U.S. Department of Transportation
Federal Highway Administration

TEACR Engineering Assessment

Sea Level Rise and Storm Surge Impacts on a Coastal Bridge:
I-10 Bayway, Mobile Bay, Alabama

This is one of nine engineering case studies conducted under the Transportation Engineering Approaches to Climate Resiliency (TEACR) Project.[1] This case study focused on the vulnerability of a coastal bridge to sea level rise and storm surge.

Overview

This assessment investigates potential structural and hydraulic engineering adaptations to increase the resilience of a coastal bridge to extreme events in rising sea levels. The assessment focuses on the I-10 Bridge, which crosses northern Mobile Bay, Alabama (known locally as "The Bayway"). The seven-mile long bridge crosses open bay waters, coastal marshes, and tidal rivers.

This is an important analysis because there are over 70,000 bridges crossing coastal tidally-influenced waters in the U.S., and many of those have been built using techniques similar to this bridge. The present-day vulnerability of these bridges to hurricanes is clear - billions have been spent rebuilding similar bridges along the Gulf Coast in the past two decades. All of those bridges were destroyed by the same mechanism - waves on storm surge. Sea level rise

Case Study Snapshot

Purpose: To evaluate adaptation options for a typical coastal bridge exposed to surge/wave attack with increases in sea level.

Location: Interstate 10, about 5 miles east of downtown Mobile, Alabama. 30° 39.8'N; 87° 57.5'W.

Approach: Evaluate the engineering implications of wave-induced loads that could be experienced by the asset during future storms.

Key Findings: The existing I-10 Bridge is vulnerable and would be destroyed by waves on storm surge under a scenario in which a storm like Hurricane Katrina directly hits Mobile in the future with 75 cm of sea level rise. Loss of the bridge could directly cost primary users $1,130,800 per day. Retrofit adaptations which strengthen the connections between the superstructure (the decks) and substructure could be designed to avoid the primary, historical damage mechanism (separation of the decks from substructure). However, such adaptations alone will likely end in the destruction of the bridge due to other failure mechanisms (deck-girder damage due to negative bending and pile damage) slightly later in the storm. The only adaptation which will protect the bridge is increasing its elevation - perhaps in combination with some of the other structural modification adaptations considered.

Key Lessons: Following the load path implications through the entire structure is required in the design of engineering adaptations for coastal bridges exposed to wave-induced loads on storm surge. Further research is required and justified prior to engineering design of bridge decks to sustain wave-induced loads.

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[1] For more information about the project, visit the project website at: https://www.fhwa.dot.gov/environment/climate_change/adaptation/ongoing_and_current_research/teacr/

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related to climate change will increase storm surge elevations and thus make many existing bridges progressively more vulnerable to this wave damage as the years pass.

In a 2014 report, *Assessment of Key Gaps in the Integration of Climate Change Considerations into Transportation Engineering,* FHWA identified a series of research gaps that represent critical barriers to integrating climate change into transportation engineering practices. [2] The following analysis addresses one critical gap related to engineering solutions for preparing for climate change including incorporating sea level rise and changing storm surge into design practices. This gap is important to address because of the critical economic value of many of these vulnerable coastal bridges.

This assessment first finds that the I-10 Bridge would be destroyed by a storm similar in strength to Hurricane Katrina (2005) but riding on a sea level which is 0.75 meters higher than today and making landfall closer to Mobile Bay. The structural engineering analyses in this assessment indicate that the ultimate capacity[3] of the existing connections between the superstructure and substructure will be greatly exceeded by the wave-induced loads during this storm scenario. These connections were designed for the traditional expected loads including dead and live gravity loads, which act downward. In addition, the bridges were designed for lateral loads from winds. However, the existing connections were not designed for wave-induced loads because it was assumed that the decks were elevated high enough to avoid such loads. The results from this investigation indicate that the bridge would be destroyed by the same failure mechanism that destroyed similar bridges along the Gulf Coast in Hurricanes Camille (1969), Frederic (1979), Ivan (2004), and Katrina (2005) – waves riding on the storm surge will break the connections between the superstructure (the simple span bridge decks) and the substructure (bent beams) that they rest on.

This assessment then investigated the use of four general alternative adaptation measures:

1. Strengthened connections between the bridge decks and substructure,
2. Improved span continuity (between adjacent bridge decks),
3. Modifications to the shape of the bridge to reduce wave-induced loads, and
4. Increased deck elevation in combination with other adaptations.

For the first adaptation measure, this study found it is possible to design connections (both retrofit and new construction) that will be able to resist the wave-induced loads in future extreme

[2] FHWA "Assessment of Key Gaps in the Integration of Climate Change Considerations into Transportation Engineering."

[3] Throughout this assessment, the structural capacities were determined using the AASHTO Load and Resistance Factor Design (LRFD) ultimate limit state (AASHTO, 2012).

events. However, a key finding of this assessment is that if just the connections are strengthened, then the bridge would likely still be destroyed by another failure mechanism:

- The bridge will likely fail from uplift-induced negative bending which the deck and girders are not designed to withstand. ("Negative" bending is the upward flexing due to the upward load acting on the bottom of the deck which is restrained at each end. Most bending, due to weight on the bridge, is in the other, "positive", direction)

- Even if the connections were strengthened and the negative bending issues with the girder and deck were addressed with retrofits, this bridge would still likely fail in this scenario storm due structural failure of the piles. This failure mode is primarily due to the lateral component of the wave-induced loads. This is an interesting finding since the lateral component of wave-induced loads on highway bridge decks are significantly smaller (1/3 in this case) than the vertical component.

- Even if the bridge superstructure (decks) is designed/retrofitted well enough to survive wave-induced loads, then those transferred loads could cause a geotechnical foundation failure. The results indicate that the foundation may be adequate to withstand the uplift forces but not the combination of lateral and uplift forces.

Evaluation of several of the adaption options was limited by a lack of engineering analysis tools due to limited research on wave loads on bridge decks. For example, improving the hydrodynamic shape of the bridge section, by making it more streamlined, has the potential to significantly reduce wave-induced loads – particularly lateral loads. However, estimates of these reductions are limited by a lack of research to quantify them adequately for planning-level or design decisions.

One adaptation measure—raising the bridge—was the only measure that would reliably protect the bridge. The alternative engineering adaptation would involve raising the bridge to an elevation that completely avoids wave loads in the most extreme storm surges. This is the approach taken throughout the Gulf Coast. This was done after the damage caused by Hurricane Ivan (2004) at the I-10 Bridge in Pensacola, Florida, and after the damage caused by Hurricane Katrina (2005) at the US 90 Bridges across Biloxi Bay and St. Louis Bay, Mississippi, and the I-10 Bridge across Lake Pontchartrain, Louisiana.

Another key finding of this engineering assessment, however, is that complete avoidance of wave loads may not be required if increased elevations are combined with some of the other adaptations listed above. The implication is that there are potentially significant life-cycle cost savings available through designs which comprehensively include considerations of the storm surge and wave-induced load risk.

This analysis also evaluated the direct costs associated with bridge closure for road-users in terms of increased transportation costs associated with alternative routes, as well as the direct and indirect impacts felt in downtown Mobile and the broader Mobile-Daphne-Fairhope economy due to reduction in tourism and other leisure trips. The research team concluded that the bridge closure would result in total direct costs of $739,354 per day for passenger vehicles and roughly $756,311 for commercial freight traffic. The increase in travel costs for commuters is likely to impact personal travel and purchase decisions, and these decisions could impact economic activity in downtown Mobile; the annual impact of a year-long bridge disruption could lead to GDP losses ranging from $18.9 million to $61.8 million. State and local governments would also experience loss in potential tax revenue, an estimate of $7,150 and $23,310 per day of bridge disruption.

This asset assessment follows the first nine steps of the Adaptation Decision-Making Assessment Process (ADAP), as shown in Figure 1. For Step 4, Develop Climate Scenarios, the climate data were readily available from the Gulf Coast Phase 2 Study.[4] These climate data were high-resolution storm surge and wave modeling estimates, which included the sea level rise, for this scenario. The approach to quantifying the exposure of the asset follows the recommended methods of vulnerability assessment from HEC-25, Volume 2: Assessing Extreme Events.[5] The focus of this assessment was on Steps 5, 6, 7, and 8 with particular emphasis on Step 7 (Assess Performance of Adaptation Options). Other scenarios (Step 5) were, in essence, assessed by evaluating loads due to lower storm surge levels which would have occurred before the peak storm surge in the scenario storm.

[4] USDOT "The Gulf Coast Study, Phase 2, Climate Variability and Change in Mobile, Alabama."
[5] FHWA "HEC25: Volume 2."

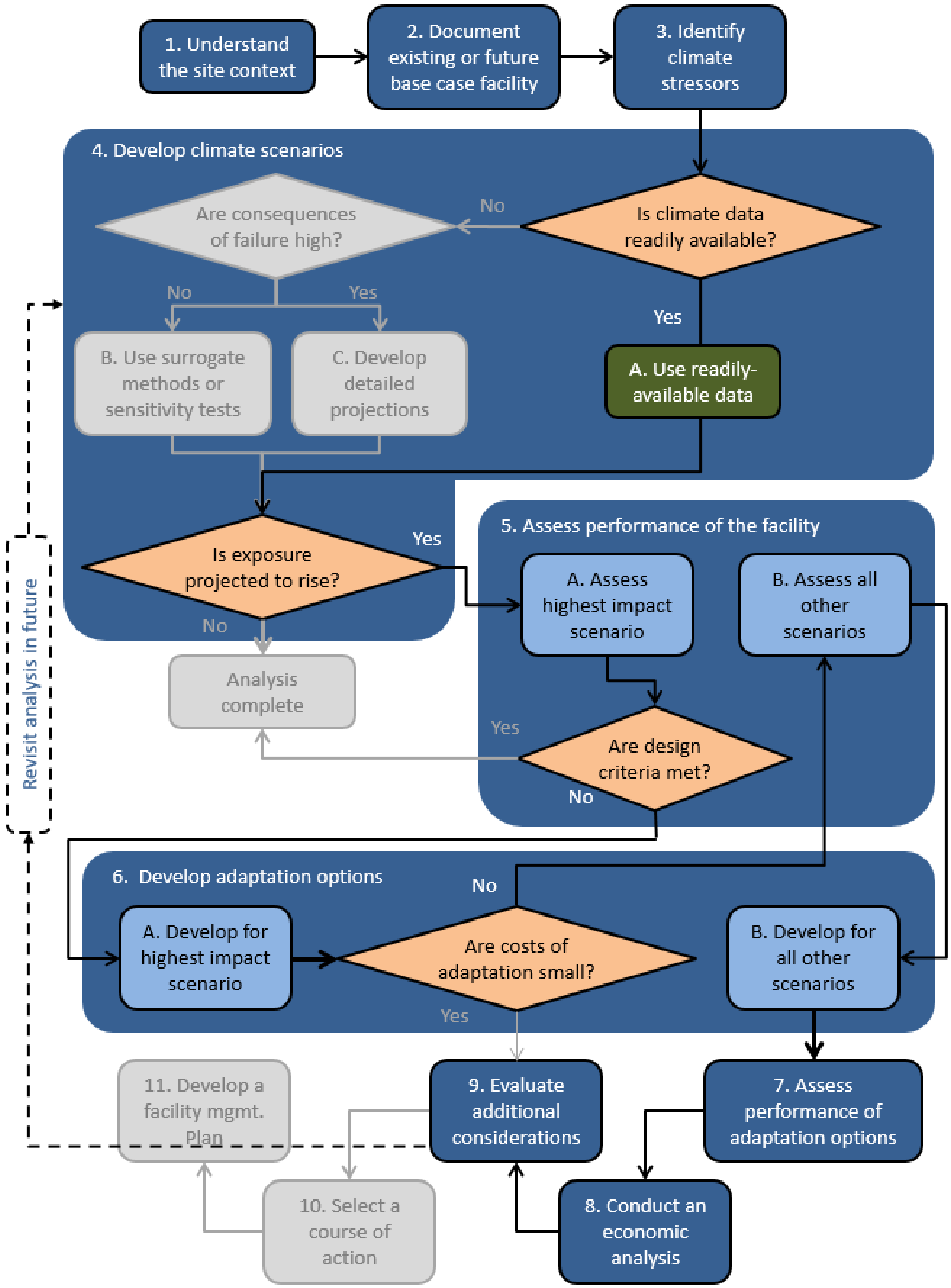

Figure 1: Adaptation Decision-Making Process (ADAP) Used for this Analysis (steps not completed are indicated in gray).

I-10 Florida Coastal Bridge

A related finding of this assessment is that one (other) major bridge in the U.S. may have already been destroyed by the increase in wave-induced loads due to the sea level rise which occurred during the life of the structure. Although it was not a focus of this assessment, these results have some significant implications for the I-10 Bridge over Escambia Bay, Florida (a bridge with a very similar design to the Alabama Bridge) which was destroyed by Hurricane Ivan in 2004:

a) Sea level rise between the time that bridge was originally designed and 2004 may have been just enough to cause the wave-induced loads to be large enough to lead to the catastrophic failure. In other words, if sea levels had not risen for the four decades prior to Hurricane Ivan, the I-10 Florida Bridge may have survived that storm.

b) The I-10 Florida Bridge may have survived Hurricane Ivan if its design had included the adaptations evaluated in this assessment. If the connections had been strengthened as outlined below, the additional strength probably would have allowed the bridge to survive long enough for the storm surge to begin to recede as that storm passed.

It should be noted that these interesting implications are for that I-10 Florida Bridge and Hurricane Ivan only. It appears that Ivan's storm surge hydrograph just barely reached the elevation at which bridge decks began to be moved and then receded. Such is not true, however, for this assessment of the I-10 Alabama Bridge and the selected storm scenario. In this scenario storm, the peak surge is much higher than the deck elevation and thus the wave-induced loads are much more extreme.

Contents

Details of the Analysis

Step 1. Understand the Site Context

The I-10 Bayway Bridge crosses the northern end of Mobile Bay and the southern end of the Mobile-Tensaw River Delta. Mobile Bay is a large, shallow estuary, much of which is less than 12 feet deep. The bridge site location at the northern end of the bay is roughly 30 miles from the barrier islands and the Gulf of Mexico. The tide range in Mobile Bay is considered microtidal at around 2 feet. Tropical storms and hurricanes regularly impact the Mobile Bay area and are a primary cause of coastal flooding and wave damage. Storm surge has exceeded 12 feet in the northern end of the Bay at the existing bridge.

The I-10 Bayway Bridge is part of the major east-west interstate along the Gulf Coast and one of the most heavily traveled roadways in the Mobile metropolitan area. The alternative roadway is U.S. 90/98, also known locally as "the Causeway," which runs roughly parallel to the Bayway but is much more susceptible to flooding from storm surge due to its low elevation on an earthen embankment. State officials are currently planning to widen the I-10 Bayway in conjunction with a new high-rise bridge over the Mobile Ship Channel to increase the capacity of the highway.[6] This represents a significant investment in the future of this bridge.

[6] Alabama Department of Transportation "Environmental Impact Statement for the I-10 Mobile River Bridge and I-10 Bayway Widening."

