364.87\sec}\right)\right)\right) \qquad \text{Eq. 5}$$

$$U_t = 128.83\frac{ft}{\sec} \qquad \text{Eq. 5}$$

$$U_t^* = 0.539 * U_t^{1.23} \qquad \text{Eq. 2}$$

$$U_t^* = 0.539 * 128.83^{1.23}_{\frac{ft}{\sec}} \qquad \text{Eq. 2}$$

$$U_t^* = 212.29\frac{ft}{\sec} \qquad \text{Eq. 2}$$

$$T_p = 7.54\tanh\left[0.833\left(\frac{gd}{U_T^{*2}}\right)^{3/8}\right]\tanh\left\{\frac{0.0379\left(\frac{gF}{U_t^{*2}}\right)^{1/3}}{\tanh\left[0.833\left(\frac{gd}{U_t^{*2}}\right)^{3/8}\right]}\right\}\left(\frac{U_t^*}{g}\right) \qquad \text{Eq. 3}$$

$$T_p = 7.54 \tanh\left[0.833\left(\frac{32.2\dfrac{ft}{\sec^2} * 7.0\,ft}{\left(212.29\dfrac{ft}{\sec}\right)^2}\right)^{3/8}\right]\tanh\left\{\frac{0.0379\left(\dfrac{32.2\dfrac{ft}{\sec^2} * 4000\,ft}{\left(212.29\dfrac{ft}{\sec}\right)^2}\right)^{1/3}}{\tanh\left[0.833\left(\dfrac{32.2\dfrac{ft}{\sec^2} * 7.0\,ft}{\left(212.29\dfrac{ft}{\sec}\right)^2}\right)^{3/8}\right]}\right\}\left(\frac{212.29\dfrac{ft}{\sec}}{32.2\dfrac{ft}{\sec^2}}\right)$$

Eq. 3

$$T_p = 2.49\sec$$ Eq. 3

$$t = 537\left(\frac{gT_p}{U_t^*}\right)^{7/3}\left(\frac{U_t^*}{g}\right)$$ Eq. 4

$$t = 537\left(\frac{32.2\dfrac{ft}{\sec^2} * 2.49\sec}{212.29\dfrac{ft}{\sec}}\right)^{7/3}\left(\frac{212.29\dfrac{ft}{\sec}}{32.2\dfrac{ft}{\sec^2}}\right)$$ Eq. 4

$$t = 365.03\sec$$ Eq. 4

The duration of U_t has converged to 365.03 sec and now the remaining wave characteristics can be determined. The following calculations will determine the significant wave height, maximum wave height, and the wave length.

Determine the significant wave height, H_s.

$$H_s = 0.283 \tanh\left[0.53\left(\frac{gd}{U_t^{*2}}\right)^{3/4}\right]\tanh\left\{\frac{0.00565\left(\frac{gF}{U_t^{*2}}\right)^{1/2}}{\tanh\left[0.53\left(\frac{gd}{U_t^{*2}}\right)^{3/4}\right]}\right\}\left(\frac{U_t^{*2}}{g}\right) \qquad \text{Eq. 6}$$

$$H_s = 0.283 \tanh\left[0.53\left(\frac{32.2\frac{ft}{\sec^2}*7.0\,ft}{\left(212.29\frac{ft}{\sec}\right)^2}\right)^{3/4}\right]\tanh\left\{\frac{0.00565\left(\frac{32.2\frac{ft}{\sec^2}*4000\,ft}{\left(212.29\frac{ft}{\sec}\right)^2}\right)^{1/2}}{\tanh\left[0.53\left(\frac{32.2\frac{ft}{\sec^2}*7.0\,ft}{\left(212.29\frac{ft}{\sec}\right)^2}\right)^{3/4}\right]}\right\}$$

$$\left(\frac{\left(212.29\frac{ft}{\sec}\right)^2}{32.2\frac{ft}{\sec^2}}\right)$$

Eq. 6

$H_s = 2.94\,ft$ Eq. 6

Determine the wave length, λ.

$$\lambda = \frac{gT_p^2}{2\pi}\sqrt{\tanh\left(\frac{4\pi^2}{T_p^2}\frac{d_s}{g}\right)} \qquad \text{Eq. 7}$$

$$\lambda = \frac{32.2\frac{ft}{\sec^2}*(2.49\sec)^2}{2\pi}\sqrt{\tanh\left(\frac{4\pi^2}{(2.49\sec)^2}\frac{7.0\,ft}{32.2\frac{ft}{\sec^2}}\right)} \qquad \text{Eq. 7}$$

$\lambda = 29.84\,ft$ Eq. 7

Determine the maximum wave height.

$$H_{\max} = 1.80 H_s \qquad \text{Eq. 8}$$

$$H_{\max} = 1.80 * 2.94 ft \qquad \text{Eq. 8}$$

$$H_{\max} = 5.29 ft \qquad \text{Eq. 8}$$

The maximum wave height, H_{max}, should be limited for depth and for steepness using the lesser of the results of Eq. 9 and Eq. 10.

$$H_{\max} \leq 0.65 d_s \qquad \text{Eq. 9}$$

$$H_{\max} \leq 0.65 * 7 ft \qquad \text{Eq. 9}$$

$$H_{\max} \leq 4.55 ft \qquad \text{Eq. 9}$$

$$H_{\max} \leq \frac{\lambda}{7.0} \qquad \text{Eq. 10}$$

$$H_{\max} \leq \frac{29.84 ft}{7.0} \qquad \text{Eq. 10}$$

$$H_{\max} = 4.26 ft \qquad \text{Eq. 10}$$

Based on the above limitations, H_{max}, equals 4.26 ft. Next determine the assumed maximum distance from the storm water level to the design wave crest, η_{max}.