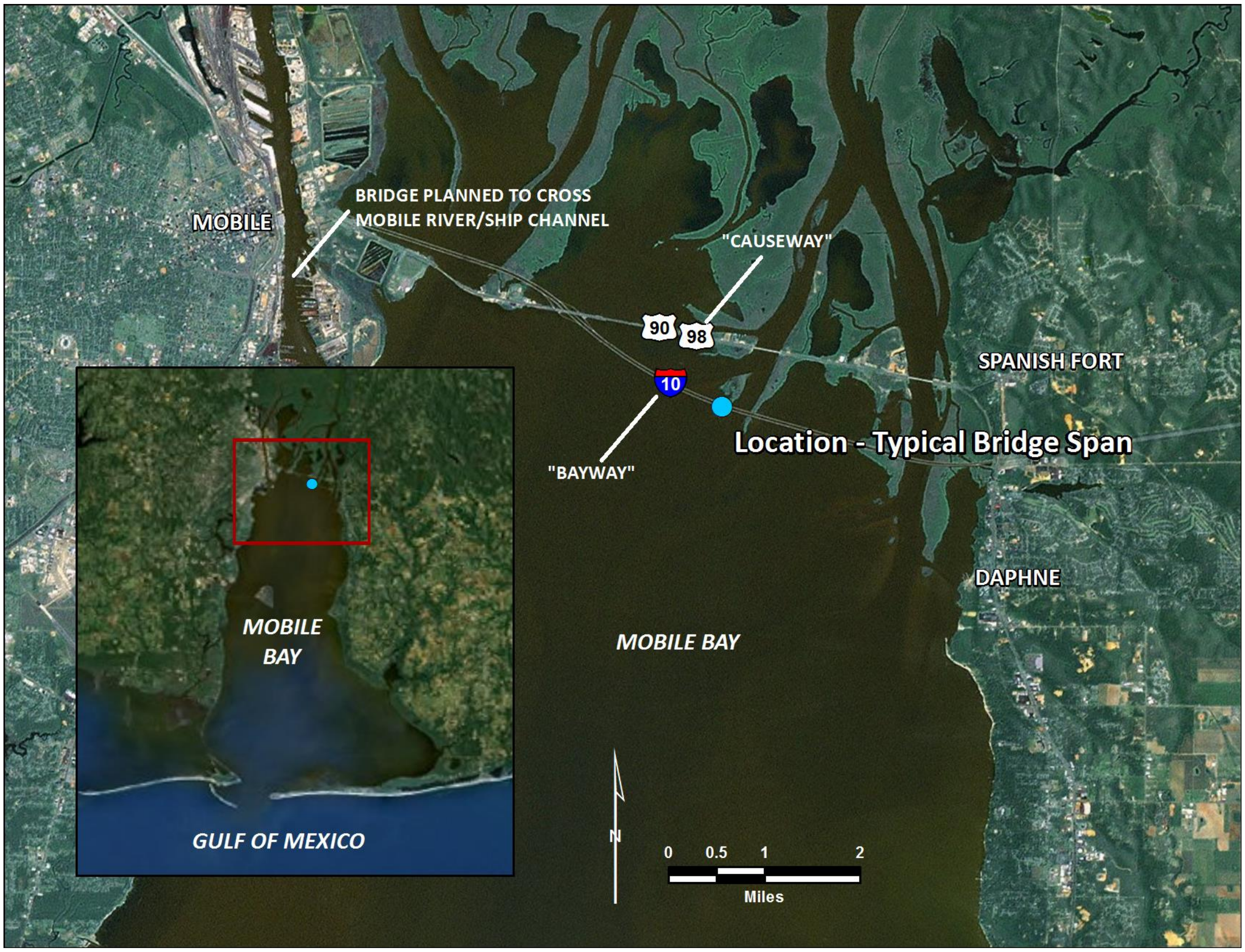

Figure 2: Map showing the location (blue circle) of the bridge span being assessed. This is a typical span on the I-10 Bayway Bridge across the waters of the north end of Mobile Bay.[7]

Step 2. Document Base Case Facility

A single bridge span was selected for this analysis to be representative of a "typical" section on the I-10 Bayway Bridge. This site (see Figure 2) is approximately 4 miles east of Mobile. Hundreds of the bridge sections throughout that portion of the Bayway have the same design. Figure 3 shows several sections of the I-10 Bayway Bridge near the selected location. Also shown in Figure 3 is the terminology used throughout this asset discussion including the girders, diaphragm, bent beam (pile cap), and pile. The selected bridge span carries the two east-bound traffic lanes of the four lane interstate highway. An identical, parallel bridge located just to the north carries the west-bound traffic lanes.

[7] Source: Graphic created by South Coast Engineers. Underlying imagery provided by Esri's World Imagery.

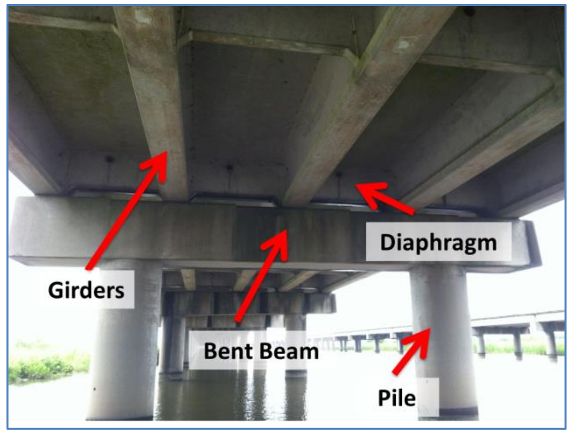

Figure 3: Photographs of a typical bridge section of the I-10 Bayway Bridge from below.[8]

Construction plans were obtained from the bridge owner, the Alabama Department of Transportation (ALDOT), for this analysis. This included both structural engineering plans and original geotechnical engineering soils data. Construction was completed in 1978, and the section exists today relatively unchanged since its completion. The typical bridge section is a prestressed concrete bridge with a deck that is approximately 40 feet wide and 65 feet long. The superstructure consists of a reinforced concrete deck cast integrally with five AASHTO Type III prestressed concrete girders. The substructure of the bridge consists of a reinforced concrete bent beam (pile cap) that is supported by two cylindrical, hollow, prestressed, precast concrete piles that are 54 inches in diameter with a 6 inch wall thickness. The piles are 96 feet long and were embedded about 78 feet into the soil. The soils around the piles are layers of very soft to soft clays. A detailed soil profile, with soil characteristics that were acquired from the testing program prior to construction, is shown in the schematic of Figure 4.

The existing bridge span has a "low-chord" elevation at the bottom of the lowest girder of +16.5 feet above mean sea level (MSL). For this analysis, the elevations from the bridge plans were converted to the present-day MSL datum for evaluation of the wave loads. The bridge plans are referenced to the NGVD (1929) datum but the storm surge elevation estimates are referenced to the MSL of today. These conversions, for sea level rise since design and between datums, followed the NOAA-established datum relationships for the area tidal stations.

[8] Photo credits: South Coast Engineers.

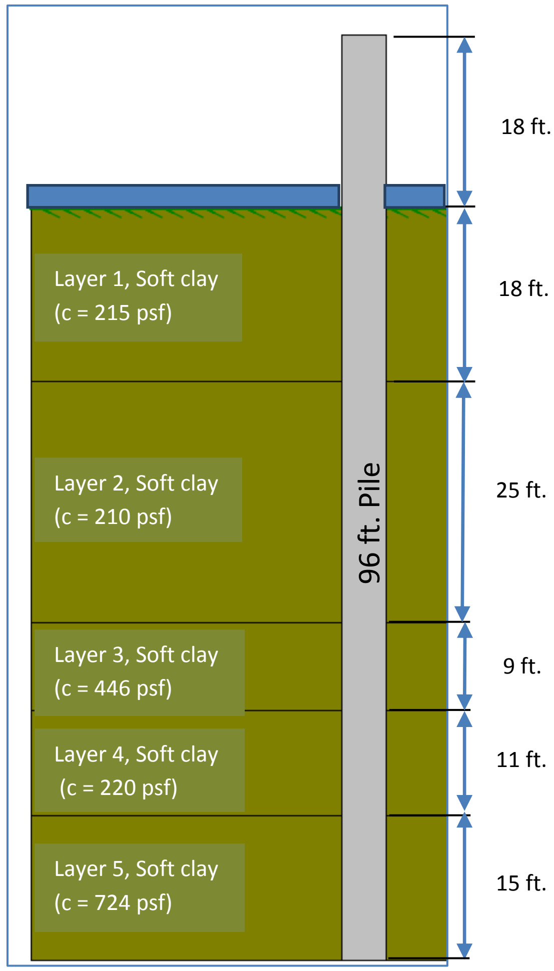

Figure 4: Schematic of Soil Profile near the location of the piles analyzed. "c" is the average undrained shear strength. [9]

Step 3. Identify Climate Stressors

The primary climate stressors affecting the I-10 Bayway Bridge are storm surge and sea level rise. During storm surge events, bridge decks that are subjected to wave action undergo significant lateral and uplift forces which can destroy the bridge. The most important factors contributing to this phenomenon are:

- The relative elevation difference between the storm surge elevation and the bridge deck elevation, and
- The size of the waves at the bridge deck riding on that storm surge.

Climate Stressors

- Sea level rise
- Storm surge
- Wave heights

Sea level rise will decrease the relative elevation difference between the wave crest on the storm surge and the bridge decks and will thus increase the wave-induced loads. Wave-induced loads can be extremely sensitive to storm surge elevation. Increased sea levels can interact non-linearly

[9] Geotechnical engineering data provided by ALDOT.

with the storm surge as it inundates local bathymetry-topography (see *HEC-25: Volume 2 – Assessing Extreme Events*). Thus, the increase in peak storm surge elevation with, say, a foot of sea level rise can be more than one foot at some locations. Secondary climate stressors for this bridge asset are potential changes in hurricane strength due to climate change and larger wave heights due to climate change.

Step 4. Develop Climate Scenarios

The research team used readily available climate scenario data for this assessment. One of the climate scenarios originally developed for the GC2 study was selected as the climate scenario for this assessment. That previous study modeled future storm surge and waves under 11 different scenarios covering a range of sea level rise and storm strengths.

> **Climate Data Overview**
>
> **Level of Detail:** Readily available data
>
> **Data Source:** High-resolution storm surge and wave models (ADCIRC and STWAVE) from GC2 study
>
> **Scenario:** A future hurricane with characteristics like those of Katrina, except shifted to make landfall farther east, with +75 cm of sea level rise
>
> **Is exposure projected to change in the future?** Yes, exposure increases with increased sea levels

For this assessment a climate scenario (from the GC2 study) represented by Hurricane Katrina with a modified track and +75 cm of sea level rise was selected. Shifting (within the framework of the numerical model) Hurricane Katrina's historical track to the east such that it made landfall near Biloxi, Mississippi created more severe storm conditions in Mobile Bay, as stronger winds blow up the length of the bay. This change represents a possible deviation in path that Hurricane Katrina could have taken in 2005. Although the original GC2 study investigated the impacts of intensified storms, this assessment did not select any of the modeled scenarios which evaluated the effect of increased storm winds and decreased central pressure due to climate change. This scenario was selected because its estimated, modeled, and peak storm surge elevations were substantially higher than the low-chord of the I-10 Bridge decks. It was anticipated that these storm conditions would produce wave-induced loads which would have destroyed the existing bridge. This indeed was a finding of the analysis as outlined below. Selection of this scenario for evaluation thus is a reasonably severe test of adaptations.

The sea level rise value selected for this analysis, +75 cm (~2.5 feet), is in the range of most projections for global, eustatic sea level by the year 2100 (e.g., see Figure 2.9 of HEC-25: Volume 2 which is from the National Climate Assessment). This value can be considered as the local relative sea level rise including both the global eustatic sea level rise and vertical land movement for this case study. The historic rate of relative sea level rise measured by the NOAA tide gauge at the south end of the bay, Dauphin Island, Alabama, is 1.05 feet per century. Thus, this scenario represents an assumption of increased relative sea level rise over the historic rates if it is viewed as a 2100 scenario.

The GC2 investigation involved simulating the "Katrina-Shift+75" scenario using the Advanced CIRCulation model (ADCIRC) and the STeady-state WAVE model (STWAVE). Figure 5 shows the surge inundation depths around Mobile Bay for this storm. The research team gathered the detailed storm surge elevation data and wave height information from the existing results of GC2 for the selected scenario at the bridge location. These modelled data are shown in Figure 6. The storm surge will rise from about +2 feet (above MSL) to a peak of +22.3 feet over about 12 hours and then fall back to about +6 feet over the next 20 hours. The corresponding waves at the peak of the storm at that location are estimated with STWAVE as having a significant wave height[10] of H_s = 5.6 feet with a peak wave period[11] of T_p = 7.1 s.[12]

[10] Spectral-based estimate of the significant wave height characterizing the energy in an irregular sea-state.
[11] The wave period associated with the peak of the wave energy density function.
[12] FHWA “HEC 25: Volume 1.”

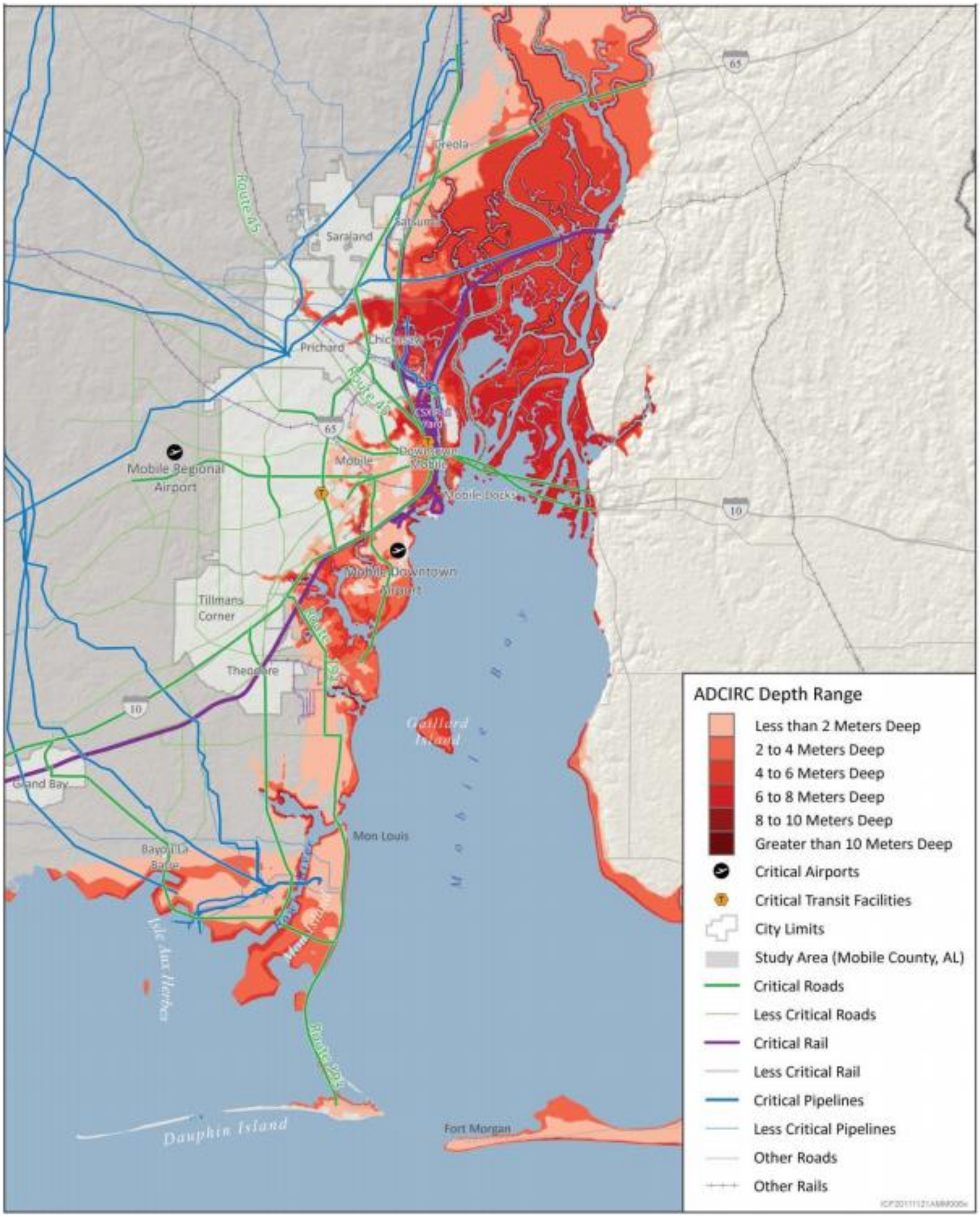

Figure 5: Surge inundation model results for the scenario storm (characteristics of Hurricane Katrina except making landfall near Biloxi and on +75 cm of sea level rise).[13]

[13] USDOT "The Gulf Coast Study, Phase 2, Climate Variability and Change in Mobile, Alabama."

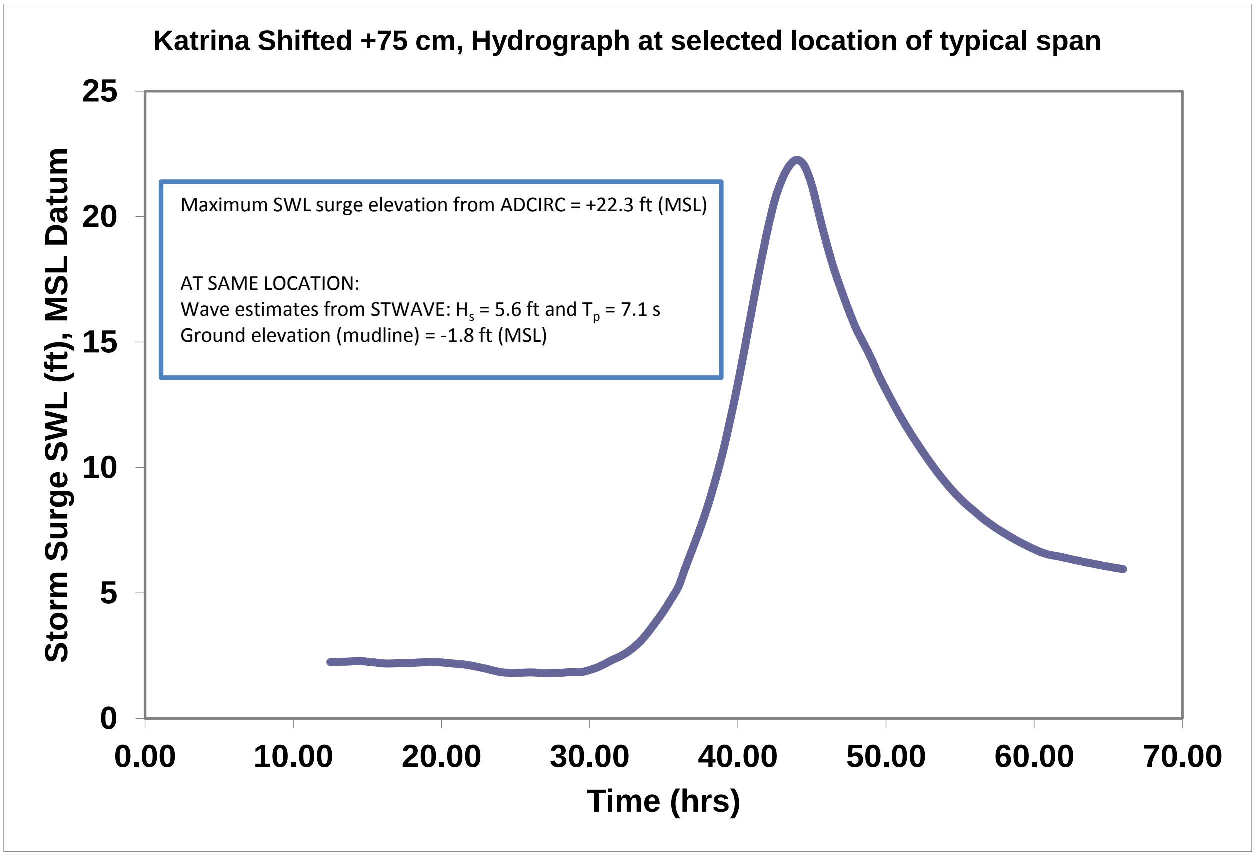

Figure 6: Storm surge hydrograph at the selected span location of the I-10 Bridge during the selected scenario storm (characteristics of Hurricane Katrina except making landfall near Biloxi and on +75 cm of sea level rise) as estimated by ADCIRC. The corresponding wave heights estimated by STWAVE are also shown in the box.[14]

To put the peak value of storm surge elevation shown in Figure 6 into perspective, that elevation is just 0.5 feet below the elevation of the top of the rails (+22.8 feet) of the existing bridge deck. Thus, if the bridge survives until this level of the storm, the waves would be breaking on and across the decks.

It is noted that the storm surge levels shown in Figure 6 and evaluated below are much higher than any recorded, historic storm surge at this location. The highest recorded storm surges experienced here were on the order of +11 to +12 feet during passage of Hurricane Katrina. The existing bridge has low-chord elevations of the girders of about +16.5 feet (above MSL). The wave crests during Katrina were likely just barely passing under the decks with little or no contact with the girders. However, storm surge in Gulf coastal bays is extremely sensitive to the track location of the storm as well as the other storm characteristics (maximum wind speed, forward speed, etc.) and the results shown in Figure 6 are reasonable for this scenario storm. Use of high-resolution models allow for evaluation of "what-if" scenarios like this one and are used

[14] USDOT "The Gulf Coast Study, Phase 2, Climate Variability and Change in Mobile, Alabama."

extensively in estimating coastal flood risk for this reason. A related issue is whether or not this scenario storm is a "reasonable" storm to consider for this assessment. It certainly seems reasonable to assume that a large storm like Katrina could have made landfall along the northern Gulf Coast some 50 miles farther to the east of where it actually made landfall. A "Level of Effort 3" analysis, using the terminology of HEC-25: Volume 2: Assessing Extreme Events, would include developing an estimate of the probability of this event in a more complete risk analysis.[15]

Step 5. Assess Asset Performance under the Scenario

> **Facility Performance Overview**
>
> **Impact Scenario:** A future hurricane with characteristics like those of Katrina, except shifted to make landfall farther east and with +75 cm of sea level rise.
>
> **Asset Design Standards:** ALDOT typically requires bridges that span rivers to maintain 2 feet of freeboard above the design stage. However, the detailed rationale for the selection of the deck elevation of the existing bridge is unknown.
>
> **Key models, tools, and assumptions:** FHWA's HEC-25, ASHTO LRFD Bridge Design Specifications, the PCI Design Handbook, ACI guidance for concrete structures, and FHWA's guidance manual on driven pile foundations were used in the asset performance and adaptation option evaluations. Reasonable assumptions were made about the distribution of the wave-induced loads throughout the structural components.
>
> **Is the structure resilient?** No. The existing bridge would be destroyed by this assessment's selected storm scenario.

The performance of the existing bridge during the selected scenario storm was analyzed with existing engineering methods. The specific focus was on the wave-induced loads and how the existing bridge structure would perform if it were subjected to that selected storm scenario. The method outlined in *HEC-25 (2nd ed.) – Appendix E: A Method for Estimating Wave Forces on Bridges*[16] was used to estimate the wave-induced loads. The structural capacity of the existing bridge was computed based on evaluation of the construction plans. These plans included all bridge component elevations and all structural design details. With bridge element elevations ranging from +16.5 to +22.8 feet (low-chord of girder to top of safety rail) and a peak storm surge elevation of +22.3 feet, the bridge will be well within the regime of extreme wave-induced loads during the scenario storm.

Figure 7 shows the details of the connections between the superstructure and the sub-structure. The connection is with bolts into the girder and the bent beam (pile cap) though structural steel angles. The connections are on the outermost girder, the lower four corners, on each bridge deck, as well as the center girder. The bolts embedded into the concrete of the girder bottoms are 3 inches long. It is this connection that has typically failed on other, similar bridges exposed to hurricane surge and waves as the extremely large, wave-induced loads have caused the concrete around these bolts to break.

[15] FHWA "HEC25: Volume 2."

[16] FHWA "HEC25: Volume 1."

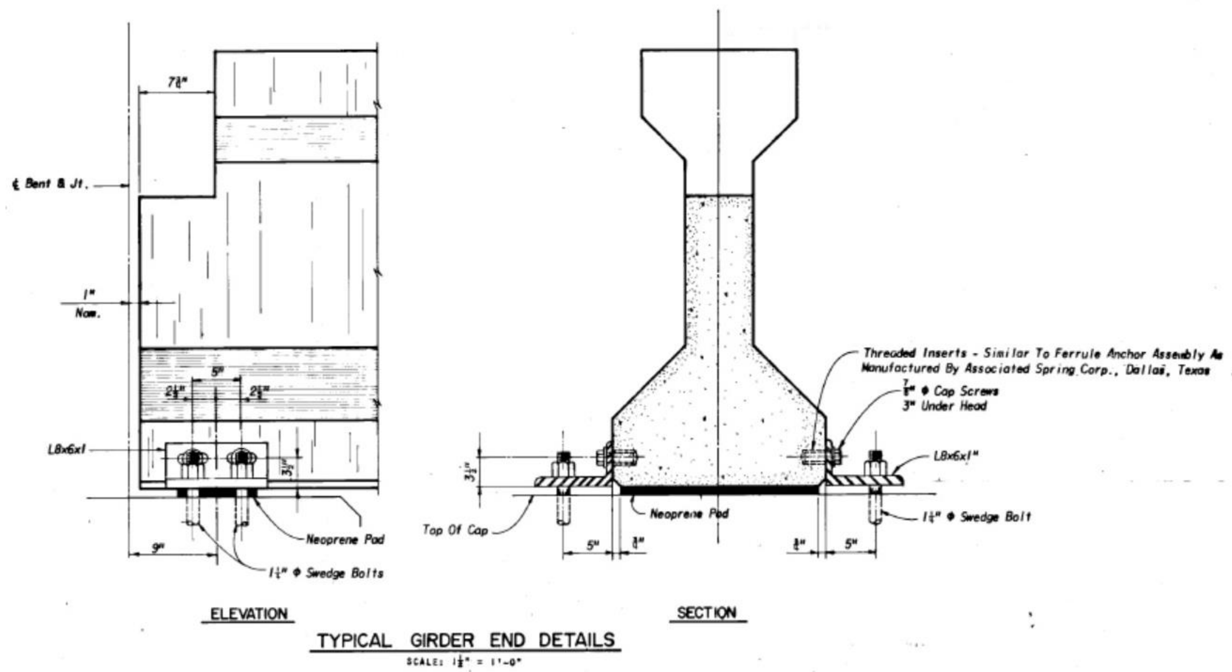

Figure 7: Construction plans showing the detail of the connections between the superstructure and the substructure.[17]

Table 1 summarizes the loads, capacity and expected performance of the existing bridge during the selected storm scenario. The first row in Table 1 presents the components of the wave-induced loads on the bridge (if the bridge were to survive until the peak of the storm surge). The second row of Table 1 summarizes the estimated ultimate capacity of the bridge connections, or how much load the connections can withstand before they break and the bridge deck separates from the substructure. The load (first row) greatly exceeds the capacity (second row). Thus, the implication is that the connection between the bridge deck (specifically the bottom of the girders) and substructure (bent beam) will fail before the peak of the surge hydrograph. The ratio of capacity to demand (at the peak of the surge) for the vertical loads is 1:3. This ratio includes the weight of the superstructure (girders, deck, and rails). For horizontal loads, the ratio is approximately 1:5. The conclusion is that the connections will fail and the bridge will be destroyed by this selected scenario storm. The methodologies used in developing the capacities in Table 1, as well as subsequent findings related to a more complete evaluation of the load path, are summarized in Step 7 below.

[17] Plans provided by the Alabama Department of Transportation.

Table 1: Summary of the loads, capacity, and expected performance of the existing I-10 Bridge in the selected scenario storm.

Summary of loads, capacity, and expected performance of existing I-10 Bridge	Vertical Direction (kips)	Horizontal Direction (kips)
Wave-induced loads on bridge deck during the selected scenario storm	1,550	530
Capacity of existing connections between the girders and the bent beams	520	110
Failure (Yes/No)	Yes	Yes

The primary finding of this step of this assessment is that the existing bridge will be destroyed by the selected storm scenario. The failure mechanism will be the same as that experience by similar bridges in the storms of 2004 and 2005 – the connections between the superstructure and substructure will fail under the extreme loads and the bridge decks will be moved off the bent beams (pile caps).

Given this finding, the existing bridge will fail in a storm with a peak surge of +22.3 feet, logical questions include "will it fail in a storm with a lower peak surge?" and "what is the highest storm surge level the existing bridge can survive?" The wave-induced load estimation methodology can be applied to determine that critical surge level. The capacity of the existing connections will be exceeded when the storm surge rises to a level around +14.9 feet. Specifically, the horizontal wave-induced loads will exceed the capacity at about that storm surge level. It is interesting that the horizontal wave-induced loads are the critical loads. While the vertical wave-induced loads are larger, they are partially resisted by the substantial weight of the decks. Since the vertical wave-induced loads will remove or reduce the normal forces, and therefore the frictional forces, at the bearings, it was assumed the only resistance to the horizontal loads are the connections. The uplift capacity of the connections will be exceeded when the storm surge stillwater level (SWL) is at approximately 16.1 feet and the lateral capacity will be exceeded at 14.9 feet. For the purposes of this assessment, it was assumed that wave conditions were the same for all surge levels, a reasonable first approximation for these purposes.

This finding, that the bridge will fail when the storm surge SWL reaches approximately 15 feet, implies that much less surge is needed than the peak of the surge (22.3 feet) for this selected scenario. This behavior is consistent with the damage to other bridges caused by Hurricane Katrina: most of the damage to the Mississippi US 90 bridges probably occurred before the peak of the storm surge. Another implication of this finding is that this I-10 Alabama Bridge across Mobile Bay, which experienced a peak surge of between 11 and 12 feet during Katrina, probably could only have withstood surge levels which were 3 to 4 feet higher that day.

One other interesting finding of this analysis is that the bridge is more vulnerable today, in 2015, than it was when originally constructed in 1978. This became obvious during the process of calculating the clearance between the surge and bridge elevations while estimating the wave-induced loads on the bridge. In order to properly compute the loads, the relationship between the constructed bridge elevations and water level during the storm must be computed. An adjustment was made for the change in sea level since construction. Relative sea level has risen about +0.25 feet since construction and the bridge clearance is correspondingly that much "lower" in terms of its clearance above the water. That reduced clearance, while it may not seem like much, could be critical given that wave-induced loads are extremely sensitive to clearance when the surge levels are close to the bridge decks. For this bridge deck and this storm, reducing the clearance by 0.25 feet will increase the resultant wave-induced loads about 44 kips (44,000 pounds). The increase in load due to the reduced clearance has been estimated with the methodology outlined in HEC-25 and no load factor has been applied. While more research is needed to more precisely understand wave-induced loads and validate the existing methods, all investigations and experience indicate that wave-induced loads are extremely sensitive to clearance when the water level is near the low-chord of the bridge.

Step 6. Develop Adaptation Options

Four adaptation alternatives were developed and evaluated in this assessment (see box). Some of these adaptation alternatives could be used as retrofits and some could be used with new construction and design. A key finding in the next section is that strengthening the connections could lead to deck failure due to negative bending later in the storm. Thus, strengthening the connections alone may not "buy" much more storm or climate resilience. The results outlined below indicate that only the fourth of these adaptation options, increased deck elevations, is clearly an adequate adaptation option for this case study assessment at this time.

Adaptation Options Evaluated

1. Strengthened connections between the bridge superstructure and substructure
2. Improved span continuity
3. Bridge shape modifications to reduce loads
4. Increased deck elevations in combination with other adaptations

The first adaptation considered was modifying the connection between the bridge superstructure and substructure. This is the obvious adaptation since this is the failure mechanism predicted in the previous section and the common failure mechanism for similar bridges historically. Several different options, including a form of a shear block[18], were considered as outlined below and shown in Figure 8.