$$\eta_{\max} = 0.70 H_{\max} \qquad \text{Eq. 11}$$

$$\eta_{\max} = 0.70 * 4.26 ft \qquad \text{Eq. 11}$$

$$\eta_{\max} = 2.98 ft \qquad \text{Eq. 11}$$

In order for the wave forces to be acceptably accurate determine is the following criteria is satisfied.

$$0.035 \leq \frac{H_{\max}}{\lambda} \leq 0.15 \qquad \text{Eq. 12}$$

$$0.035 \leq \frac{4.26\,ft}{29.84\,ft} \leq 0.15 \qquad \text{Eq. 12}$$

$$0.035 \leq 0.14 \leq 0.15 \qquad \text{Eq.12}$$

$$3\sec \leq T \leq 10\sec \qquad \text{Eq. 13}$$

$$3\sec \leq 2.49\sec \leq 10\sec \qquad \text{Eq. 13}$$

The criteria from Eq. 12 are met and from Eq. 13 are not. This is due to the short fetch length that can only form in Little Assawoman Bay.

Superstructure Clearance

To determine if the wave will contact the superstructure the wave crest height above storm water level, η_{max}, must be greater than the distance from the storm water level to the bottom of the girder, z_c. Figure 6.1 shows more detail on this scenario. For the Fenwick Island Bridge z_c = 4.0 ft and η_{max} = 2.98 ft. In this case the wave misses the lowest part of the superstructure by 1.02 ft. This satisfies the requirement to have at least 1 ft of clearance over 100-year design wave crest elevation and no calculations will need to be performed to determine any forces on the bridge.

Calculation Check

The calculations performed in chapters 6, 7, and 8 were checked using a spreadsheet developed by Timothy Stuffle from Modjeski and Master, Inc..

Chapter 9

CONCLUSION

The vulnerability to Delaware's coastal bridge inventory to hurricane forces is low. Delaware has never been struck by a hurricane and there are very few coastal bridges that are located in places that may be impacted if a large coastal storm does occur. Additionally, three vulnerable bridges were analyzed and they provided adequate clearance to prevent impact from a wave during a 100-year event. Table 9.1 shows a summary of the findings.

Table 9.1 Summary of results for the three bridges analyzed.

Input	**Bridge 3-156**	**Bridge 3-437**	**Bridge 3-460**
Water Depth for Normal High Tide (ft)	50.00	4.00	4.00
Fetch Length (ft)	50350	7000	4000
Storm Surge, 100-Year (ft)	6.37	7.66	3.00
Clearance Above Storm Water Level (no wave), Zc (ft)	25.27	5.80	4.00
Wave Height Above Storm Water Level, Nmax (ft)	13.65	4.35	2.98
Clearance Above Design Wave, (ft)	11.62	1.45	1.02

Recommendations

Based on the finding from this report DelDOT does not need to take immediate action to retrofit any of these three coastal bridges. However, they should become familiar with the new specifications and analyze any other bridges they may deem necessary. The new Indian Inlet River Bridge should take into consideration the storm surge and wave heights calculated in this report. Since the existing bridge elevation is more than adequate, the new design should meet the requirements easily.

Additionally, it is recommended that DelDOT familiarizes themselves with recovery techniques incase there is a storm that severely damages a coastal bridge. In previous disasters portions of the superstructure were removed from their bearings and came to rest in the water adjacent to the bridge. It was found the superstructure remained in good condition and could be lifted out of the water and replaced. The probability of needing to use a technique like this in Delaware is very low, but the methods should still be understood to allow for the quickest recovery possible.

Also, as previously discussed the values used for the inputs were not as readily available and accurate as they should be. Several studies were performed determining storm surge heights and they provided contradicting values. It is recommended that an updated study be performed to determine more accurate storm surge heights. If this study is not completed, use engineering judgment to determine which values from previous reports are most acceptable.

Specifications Comments

The *Guide Specifications for Bridges Vulnerable to Coastal Storms* provides clear guidelines to give owners the ability to determine bridges that should be analyzed for coastal loads and any damage the loads may cause to the bridge. It clearly and, from what was observed in this report, accurately allowed for the design wave crest elevation to be calculated. Since none of the bridges analyzed were impacted by the waves, it is not known if the forces on the superstructure that would have been calculated are critical. Also, for an engineer that does not have an extensive background in coastal engineering, the guidelines are easy to follow. Additionally, the explanation of all terms and techniques and the commentary provided were well thought out.

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Appendix A

NOMENCLATURE

d	= average water depth over the fetch including surge, astronomical tide, and local wind setup (ft)
d_s	= storm water depth at the bridge (ft)
F	= fetch length in the direction of the wind from the upwind shore (ft)
g	= gravitational constant (ft/sec^2)
H_{max}	= maximum wave height (ft)
λ	= wave length (ft)
η_{max}	= distance from the storm water level to design wave crest (ft)
t	= duration of U_t (sec)
T_p	= period the waves with the greatest energy exhibited in a spectrum (sec)
U_t	= 100-year design wind velocity at the standard 32.8 ft elevation modified for duration "t" (ft/sec)
U^*_t	= wind-stress factor (ft/sec)
Water Level	= mean sea level if storm surge includes astronomical tide = mean higher high water level if astronomical tide not included in surge

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)