[18] A shear block (sometimes called a shear key) is a connection type that resists horizontal displacement, but allows vertical displacement.

The second adaptation considered was increasing the continuity between adjacent spans. Several of the railroad bridges that survived Hurricane Katrina near highway bridges, which did not survive, used continuous spans. Figure 8 also shows a potential retrofit to the existing I-10 Bridge which would increase continuity as outlined below.

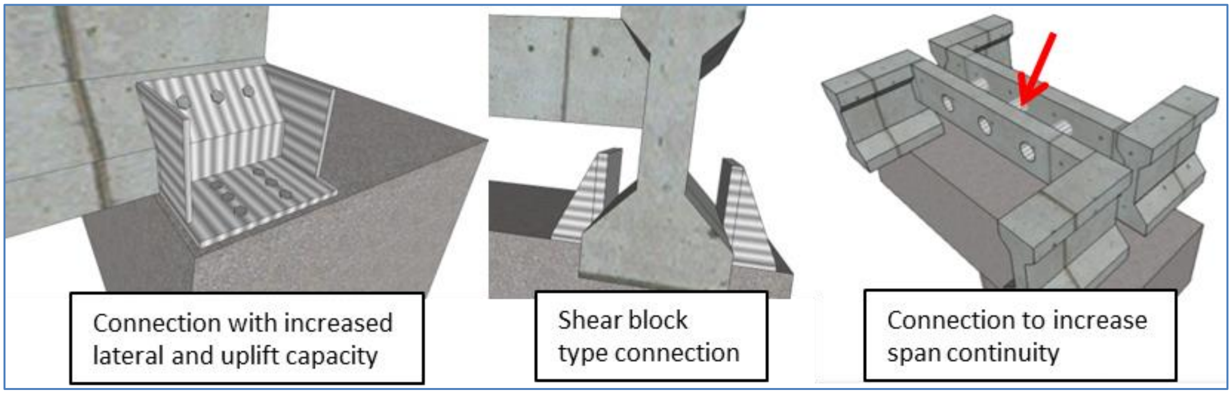

Figure 8: Examples of potential retrofit options for several of the adaptations considered: (a) strengthened connection to carry full loads, (b) strengthened connection to carry lateral loads (shear block), and (c) increased span continuity.

The third adaptation considered was modifying the hydrodynamic shape of the bridge cross-section. Figure 9 is an image of the outside edge of the existing asset – which is typical of many coastal bridges. The geometry of the "seaward" side of the bridge configuration is concave and will trap water as waves impact the structure. Very high, short duration, loads can be generated when a small air pocket is trapped between a rigid structure and a breaking wave.

Figure 9: Outside edge of bridge showing concave configuration which traps an air pocket when a wave strikes it and potentially increases lateral loads.[19]

[19] Photo credit: South Coast Engineers.

A fourth adaptation considered was increasing the elevation of the bridge in combination with the other adaptations. Increased bridge elevation reduces both lateral and uplift forces, thus reducing the structural demand on bridge components. Several scenarios were evaluated with increased elevation. Table 2 presents the adaptations considered and some of the general pros and cons.

Table 2: Brief overview and descriptions of the adaptation alternatives considered in this assessment.

Adaptation	Description	Pros	Cons
Strengthened connections	Improve the connection of the girders to bent beams to resist wave-induced loads	Retrofit or new design option, provides a physical load path mechanism	Transfers loads to other elements of the bridge which can lead to other failure mechanisms and limited guidance requires more research
Improved Span Continuity	Bridge decks are integrally connected to the adjacent decks	Increases the "effective" dead load of superstructure	Does not address potential failures in the superstructure and possible foundation or substructure failure
Modified bridge shape	Modifications to the bridge cross-section to improve its hydrodynamics and reduce horizontal loads	Potential to reduce lateral wave-induced loads	May not be an acceptable structural engineering option and more research would be required to develop guidance
Combination of increased elevation and other adaptations	Construction of bridges at a higher elevation but not so high as to always avoid wave-induced loads	Can be designed to work in all cases. Could reduce wave-induced loads on structure to a level within the design capacity	May be valuable as a new design adaptation strategy but more research would be required to develop guidance

Adaptation: Strengthened connections between the bridge girder and bent beam

The first adaptation considered was modifying the connection between the bridge superstructure and substructure. Two types of strengthened connections were considered under this adaptation. The first type of connection can carry both the lateral and uplift loads. The second type of connection is a shear block type connection that can carry the lateral loads, but allow vertical motion to dissipate the uplift loads.

The first option considered was strengthening of the connections to carry both the lateral and uplift loads. The original connection, shown conceptually in Figure 10a, engaged the girders with bolts connected to the edge of the girder. The capacity of these bolts is low under uplift forces, and the majority of the capacity is due to the dead load of the girder. The retrofit option increases the uplift capacity by directly engaging the girder, not by relying on the bolt capacity. Figure 10b shows an example of how the reinforced connection may look. Both connection types shown in Figure 10 need to be designed to account for the expected thermal movements in the girders (i.e. slotted bolt holes) and for bearing pads. To increase the capacity of the connections, three primary design modifications were considered. The first modification was increasing the number of connections. The current bridge design only includes connections at three of the five girders. A retrofit option includes connections at each girder. The second modification includes changing the shape of the connecting element to engage the girder along the sloped surface. The third modification increases the number of bolts into the bent beam to increase both uplift and lateral capacity.

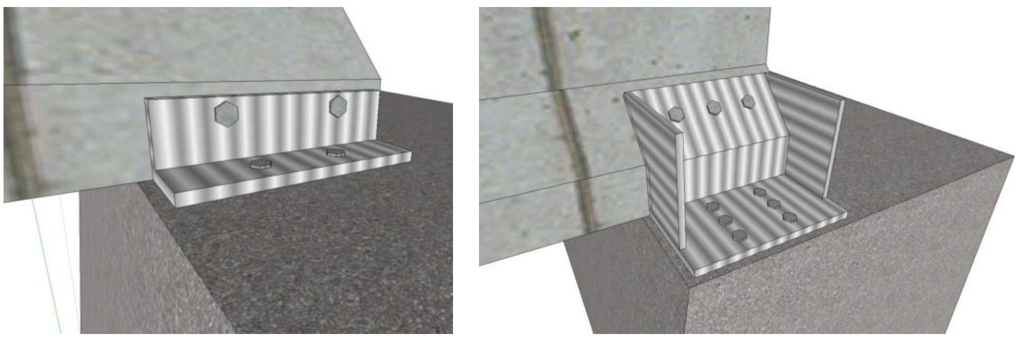

Figure 10: Conceptual detail of connections between the girders and bent beam: (a) current connection configuration, (b) reinforced connection to carry both lateral and uplift forces.

The second type of connection, a shear block, considered under this adaptation is shown in Figure 11. This connection type provides restraint against lateral loads, but does not resist uplift forces. Instead, the bridge superstructure is allowed to move upwards until the dead loads overcome the wave-induced uplift forces. The primary advantage of this system is that the uplift forces are not transferred to the bridge substructure. Shear block or shear key type connections are used often in bridge construction, particularly in seismic regions. However, one of the major differences in this application is the large vertical displacements that the structure will experience.

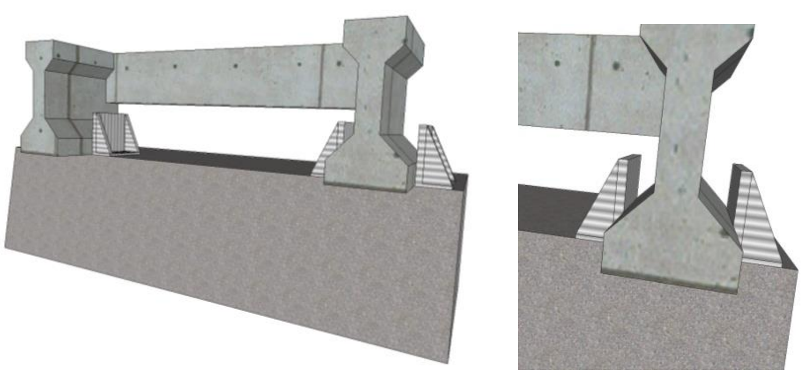

Figure 11: Shear block type connection that allows upward movement of the superstructure to dissipate uplift loads, but resists lateral loads.

Adaptation: Improved span continuity

The existing bridge superstructure is designed to be simply supported between each bent beam. This is a typical design approach for coastal bridges. Each span is structurally independent of the adjacent spans. This adaptation option will increase the span continuity by connecting adjacent spans for uplift forces, while allowing them to act as simply supported for live loads and thermal expansions. This adaptation is based on the realization that there is some limit as to how much linear span of a bridge will be acted on by a wave crest at any one time due to the short-crested nature of storm waves. One potential option to achieve the desired span continuity is to add a doweled connection between the adjacent spans at the end diaphragms as shown in Figure 12. The dowels could be installed in such a way to allow thermal expansion and contraction of the superstructure and to allow small rotations due to live loads. It was assumed for this assessment that waves would not impact adjacent spans simultaneously. This adaptation would then effectively increase the dead load of the superstructure for uplift loads, since the loads would only impact one span, but the dead loads from three spans would be engaged. The use of continuous spans for bridge structures is not unusual, however, the adaptation of simply supported spans to continuous spans is. It is important from a design perspective that the girders continue to function as simply supported under live loads, but would function as continuous under the superimposed wave loads.

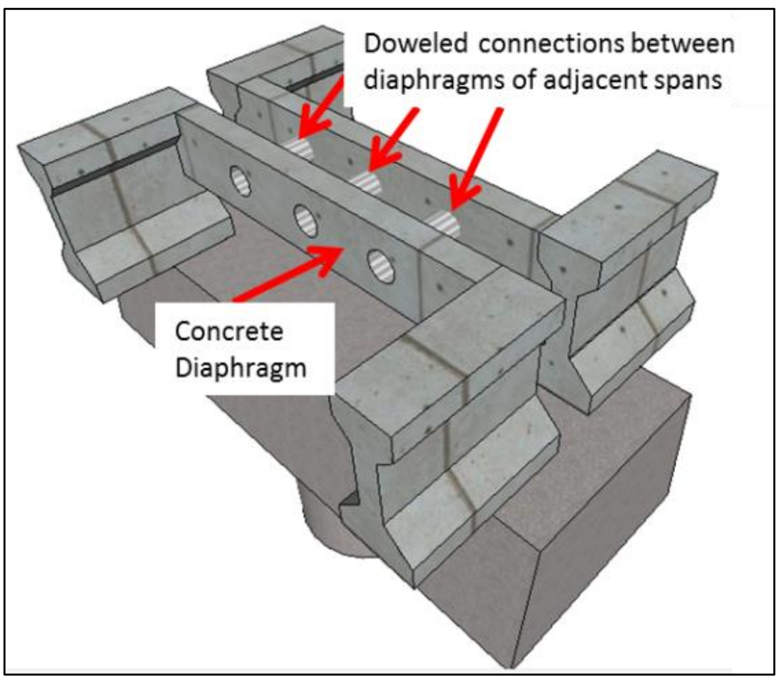

Figure 12: Potential conceptual retrofit option to develop continuity between adjacent spans for uplift forces.

Adaptation: Modified bridge shape

One possible adaptation is to change the cross-sectional shape of the typical coastal bridge to reduce the wave-induced loads through hydrodynamic considerations. In unidirectional river flow, it is well known that more hydrodynamic shapes for piers and decks reduce the drag force on the structure. It is clear by inspection of Figure 9 that the typical, coastal bridge deck cross-sectional shape is not particularly hydrodynamic even for unidirectional flow. This issue is compounded in the wave environment because of the likelihood of pockets of air being trapped between the wave face and the concave surface of the bridge girder-deck combination. Also, rail openings might reduce the lateral wave-induced loads due to reduced cross-sectional area exposed to the waves.

This possible adaptation, of improving the hydrodynamic shape of bridge decks exposed occasionally to extreme wave loads, has been suggested both as retrofit and a new design option. As a retrofit, this could be some lightweight plastic attachment on the side of the bridge or under the bridge. There are obvious design issues to be addressed. As a new design option, this could be something as common as a traditional box beam section instead of the girder-deck sections. This could be more complex as has been done in wind engineering for a few bridges in the world. Changing the traditional or typical cross-sections of bridges, which have served state DOT's well for many years, will have to be very well justified.

Adaptation: Increased deck elevation in combination with some of the above

Another adaptation option is increasing the deck elevation in combination with some of the other adaptation options discussed above. This approach is appropriate for consideration for new construction but would be difficult or expensive to implement as a retrofit on an existing bridge. For new construction the question becomes, if the bridge can indeed tolerate some level of wave-induced load at the peak of the storm surge, why not incorporate that knowledge into the design of the structure? The low-chord of the bridge deck may be able to be set lower than the crest of the largest waves at the peak of the storm surge. The amount of additional elevation above that wave crest elevation can be considered freeboard (typically taken as 1-2 feet based on the similar concept in riverine hydraulics). This procedure results in much higher deck elevations, relative to the peak storm surge elevation, than those discussed here.

Other Adaptation Options

A number of other adaptation options could be envisioned (holes in decks, open deck grates, etc.) but were not pursued in this analysis because the research team judgement was that they were not viable due to other, significant, practical problems. One possible adaptation option is the use of a separate breakwater to reduce the wave heights and thus the wave-induced loads on the bridge decks. Wave-induced loads were the critical loads leading to failure of the bridges damaged in the hurricanes of 2004 and 2005. Storm surge alone, without waves, likely will not damage this bridge as the buoyant forces are less than the structural capacity of the design. Thus, an adaptation that calms the waves, a breakwater south of the bridge, could be effective. The breakwater would have to knock the storm wave heights down significantly and thus, would have to be very substantial in terms of crest elevation and strength. Given the location, such an approach would be problematic along the entire 7 miles of bridge deck, particularly as it would affect tidal circulation. However, it could be viable as an adaptation option protecting a much shorter portion of the bridge such as the lower elevation on-ramp at Exit 30.

Step 7. Assess Performance of Adaptation Options

Each of the four adaptations considered were evaluated for their performance. A load path analysis was performed to determine where a failure would likely occur if each adaptation was adopted. The load path analysis included a structural evaluation of the superstructure and substructure, as well as a geotechnical evaluation of the foundation system. The load path analysis primarily focused on the first adaptation option, strengthened connections, to address questions about where the loads would be transferred to if the connections were strengthened to a level that they did not fail. The results are one of the key findings in this analysis: the loads will likely destroy the bridge due to other mechanisms, including negative bending of the deck, anyway.

Structural Evaluation

The structural evaluation included a load path analysis, which consists of tracing the loads that are imposed on the structure through the structure and the supporting elements (foundation system). The load path analysis was then used to assess the performance of the adaptation options by determining the most likely failure mechanism (if any) for each adaptation. The load path analysis included evaluating:

- The connection of the bridge superstructure to the substructure,
- The capacity of the girders in transverse tension (forces pulling the girder from the top and bottom),
- The connection of the girders to the bridge deck,
- Negative bending in the girders,
- Negative bending in the bridge deck,
- Bending in the bent beams,
- The connection of the bent beam to the piles, and
- The structural capacity of the piles.

The structural capacities of the bridge components were determined using a combination of the AASHTO LRFD Bridge Design Specifications (2012),[20] PCI Design Handbook (6th edition),[21] and ACI 318-11.[22]

The load path analysis had two steps: one, determining the wave-induced loads as they are transferred to each structural element to find the structural demand on that element, and two, finding the structural capacity of the element using standard structural analysis/design procedures. For each analysis it was assumed that the load was transferred from the previous step, even if the structural demand was greater than the capacity. In other words, the analysis assumed that each element that was found to be inadequate was strengthened to allow the next failure mode to occur.

Table 3 summarizes the structural demand and capacity for each of the elements of the bridge design. When the structural demand exceeds the structural capacity, that failure mode will likely occur (the failing element rows are shaded in Table 3). Note that the values presented in Table 1 above, for the connections, are repeated in the first two rows of Table 3 for completeness. The conclusion, from inspection of the summary results in the right two columns of Table 3, is that in addition to the connections between the superstructure and substructure discussed above, there are three other potential failure modes: girders in negative bending, bridge deck in negative bending, and structural failure of the piles.

The failure of the connections (the first two rows in Table 3) was briefly discussed in Step 5: Assess Asset Performance under the Scenario above. The structural demand is greater than the structural capacity and thus, the connections will fail. That is the same failure mode that severely damaged the other bridges in the hurricanes of 2004 and 2005. Given the similarities in bridge design and location, this failure mode result was expected. However, the other potential failure modes, particularly those due to negative bending, were not expected at the beginning of this assessment.

[20] AASHTO "AASHTO LRFD Bridge Design Specifications: Customary U.S. Units."
[21] PCI "PCI Design Handbook: Precast and Prestressed Concrete."
[22] ACI Committee 318 "Building Code Requirements for Structural Concrete and Commentary."

Table 3: Summary of the structural demand compared to structural capacity for each element/failure mode due to the wave-induced loads at the peak of the storm. Shaded rows are the failure modes that will likely be problematic because the demand exceeds the capacity.

Failure Mode	Description	Structural Demand	Structural Capacity
Vertical capacity of connections between superstructure and substructure	Vertical capacity including dead load of superstructure	1,550 kips	520 kips
Horizontal capacity of connections between superstructure and substructure	Horizontal capacity of connections only	530 kips	110 kips
Negative bending in girders	Moment capacity of each girder	1,890 kip-ft.	380 kip-ft.
Negative bending in bridge deck	Moment capacity	2.8 kip-ft.	0.4 kip-ft.
Structural capacity of pile	Moment capacity	2,830 kip-ft.	2,150 kip-ft.
Transverse tensile load in girder	Tensile capacity at each connection (girder end)	160 kips	170 kips
Connection of girder to bridge deck	Total tensile capacity of connection	1,550 kips	1,710 kips
Bending in bent beams	Moment capacity	1,090 kip-ft.	1,370 kip-ft.

The structural demand values, in the third column of Table 3, are from the wave-induced loads at the peak of the scenario storm. The wave loads were estimated using the methods outlined in HEC-25 (2nd ed.) – *Appendix E: A Method for Estimating Wave Forces on Bridges*[23]. This method estimates the two load components on the structure, a maximum vertical uplift load and a maximum horizontal load, as a wave strikes the bridge superstructure. It was assumed, for this assessment analysis, that the maximum of each load component occurs at the same time, that each span acts independently of the others, that the wave-induced loads absorbed by the span are evenly distributed to the two pile bents at the ends of the span, and that these loads are evenly distributed to the four piles supporting the two pile bents. These assumptions are reasonable for this level of assessment.

The capacity of the connections was determined following the provisions for concrete strength outlined in ACI 318-11[24] using the details in the construction plans (e.g. bolt size and embedment depth, angle size, etc., see Figure 7). The vertical capacity of the connections (520 kips) was based on the dead load of the girder (504 kips) and the capacity of the bolts connected to the edge of

[23] FHWA "HEC 25: Volume 1."

[24] ACI Committee 318 "Building Code Requirements for Structural Concrete and Commentary."

the girder calculated using ACI section D.6.2. The horizontal capacity of the connections (110 kips) was determined from the concrete breakout strength of the connections (bolts connecting the angle to the bent beam) on the down-wave side of the girder as calculated using ACI section D.6.2 and the strength of the connections (bolts connecting the angle to the girder) on the up-wave side as calculated using ACI section D.5.2. As the waves impact the structure, the superstructure wants to "separate" from the substructure. The vertical wave forces are resisted by the dead load (or weight) of the superstructure and the connections between the superstructure and the substructure. The horizontal wave forces, however, are resisted by the connections alone. Since the structural capacity is greatly exceeded by the structural demand, the implication is that the connections will fail.

The next step in the load path analysis is to check the capacity of the superstructure itself if the connections were strengthened. There were four potential superstructure failure modes that were investigated: (1) transverse tension in the girder, (2) the connection of the girder to the bridge deck, (3) negative bending moment in the girder, and (4) negative bending in the bridge deck. The vertical uplift forces acting on the bridge superstructure are "pushing" upwards on the bridge deck while the connection between the super- and substructure are "pulling" down on the bottom of the girders. This will cause tension perpendicular to the length of the girders. The capacity of the girders for the transverse tension forces was determined by calculating the tensile forces in the shear reinforcement using basic mechanics and determining the tensile capacity of the shear reinforcement following the provisions in ACI 318-11[25] (i.e. limiting the strain to prevent yielding). Since the structural capacity exceeds the structural demand, as shown in Table 3, transverse tension in the girder is not a likely failure mode. The same forces that create tension in the girders also create tension between the bridge deck and the girders. This force is resisted by the shear reinforcement that is continuous from the girders into the cast-in-place deck. The structural capacity was calculated following ACI 318-11, in a similar manner as above, using the details of the construction plans. Since the structural capacity exceeds the structural demand, as shown in Table 3, the connection of the girder to the bridge deck is not a likely failure mode.

The vertical uplift forces from the waves cause negative bending (or bending in the opposite direction of dead and live loads) in both the girders and the bridge deck. The structural demand of these elements was determined by distributing the vertical wave forces over the surface area of the bridge span and calculating the moment in each element. The structural capacity of the girder and the deck for negative bending was calculated by following the provisions outlined in the AASHTO LRFD Bridge Design Specifications[26] and the PCI Design Handbook[27] using the details in the construction plans. Figure 13 shows a portion of a typical section of the bridge deck and

[25] ACI Committee 318 "Building Code Requirements for Structural Concrete and Commentary."
[26] AASHTO "AASHTO LRFD Bridge Design Specifications: Customary U.S. Units."
[27] PCI "PCI Design Handbook: Precast and Prestressed Concrete."

the reinforcement schedule. The detail indicates that there are number 5 reinforcing bars at the top and bottom of the deck in the transverse direction (the direction that would resist the negative bending due to the wave loading). The bars at the top of the deck are spaced at ten inches on center. The capacity of the bridge deck for negative moments was determined following the Strip Method of analysis (AASHTO 4.6.2.1).

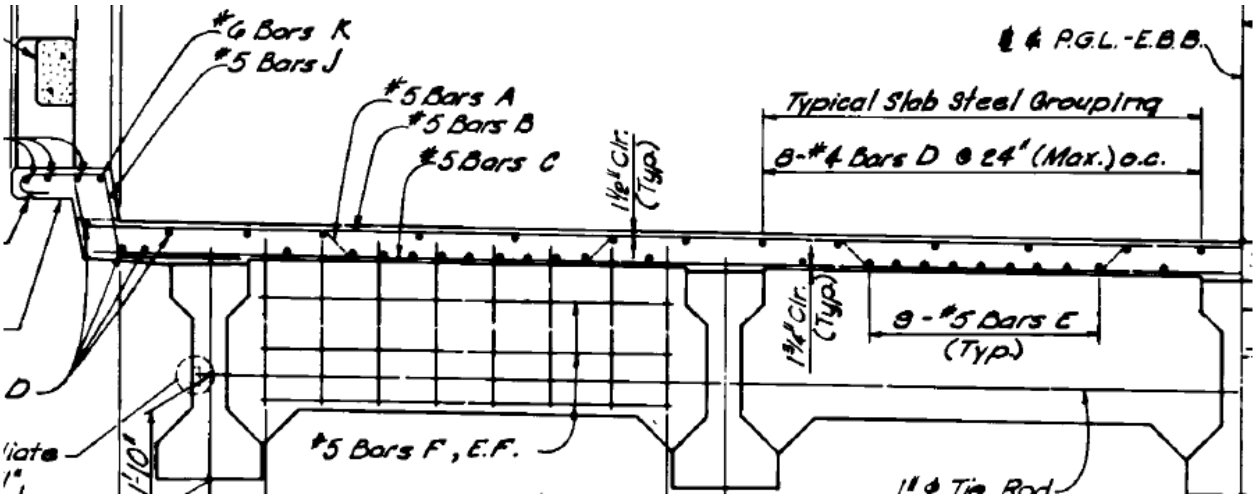

Figure 13: Construction plan showing typical section of bridge deck with reinforcement schedule.[28]

Figure 14 shows a portion of a typical section of the prestressed concrete girders. It can be seen that at midspan (where the maximum moments occur) there is very little prestressing steel at the top of the girder (two ½ inch diameter strands). The majority of the prestressing steel is located at bottom of the girder (twenty-eight ½ inch diameter strands) to resist the "positive" moments, but the girder has very little resistance to the "negative" moment induced by the wave loads. The capacity of the girders to negative bending included composite action with the deck and was calculated following AASHTO section 5.7. Since the structural capacity exceeds the structural demand, as shown in Table 3, negative bending due to wave loads in the bridge deck and in the girders are likely failure modes. This is a key finding of this assessment analysis and is something that has previously not been emphasized in the literature: if the connections between the superstructure and substructure are strengthened to survive wave loads, the superstructure will likely fail due to negative bending anyway. These calculations are just for this specific bridge and this specific scenario storm. However, this issue is likely going to problematic at any coastal bridge that is exposed to significant wave-induced loads in extreme events.

[28] Plans provided by the Alabama Department of Transportation.

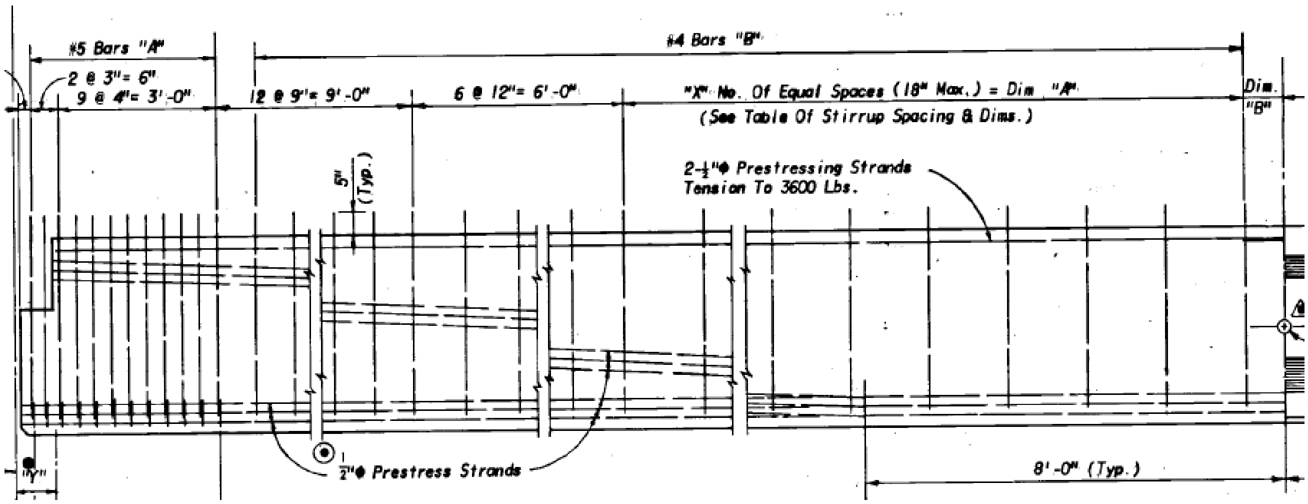

Figure 14: Construction plan showing typical section of a girder.[29]

The next step in the load path analysis was calculating the structural capacity of the substructure. The substructure analysis included investigating four failure modes: (1) bending in the bent beam, (2) moment capacity of the connection between the bent beam and the pile, (3) shear capacity of the connection between the bent beam and the pile, and (4) the structural capacity of the pile. The structural demand in the bent beams was determined by calculating shear and bending moment caused by the vertical uplift forces transferred through the connections between the super- and substructure. The capacity of the bent beam was determined by following the AASHTO LRFD Bridge Design Specifications. The moment capacity was determined by AASHTO section 5.7 and the shear capacity was determined by following AASHTO section 5.8. The connection of the pile to the bent beam was checked for both moment capacity and shear capacity. The moment demand on the connection was determined from the maximum moment at the top of the pile from the software program used to analyze the piles (LPILE). The shear demand on the pile was determined from the total horizontal wave force divided by the number of piles resisting the force (four piles, two at each bent). The capacities of the connections were determined following the same AASHTO LRFD Bridge Design Specifications as the bent beam. The final step in the load path analysis was determining the structural capacities of the piles. The structural demand on the piles was determined using the software program LPILE. The capacity was calculated two ways. First, LPILE was used to output the capacity of the pile, and then the capacity was verified by calculating the capacity following the AASHTO LRFD Bridge Design Specifications and the PCI Design Handbook.

The structural analyses results shown in Table 3 were calculated for the peak of the storm assuming that the bridge structure was still there. The analyses were extended to estimate the critical surge elevation for each of the critical failure modes. It was assumed, as above as a

[29] Plans provided by the Alabama Department of Transportation.

reasonable first approximation for these analyses, that wave conditions do not decrease significantly with decreased surge elevation. The subsequent failure by other mechanisms will likely occur later in the scenario storm as the surge levels increase and the wave-induced loads increase. These analyses provide a way to evaluate when on the rising hydrograph limb each failure mode would occur if another failure mode had not yet destroyed the bridge. The results for the I-10 Bridge are:

- The connections between the superstructure and substructure will likely fail when the storm surge elevation is around 14.9 feet,
- The bridge deck will likely fail due to negative bending when the storm surge is around 15.7 feet,
- The girders will likely fail due to negative bending when the storm surge is around 18.2 feet, and
- The piles will likely fail when the storm surge is at around 20 feet (see below).

A key finding of this engineering assessment is that failure due to negative bending of the deck could occur relatively shortly (in the storm) after failure due to the connection. Thus, strengthening the connections alone will really not "buy" much more storm resilience - less than a foot of storm surge. Considering the timing of the rising hydrograph of the selected scenario storm shown in Figure 6, this implies that the bridge could fail due to negative bending about 15 minutes later even if the connections were strengthened. Consideration could also be given to a retrofit adaptation which increased the capacity of the bridge deck to withstand negative bending. One such conceivable retrofit adaptation might include addition of more deck weight by increasing the thickness of the deck (and perhaps adding reinforcing). However, if this was done a full analysis of the capacity of the structure, as well as the capacity of the foundation system would need to be performed.

Geotechnical Evaluation

A geotechnical engineering evaluation of the pile foundation was performed to determine its capacity due to both uplift and lateral forces. The result of this geotechnical evaluation is that the substructure likely will not be strong enough to withstand the wave-induced loads at the peak of the storm even if they could be successfully transferred from the superstructure to the substructure. To be consistent, the geotechnical analysis assumed that each bridge span was independent and the wave forces would only impact a single span at a time. This assumption may not be conservative for design but is appropriate for this assessment.

The first geotechnical analysis performed was a comparison of the uplift load to the estimated uplift resistance of the pile, which includes the weight of the decking, the girders, the pile bent beam, the weight of the piles, and the shaft resistance generated by the interaction of the pile with the embedded soil. The shaft resistance due to the interaction of the pile with the surrounding soils was determined by utilizing the total stress analysis set forth in the FHWA NHI-05-042 design reference manual. The estimated shaft resistance (R_s) for each pile is 390 kips, however using the recommended factor of safety of three this would equate to 130 kips. The total uplift resistance of each bent, including the weight of the structure and the shaft resistance, is 1,068 kips. The total uplift force due to the wave loads is 775 kips (assuming evenly distributed between the substructures at each end of the deck; and the adjacent deck is not contributing uplift loads). Therefore, the conclusion is that the foundation system may have enough capacity to resist the extremely large wave-induced uplift forces. But it should be noted that changing the initial assumption that wave loads only act on a single span, which is based on an assumption of short-crestedness in the storm wave field could change this conclusion

The second geotechnical analysis was performed to determine the overall capacity of the foundation to the combined lateral and uplift loads induced by the waves. Due to the complexity of a coupled lateral and uplift analysis, the software program LPILE 2015 (Ensoft, Inc.) was utilized. The lateral analysis was performed on a single pile in shear and bending using a non-linear soil response under cyclic loading conditions. A pushover analysis of the pile was performed to determine the deflection at the top of the pile as a function of the shear force (lateral wave-induced load). Common industry practice is to set a deflection limit of one to two inches for serviceability and six inches for survivability. The lateral loads would cause greater than 6 inches of deflection (for any level of assumed fixity of the head condition). This analysis assumed that the total demand on a single pile is one-fourth of the lateral, wave-induced load or 127 kips assuming the load is evenly distributed between the four piles considered to support one span. The conclusion is that the pile may not have enough strength to withstand the extreme wave-induced loads at the peak of the scenario storm. This pile failure would be due primarily to the lateral load. Thus, the overall result of the geotechnical evaluation is that the existing bridge substructure likely may not be strong enough to withstand the wave-induced loads at the peak of the storm even if all the structural failure modes were addressed with sufficient adaptations.

Adaptation: Strengthened connections between the bridge girder and bent beam

One of the key original findings of this engineering assessment is that retrofit adaptations that just strengthen the connections between the decks and the substructure alone will likely lead to destruction of the bridge due to other failure mechanisms (deck and girder damage due to negative bending and, ultimately, pile damage). The existing bridge girder-deck system is designed for substantial dead and live loads acting downward on each individual span. These downward loads cause substantial bending forces, or moments, in the middle of the deck and the design of the decks is largely controlled by these downward loads. The typical girder-deck design is efficient at withstanding these (positive) bending moments. For example, most of the prestressing and reinforcing steel is at the bottom of the girders specifically to handle the bending forces due to these downward acting loads. However, wave-induced loads primarily impart an upward directed force that causes the individual spans to bend in the other direction – called negative bending. And the existing bridge decks are not specifically designed to withstand any negative bending induced by upward acting loads. The primary resistance to negative bending in the existing design is just the dead load or weight of the deck structure itself. Those weights are incorporated into the capacity listed in Table 3.

The second connection detail considered, shear block, was also assessed for its potential performance. As previously mentioned, this type of connection resists horizontal displacements, but does not resist vertical displacements. Since the vertical displacements are not resisted, a vertical reaction is not developed and the only force resisted in the vertical direction is the weight of the bridge span. Based on the load path analysis, Table 3, several potential failure modes are eliminated by eliminating the vertical reactions, namely negative bending in the girders and bridge deck. Additionally, there is a potential reduction in the horizontal load, however, no research to date has been performed on the behavior of shear block type connections including the effects on horizontal loads, the potential for the bridge superstructure to lift up and over the shear block, and the negative effects of the impact caused by the superstructure dropping after the passing of the wave. For this assessment an approximate analysis was performed to evaluate the use of shear block type connections. To determine the amount of upward movement expected in the bridge superstructure, the height when the loads were equal to the dead load was determined. This was done in-lieu of a more detailed analysis (i.e. energy method) due to the lack of research into these systems. The approximate analysis is conservative in the sense that it is expected to overestimate the height that the girders move as compared to a more detailed analysis or laboratory testing. Based on the analysis, the bridge superstructure would be expected to move upward approximately 6.25 feet. The limiting factor for the height of the shear blocks is the distance from the top of the bent beam to the bottom of the bridge deck, which is approximately 3 feet. Therefore, the superstructure would be expected to fail due to moving up and over the top of the shear keys. However, the approximate analysis performed is extremely conservative and the results of a more refined analysis or experimentation would be expected to show a much smaller displacement of the deck. More research is needed before this type of adaptation should be considered.

Adaptation: Improved span continuity

The second adaptation assessed was improving the span continuity for vertical uplift loads, this adaptation was evaluated based on the load path analysis. Since the analysis assumed only one adjacent bridge span would be impacted by a wave simultaneously, this adaptation would increase the effective dead load of the span that was impacted. Although this adaptation would reduce the demand on the connections between the super- and substructure, it would not eliminate the failure modes of negative bending in the girders and bridge deck. Therefore, it was determined that this adaptation alone could still lead to destruction of the bridge.

Adaptation: Modified bridge shape

Unfortunately at this time, no existing design methodologies, or even any research results, were identified to evaluate this possible adaptation option. Thus, analysis of this adaptation option was not emphasized in this assessment. There is some research on the effects of streamlining bridge cross-sections in unidirectional flow based on small-scale physical model testing at the Turner-Fairbank Hydraulics Research Center (Hydrodynamic Forces on Inundated Bridge Decks, Publication No: FHWA-HRT-09-028, May 2009). That research found drag forces could be reduced by as much as half for a streamlined bridge cross-section as compared with a typical girder-deck bridge cross-section.

For discussion purposes, that result (loads are 50% of those on the typical girder-deck bridge cross-section) from that research is applied to this much different situation. Specifically, the reductions in peak loads in the oscillatory wave flow situation are assumed to be of a similar magnitude to the reductions in drag forces found in the uni-directional flow situation. This reduction is only included for the lateral loads in this assessment. Uplift loads could also be reduced but it is assumed here that they are not. It is recognized that there are a number of problems with this analogy. In waves there are drag force components and inertial force components due to accelerations in the underlying flow field which are not present in uni-directional flows. Also, the air trapping that leads to slamming forces in the wave environment are not an issue in uni-directional flows.

The lateral loads on bridge decks are typically smaller than the vertical loads and thus the value of shape changes which reduce them might be questioned. However, the load path evaluation described below found that the horizontal component of the wave-induced loads is more critical in several considerations. The structural engineering analysis in this assessment indicates that the connections break primarily due to the lateral wave-induced loads. The weight of the structure itself provides some significant resistance to the vertical wave-induced loads. Nothing but the connections resists the lateral loads. Also, the geotechnical engineering analysis (below) concludes that while the soils may be able to withstand wave-induced uplift loads alone, the soils cannot withstand the combination of lateral and vertical loads. Thus, there might be some value in research on reductions in wave-induced loads afforded by more hydrodynamic shapes.

Adaptation: Increased deck elevation in combination with some of the above

Increasing the deck elevation in combination with one or more of the adaptation designs described above, the fourth adaption considered here, is a viable adaptation option. This is particularly viable for new construction design. Using the same methods as above, an analysis was performed to determine the expected reductions in load due to an increase in the deck elevation. This is similar to the questions which might be addressed in establishing the design bridge elevation if the selected storm scenario were the “design storm.”

Table 4 shows the resulting elevations required to survive the scenario storm if different possible adaptation options were included in the design. The first row in Table 4 also shows the bridge elevation which might be set if the selected storm scenario were the "design storm" for the bridge. Such a rare storm would likely not be the design storm but could be a "check event" storm such as the 500-year storm is often considered a "check event" for bridge scour designs. The first line of Table 4 shows that the low-chord bridge elevation would have to be above +31 feet (16.5+14.9) in order to satisfy a typical design procedure used today with this scenario storm. The first two lines In Table 4 indicate that the typical design procedure would set the bridge deck elevation about 7 to 8 feet higher than it would have to be set to survive even if no additional adaptations were employed. The implication is that the typical design procedures may be leading to bridge deck elevations which are higher than needed to survive the design storm. This additional elevation represents, however, additional resilience to extreme events and climate change. Table 4 also indicates that with the other adaptations, it may be possible to set the bridge deck elevation to an elevation only 3.3 feet above the existing elevation and have it possibly survive the selected scenario storm. It should be noted that the values in Table 4 are based on the load estimates and that actual design decisions would include factors of safety, or other decisions to reduce the probability that capacity levels are not exceeded, which were not included in this analysis.

Table 4: Deck elevation increase above the existing elevation (+16.5 ft. MSL) required to survive the selected storm scenario.

Adaptation Method Used in Combination with Increased Deck Elevation	Additional Deck Elevation to Survive the Selected Scenario Storm
Typical design procedure (1 foot of freeboard above highest wave crests at surge peak)	14.9 feet
Existing connection design (no other adaptation)	7.4 feet
Increased connection strength	6.6 feet
Increased span continuity	5.3 feet
Increased connection strength and deck reinforcement	4.1 feet
Shear key type connection	3.3 feet

Summary of Performance of Adaptation Options

Four adaption options were considered and evaluated for this bridge structure: (1) strengthening the connections (two options were evaluated), (2) increasing span continuity, (3) modifying the shape of the bridge deck to reduce lateral wave-induced loads, and (4) increasing the deck elevation in combination with the other adaptations. None of the adaptations by themselves, with the exception of increasing the deck elevation, were found to be adequate to ensure the survival of the structure in the scenario storm. For example, if the existing connections between the bottom of the girders and the bent beam were reinforced, the bridge deck and girders would fail under the wave-induced loads anyway (when the storm surge elevation reached +15.7 feet).

The failure mechanism would be negative bending beyond the capacity of the existing bridge design. This means that in addition to reinforcing the connections, the girders and bridge deck would also need reinforcement. Although no single adaptation was found to be adequate, each adaptation would increase the capacity of the structure, and the increase in capacity would improve the survivability of the structure under less intense storm events.

Figure 15 and Table 5 summarize some of the adaptations considered, their failure modes, and the estimated storm surge still water level (SWL) at failure. Figure 15 summarizes the findings of this study schematically with the bridge, surge and wave geometry shown to scale. The blue wave form shown represents a wave moving to the right, just before impact with the structure, which is consistent in shape (wave height, wave length, water depth, and water surface shape[30]) to the maximum wave in the scenario storm at the different surge elevations.[31] [32] Y_{max} is the elevation of the crest of that maximum wave. The bridge will be damaged as the connections fail at a storm surge of around 14.9 feet without any adaptations (Figure 15a). Even with strengthened connections, the bridge will likely fail by another failure mechanism, negative bending, when storm surge is around +15.7 feet (Figure 15b). And even with strengthened connections and a strengthened deck/girder system, the bridge will likely fail by other damage mechanisms, when the storm surge has reached +20 feet (Figure 15c). Figure 15d shows the relative position of the waves, surge, and structure at the peak surge (+22.3 feet) of the storm. Table 5 summarizes these same findings of this study in terms of the adaptation options evaluated, expected failure modes and surge elevation at failure during the selected storm scenario.

[30] Using cnoidal wave theory to estimate wave form.
[31] FHWA, "HEC25: Volume 1."
[32] US Army Corp of Engineers, "Shore Protection Manual."

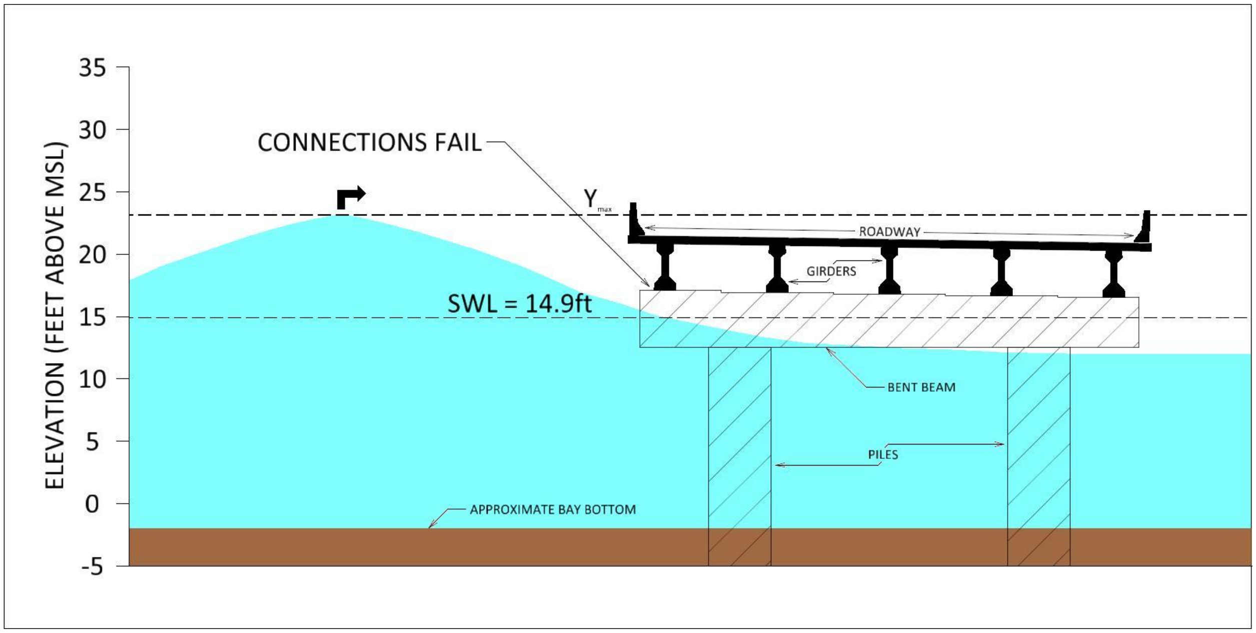

Figure 15a: The findings of this study indicate that the bridge will be damaged by failure of the existing connections when the storm surge reaches about +14.9 feet.

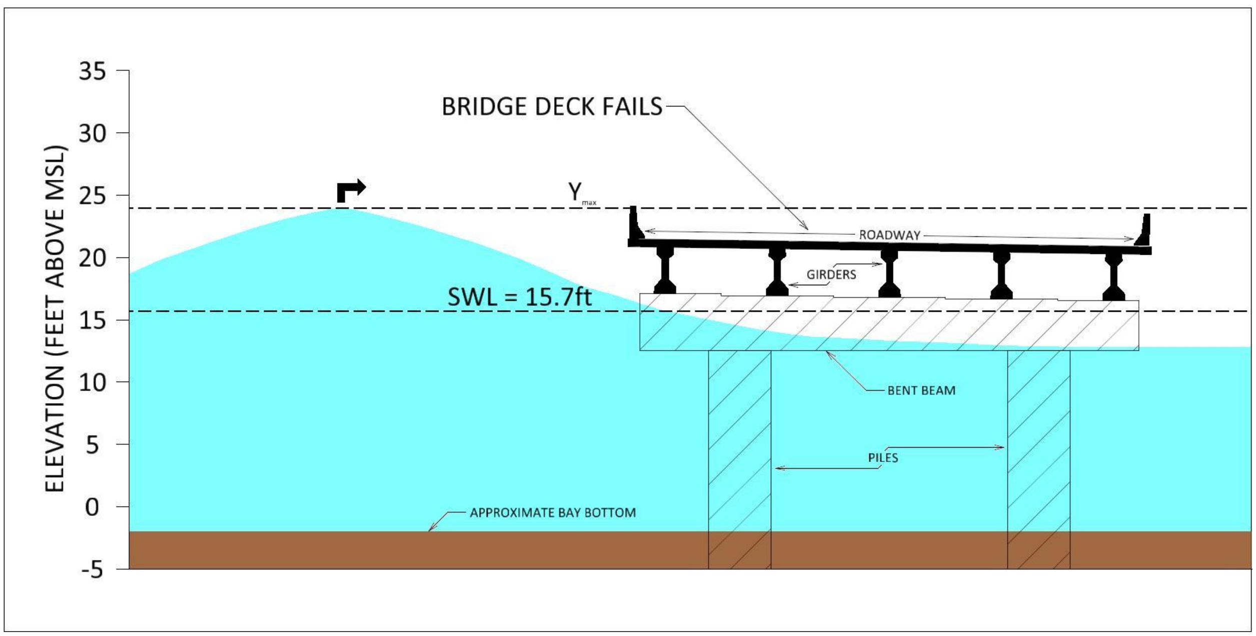

Figure 15b: The findings of this study indicate that the bridge will be damaged by failure of the bridge deck, due to negative bending in response to the wave-induced uplift loads, when the storm surge reaches about +15.7 feet (even if the connections are strengthened).

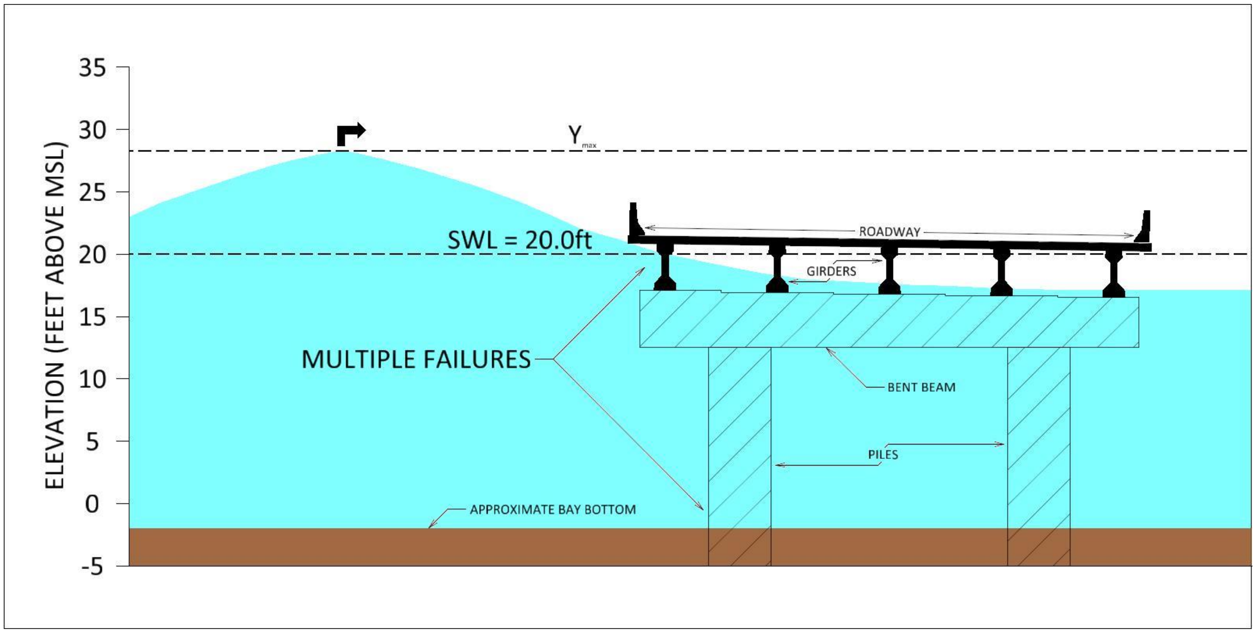

Figure 15c: The findings of this study indicate that the bridge will be damaged by failure of the bridge, due to multiple failure mechanisms including pile failure and geotechnical failure in response to the wave-induced loads, when the storm surge reaches about +20 feet (even if the connections and the deck are strengthened).

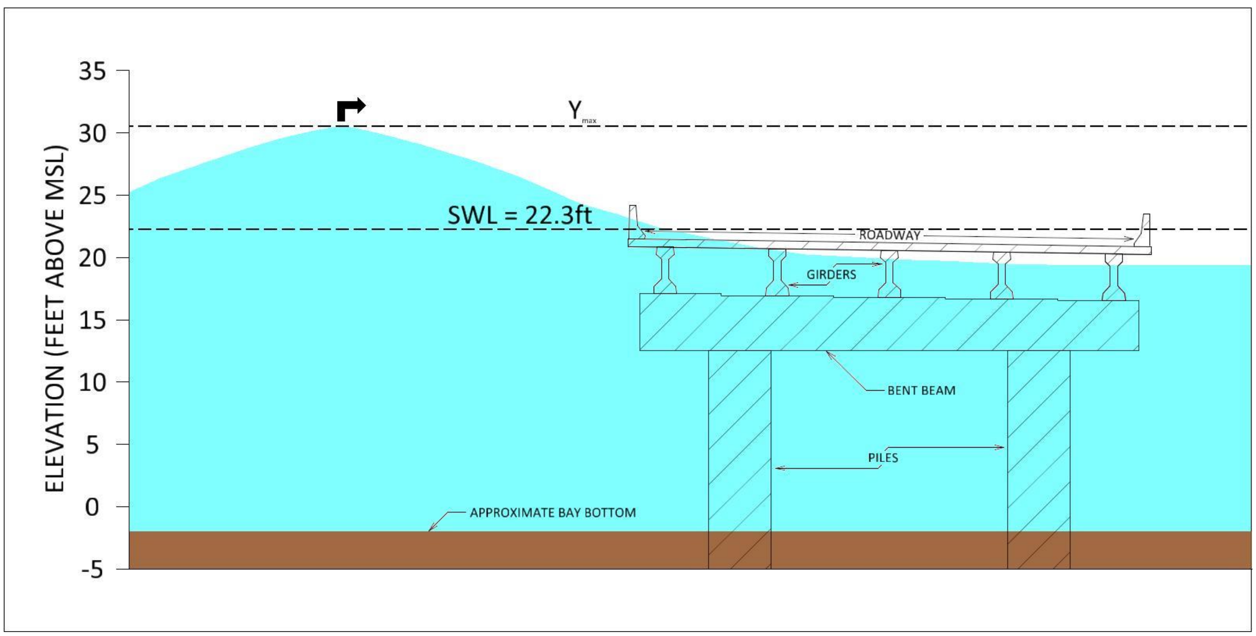

Figure 15d: The findings of this study indicate that this bridge will be damaged by wave-induced loads prior to the peak (+22.3) of the storm surge for a hurricane like Hurricane Katrina but making landfall closer to Mobile with +75 cm (2.46 feet) of sea level rise. This damage will occur even with any of the adaptations considered here (except raising the elevation of the bridge).

Table 5: Summary of some of the adaptation options evaluated, expected failure modes and surge elevation at failure during the selected storm scenario (peak storm surge of +22.3 feet).

Adaptation Method	Description (and Other Adaption)	Failure Mode	Storm Surge Elevation at Failure (SWL)
None	Existing conditions	Failure of girder to bent beam connection	14.9 feet
Increased Connection Strength	No other adaptation	Deck/girder failure	15.7 feet
Increased Connection Strength	Deck reinforcement	Girder failure	18 feet
Increased Connection Strength	Deck and girder reinforcement	Pile foundation (displacements)	18 - 20 feet
Increased Connection Strength (Shear Key)	No other adaptation	Deck moves up and over shear key	19 feet
Increased Span Continuity	No other adaptation	Connection failure	17 feet
Increased Span Continuity and Shear Key	Combination of adaptations	Pile foundation (displacements)	18 feet

Step 8. Conduct an Economic Analysis

The I-10 Bridge and the parallel US-90 Bridge are critical regional connectors; together they transport over 88,000 vehicles per day and more than 31,000 passenger vehicles during peak commute hours. Because of their regional importance, a storm surge event that compromises the use of the I-10 and US-90 bridges and requires a lengthy 40-mile detour will likely have a negative impact on both users of the bridges as well as the broader regional economy, driving up costs for passenger and freight traffic and discouraging tourism and other leisure trip spending in downtown Mobile. This analysis evaluates the *direct costs* associated with bridge closure for road-users in terms of increased transportation costs associated with alternative routes. It also explores the potential magnitude of impact that a bridge closure could have on downtown Mobile's economy due to loss of business activity and the subsequent 'ripple effect' that this lost activity could have on the economy of the greater Mobile-Daphne-Fairhope CSA; inclusive of both direct and *indirect* impacts (costs). For purposes of this analysis, we assumed that since the I-10 Bridge and the US-90 Bridge are parallel, if the I-10 Bridge is disrupted, the US-90 Bridge would be impassable as well.

This analysis focuses on quantifying the costs associated with a potential closure of the I-10 Bayway Bridge; the benefits of access can be considered as the avoidance of these costs. Unlike in the other TEACR engineering assessments, this economic analysis does not estimate the cost of the adaptation options proposed in Step 7. The economic assessment is simply designed to quantify the impacts of losing the bridge which helps engineers understand the potential value of investing in adaptation strategies.

Introduction to the Economic Model

To estimate the economic impacts of a potential bridge closure on the regional economy, the research team used the economic impact modeling software IMPLAN, which is created and maintained by the Minnesota IMPLAN Group (MIG) and widely used throughout the U.S. The IMPLAN model is a static input-output framework used to analyze the effects of an economic stimulus on pre-specified economic regions; in this case, the city of Mobile and the broader Mobile-Daphne-Fairhope CSA. The IMPLAN model is based on the input-output data from the U.S. National Income and Product Accounts (NIPA) from the Bureau of Economic Analysis. The model includes 536 sectors based on the North American Industry Classification System (NAICS). The model uses state-specific multipliers to trace and calculate the flow of dollars from the industries that originate the impact to supplier industries. These multipliers are thus coefficients that "describe the response of the economy to a stimulus (a change in demand or production)."

There are three types of impact outputs calculated by IMPLAN:

- **Direct impacts**, which are impacts in the primary industries where spending by consumers would be focused. In this sensitivity analysis, all of the reduced economic activity was assumed to occur in leisure sectors—those associated with tourism, entertainment, sports, dining, and arts. When consumers spend less, these industries experience a reduction in direct revenue, employment, etc.
- **Indirect impacts**, which are impacts in the industries that supply or interact with the primary industries. For example, when consumers spend less at restaurants, a decline in business activity will also be felt in the industries that supply food to the restaurant.
- **Induced impacts**, which represent spending by workers who rely on money from the industry activity in the primary and secondary sectors. For example, when a restaurant employee has less income to spend at a bookstore, this represents an induced impact in the book retail sector.

The total impact is the sum of indirect and induced impacts that remain in the region (as opposed to "leaking out" to other regions or states). IMPLAN then uses this total impact to calculate subsequent impacts such as total jobs created and tax impacts.

Economic Analysis Methodology

The research team estimated the direct costs incurred by bridge users forced to take a detour route, as well as the direct and indirect impacts felt in downtown Mobile and the broader Mobile-Daphne-Fairhope economy due to reduction in tourism and other leisure trips. The research team estimated these impacts by taking the following two steps:

1. Quantifying direct costs associated with disruption to primary users
2. Estimating the region-wide impacts of the I-10 Bayway Bridge closure on the broader economy of Mobile and the Mobile-Daphne-Fairhope CSA

Direct costs are those that are immediately incurred by primary users of the bridge, such as passenger vehicles that are now subject to a detour route. Broader economy-wide impacts are those costs that “ripple” through the regional economy as a result of bridge closure. While we are not able to predict the actual value of business in downtown Mobile that would be disrupted due to lack of convenient access from the eastern part of the Mobile-Daphne-Fairhope CSA, we can evaluate the impact across a range of disruptions.

Quantify Direct Costs

First, the research team quantified the costs associated with a disruption in the use of the I-10 Bridge on the bridge’s primary users—freight vehicles and passenger vehicles that use the bridge during peak commuting times. The analysis focuses on peak travel times because these trips are assumed to be work commutes. The intent in focusing on work commutes is that they are the most inelastic (individuals are largely unable to avoid or shift around these commutes if they are required to be in an office for a traditional work schedule) and therefore are most likely to incur economic costs. Additionally, as these trips are most likely to intersect with business activity and the broader economy, they are the most likely to incur further economic costs. The estimates of direct costs are considered conservative in that they do not represent other types of trips (leisure, etc.) during non-peak times. These direct costs include the additional travel time incurred by passenger vehicles, the additional operation and maintenance costs incurred by passenger vehicles, and the operation and maintenance costs incurred by freight vehicles. The research team used average daily traffic (AADT) data for both commuter vehicles and freight to quantify the number of passengers and freight vehicles directly impacted per day. Then, in order to determine the incremental difference in direct cost for the primary users to take the alternate route, the analysis established a baseline route between two points across the Bayway Bridge. This baseline route is presented below in Figure 16.

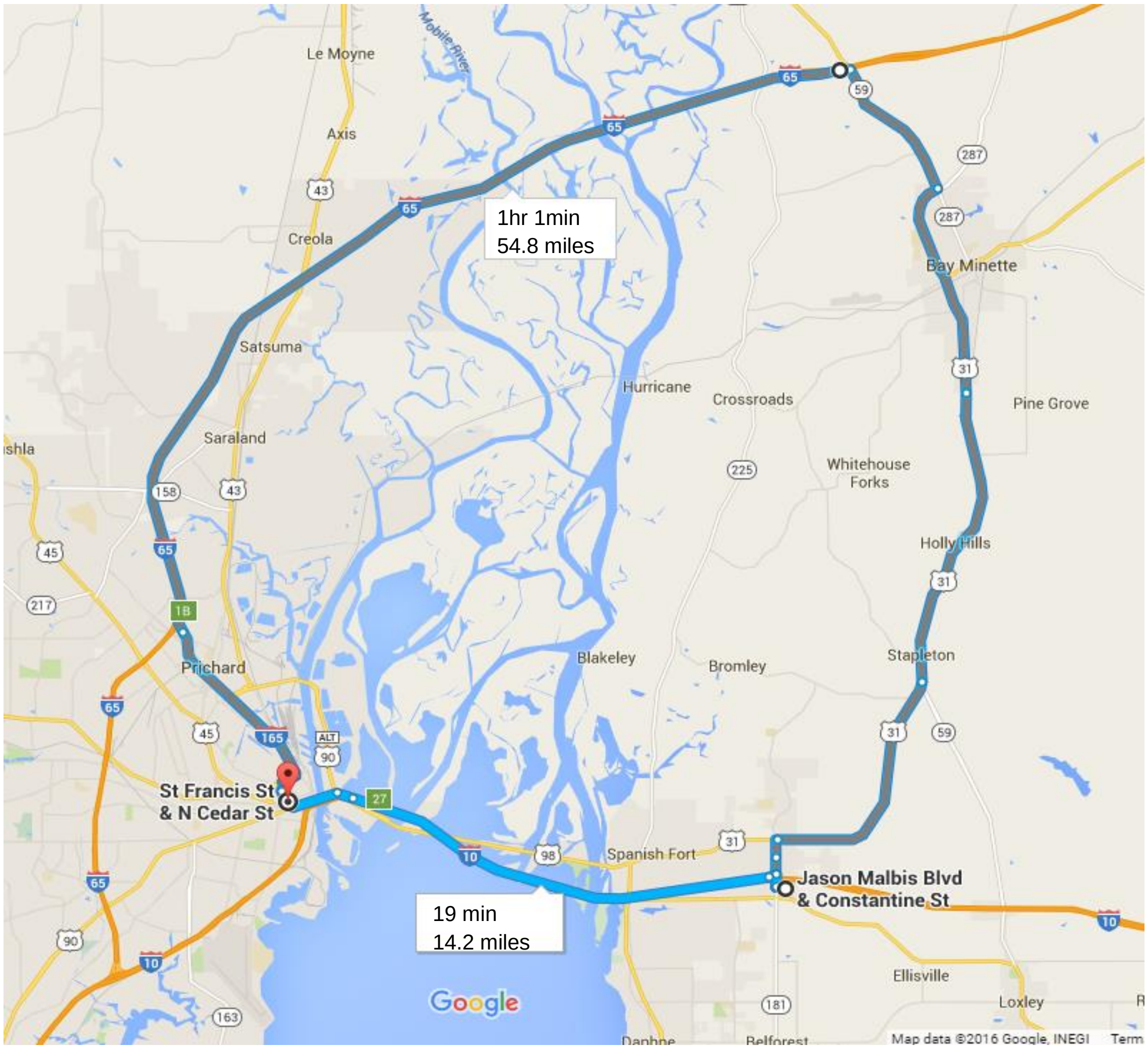

Figure 16: Baseline & Alternate Route. The baseline route is presented in blue along the bay on I-10. The alternate route is presented in gray, spanning north on I-65 and Route 31.[33]

The alternate route was determined using the same start and end points as the baseline route, but using other main thoroughfares to travel. The most reasonable alternative route is presented in Figure 16. This route adds 40.6 miles and approximately 42 minutes to travel across the bay in either direction.

The difference between the baseline and alternative routes in terms of miles traveled and time traveled constitute the incremental difference in direct impacts for most of the analysis.

For passenger traffic, the research team used a methodology that is often used to estimate the impacts of congestions to quantify the value of lost worker productivity and "wasted" fuel due to the required use of a less direct, alternate route. We accounted for both additional distance

[33] Source: Google maps.

and time to travel the alternate route. The value of Additional Commuter Travel Time (ACTT) was used to determine lost wage productivity and was derived from the incremental time difference between the baseline and alternate route (see maps above). To calculate this incremental time difference, we calculated the difference in travel time between the baseline route (approximately 19 minutes) and alternate route (approximately 61 minutes) during peak commute time (8 a.m.) Peak time was used to reflect likely congestion and greater traffic volume, as well as most representative of the traffic volumes facing peak commuters. Additionally, we used the incremental mileage difference between the baseline and alternative route to estimate Additional Commuter Vehicle Miles Traveled (ACVMT), which was used to determine "wasted" fuel and vehicle wear.

ACTT and ACVMT were then monetized using secondary source information gained through a literature search. ACTT was monetized using the national average value of time of $17.67[34] from the 2015 Texas A&M Urban Mobility Scorecard. ACVMT was monetized using the IRS Standard Mileage Rate of 57.5 cents per mile.[35] See Table 8 below for summarized value of the magnitude of impact on passenger and truck vehicles.

For freight traffic, the research team quantified the increase in business costs due to additional time needed for alternate routing. Using the freight average daily traffic (FAADT) for the I-10 corridor – derived from the percentage of freight traffic as a percentage of AADT - we estimated the Additional Freight Travel Time (AFTT) derived from the incremental travel time difference between the baseline and alternate route. Again, AFTT was monetized using the freight travel and operating costs from the 2015 Texas A&M Urban Mobility Scorecard, which is $94.04 per hour. These calculations, presented in Table 6 (input variables), Table 7 (calculations), and Table 8 (results), allowed the research team to quantify the direct magnitude of loss for the key users of I-10 and US-90 due to a disruption in its use.

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[34] The Texas A&M Transportation Institute Annual Mobility Scorecard is accessible at: http://mobility.tamu.edu/ums/.

[35] "The standard mileage rate for business is based on an annual study of the fixed and variable costs of operating an automobile" - The Internal Revenue Service (IRS) 2016 Standard Mileage Rates for Business, Medical, and Moving is accessible at: https://www.irs.gov/uac/Newsroom/2016-Standard-Mileage-Rates-for-Business-Medical-and-Moving-Announced.

Table 6: Variables for Direct Cost Impacts.

Variable	For Equations	Value	Source
Annual Average Daily Traffic on I-10 (AADT I-10)	A	70,195 vehicles	Alabama Traffic Data 2014
Annual Average Daily Traffic on US-90 (AADT US-10)	B	18,630 vehicles	Alabama Traffic Data 2014
Annual Average Daily Traffic (AADT)	C	88,825 vehicles	Alabama Traffic Data 2014
% Freight Traffic on I-10 (FT I-10)	D	0.15%	Alabama Traffic Data 2014
% Freight Traffic on US-90 (FT US-90)	E	0.04%	Alabama Traffic Data 2014
Length of Peak Traffic (LP)	F	3hours	South Alabama Regional Planning Commission
Vehicle Occupancy (VO)	G	1.04 persons per vehicle	Provided by Mobile County ACS data
Additional trip distance on alternate route in miles (M)	H	40.6 miles	Google Maps
Additional travel time on alternate route in hours (ATT)	I	0.71 hours	Google Maps
Average value of commuter time (VCT)	J	$17.67	Texas A&M Urban Mobility Report
Average vehicle operating, fuel, and depreciation costs per mile ($/mile) (VOC)	K	$0.575	IRS
Value of freight travel time ($/hour) (VFT)	L	$94.04	Texas A&M Urban Mobility Report

Table 7: Calculations for Direct Cost Impacts.

Variable	For Equations	Value	Formula
Freight Annual Average Daily Traffic on I-10 (FAADT I-10)	M	10,529.25 freight vehicles	A*D
Freight Average Daily Traffic on US-90 (FAADT US-90)	N	745.2 freight vehicles	B*E
Annual Average Daily Freight Traffic (FAADT)	O	11,274.45 freight vehicles	M+N
Annual Average Daily Passenger Traffic (PAADT)	P	77,550.55 vehicles	C-O
Average Peak Hour Passenger Volume (PHPV)	Q	5,170.04 vehicles	(P/24)*1.6
Annual Average Daily Passenger Traffic that occurs during peak hours (PAADT)	R	15,510.11 vehicles	Q*F
Total Peak Commute Vehicle Trips affected (TCVT), per day	P	15,510.11 trips	Assumed to equal PAADT
Total one-way Commute Person Trips affected (TCPT), per day	S	16,130.51 persons	P*G
Total Freight Trips affected (TFT), per day	O	11,274.45 trips	Assumed equal to FAADT
Additional Commuter Vehicle Miles Traveled due to loss of bridge (ACVMT), miles per day	T	629,710.47 miles	P*H
Additional Commuter Travel Time due to loss of bridge (ACTT), hours per day	U	11,50.43 hours	S*I
Additional Freight Travel Time due to loss of bridge (AFTT), hours per day	V	8,042.44 hours	O*I
Lost Wage Productivity (LW), per day	W	$203,319	U*J
Additional Commuter Vehicle Operating Costs (ACVC), per day	X	$362,084	T*K
Additional Freight Operating Costs due to loss of bridge (AFOC), per day	Y	$756,311	V*L

Table 8: Magnitude of direct impact on passenger and freight vehicles.

Impact Variable	Value
Additional Commuter Travel Time due to loss of bridge (ACTT), hours per day	11,506 hours
Additional Commuter Vehicle Miles Traveled due to loss of bridge (ACVMT), miles per day	629,710 miles
Additional Freight Travel Time due to loss of bridge (AFTT), hours per day	8,042 hours

Estimating Regional Impacts

Next, the research team assessed the impact of the I-10 Bayway Bridge closure on the broader economy of Mobile and the Mobile-Daphne-Fairhope CSA in terms of *lost economic opportunity*. Most commuters and freight directed to Mobile will be forced to take a more expensive alternative route in order to access jobs and customers. However it is expected that a portion of non-business trips (i.e. shopping, tourism, leisure, etc.) that require traveling across I-10 will not occur since increased cost and inconvenience may be enough of a deterrent for optional trips. Since downtown Mobile's leisure sectors also serve local populations and those coming to Mobile from the west, business activity would be reduced in proportion with the demand that required access by way of the I-10 Bridge from the east.

Because it was beyond the scope of this analysis to conduct travel demand analysis to assess trip elasticity for I-10 Bayway Bridge traffic, the research team used trip origin and destination data provided in the US Census Bureau OnTheMap Application to approximate the current usage and provide an upper bound of the likely reduction in demand. Of the approximately 13,000 total primary jobs in Mobile Alabama, 16% (2,100) of those accessing those jobs live in Baldwin County, located to the east of Mobile across the bay.[36] We assume this population would be the primary users of the I-10 Bayway Bridge for both work-based and leisure trips and thus approximated the upper bound of reduced activity at 16%. This estimate is further rationalized by evaluating the proportion of the Mobile Metropolitan area consumer population that lives in metropolitan Baldwin County. Using US Census Bureau population estimates, the research team determined that the metropolitan area of Baldwin County represents approximately 20% of the population of the greater Mobile Metropolitan area, 49,000 and 259,000, respectively. While the metropolitan area in Baldwin County hosts recreational amenities of its own, it is assumed that a significant portion of consumers travel across the I-10 Bayway Bridge for amenity shopping. In the absence of more precise demand modeling, the research team used 16% to represent the upper-bound and 5% as the lower-bound of reduced economic activity for the sensitivity analysis of impact.

[36] The U.S. Census Bureau OnTheMap Application is accessible at: http://onthemap.ces.census.gov/.

To estimate the direct and indirect economic impacts of reduced usage of the I-10 Bayway Bridge, the research team used the economic impact modeling software IMPLAN. Data is input as a dollar value representing a change in demand (e.g. increased or decreased spending or the addition of a new business) in a given industry sector. The sensitivity analysis included sectors that are most likely to be affected, such as restaurants, bars, museums, performing arts venues, and other leisure-related services. IMPLAN then analyzes how money spent in a combination of sectors will translate into regional economic benefits—job creation, taxes, increased GDP, etc. Results in this analysis are presented in terms of total impacts, i.e. the sum of direct, indirect, and induced impacts on the local economy.

The research team used IMPLAN's multi-regional analysis function to assess the impact that reduced spending in the City of Mobile would have on the County of Mobile and neighboring Baldwin County. First, it was necessary to define the city area using ZIP code data from the United States Postal Service (USPS). All ZIP codes within the City of Mobile as defined by USPS were included, with the exception of 36612 and 36613 which have very little area within city boundaries. ZIP codes listed as "unique" ZIP codes by USPS were also not included because the IMPLAN model did not include data for ZIP codes assigned to specific companies or organizations.

Once the study area was defined, the research team configured the IMPLAN model for the City of Mobile and inputs associated with a 5% and 16% decrease in total GDP for each tourism-related sector. The reductions from the industry GDP were input as negative values into the City model and the multi-regional model comprising Mobile and Baldwin Counties to assess the City-specific impact of lost spending as well as its ripple effect across the region.

Economic Analysis Findings

The research team concluded that the use of the alternate route for the 31,020 passenger vehicles that use the bridge on a daily basis would result in total direct costs of $739,354 per day. This estimate includes both the value of increased passenger travel costs ($362,084) and lost worker productivity ($203,319) per day.

For commercial freight traffic, the impact per day would be roughly $756,311 due to increased travel time and costs. The commercial freight cost impacts will likely be passed on to consumers, increasing the costs of goods for locals as well as customers nation-wide.

The increase in daily travel costs for commuters, while significant to individuals, is unlikely to have major economic ramifications in the region. However, increased passenger travel costs overall are likely to impact personal travel and purchase decisions, and these decisions could impact economic activity in downtown Mobile. Using the IMPLAN model, the research team estimated that the macro-economic impacts of this loss of activity would be approximately $48,000 to $156,500 in daily GDP in the City of Mobile and approximately $51,000 to $169,300 in daily GDP in the broader Mobile-Daphne-Fairhope CSA economy, depending on assumptions about the percentage reduction in leisure-related activity in Downtown Mobile. The results of this analysis are presented in Table 9 and Table 10. Sustained bridge disruption could have an even more significant impact, which could be calculated by multiplying the daily impact described above by the number of days of bridge disruption. Thus, the annual impact of a year-long bridge disruption could lead to GDP losses ranging from $18.9 million to $61.8 million.

Table 9: Impact of Lost Spending per Day within the City of Mobile.[37]

Reduction in GDP of Leisure-Related Activity	Employment	Labor Income	GDP	Industry Activity
5%	-1.3	-$29,737	-$48,002	-$92,316
16%	-4.3	-$96,978	-$156,545	-$302,030

Table 10: Impact of Lost Spending per Day within the Greater CSA Region.[38]

Reduction in GDP of Leisure-Related Activity	Employment	Labor Income	GDP	Industry Activity
5%	-1.4	-$31,758	-$51,912	-$99,000
16%	-4.5	-$103,569	-$169,296	-$323,895

Within both the City of Mobile and the greater CSA region, a reduction in economic activity would have a minimal impact on employment as it is unlikely that workers would lose their jobs during short-term bridge inaccessibility. That said, their reduced productivity could translate into a loss of wages equaling $100,000 within Mobile and $103,600 across the region per day. Furthermore, an extended period of bridge closure could result in actual job losses over time.

State and local governments would also experience loss in potential tax revenue because of a bridge disruption. Fewer trips into the city and less spending translates into untapped tax revenue, as indicated below in Table 11. The City of Mobile could expect to forgo between $7,150 and $23,310 in tax revenue per day of bridge disruption. Looking at the greater regional area, the tax loss increases to between $7,560 and $24,650 per day of bridge disruption.

[37] Table Source: IMPLAN Analysis, Compiled by ICF International.

[38] Table Source: IMPLAN Analysis, Compiled by ICF International.

Table 11: Daily Tax Impact by Region and Economic Activity Loss Scenario.[39]

Daily Tax Loss	5% Reduction in Economic Activity		16% Reduction in Economic Activity	
Region	City of Mobile	Greater CSA	City of Mobile	Greater CSA
Total State and Local Tax	-$7,150	-$7,560	-$23,310	-$24,650

The analysis also identified which sectors would feel the greatest effects of a reduction in economic activity. Within each region, the industries that would experience the greatest losses in GDP are not surprisingly concentrated in the leisure-related goods and services. In the city and the greater metro area, limited-service restaurants[40] would forfeit the greatest in daily GDP due to the bridge disruption. These industries and their respective impacts are described in Table 12.

Table 12: Top Industries with Greatest Losses in Daily GDP in the City of Mobile and Greater CSA Region, 16% Reduction Scenario.[41]

Industry	Daily GDP Loss – Mobile City	Daily GDP Loss – CSA Region
Limited-service restaurants	-$50,697	-$50,952
Full-service restaurants	-$25,517	-$25,526
Hotels and motels, including casino hotels	-$13,290	-$12,934
All other food and drinking places	-$12,720	-$12,759
Real estate	-$8,531	-$9,763
Owner-occupied dwellings	-$4,227	-$5,009
Wholesale trade	-$3,246	-$3,842
Other amusement and recreation industries	-$2,923	-$2,941
Management of companies and enterprises	-$1,711	-$973
Hospitals	-$1,554	-$2,092
Employment services	-$922	-$1,881

Economic Analysis Conclusions

The research team found that loss of the I-10 Bridge could result in significant costs to daily users of the bridge as well as the region-wide economy of the greater Mobile area. These costs come primarily in the form of direct costs to passenger and freight vehicles and indirect costs to the broader CSA economy. Loss of the bridge could directly cost primary users $1,130,800 per day and result in a daily loss of up to $323,900 in industry activity.

[39] Table Source: IMPLAN Analysis, Compiled by ICF International. Note that greater CSA values include the City of Mobile.

[40] Limited service restaurants offer fast food or quick service and include fast-casual restaurants, pizza restaurants, and cafés.

[41] Table Source: IMPLAN Analysis, Compiled by ICF International.

Step 9. Evaluate Additional Considerations

A variety of factors beyond purely economic considerations are important to making the right decision on a project. In this situation, the uncertainty around the effectiveness of the adaptation options should be considered. None of the adaptations described in Steps 6 and 7, except clearance above the wave crests, should be designed without further research on wave-induced loads on bridge decks. The methodologies for estimating these loads, including the HEC-25 methodology and the AASHTO methodology,[42] have tremendous uncertainty given that they have been developed primarily with limited, and unpublished, small-scale laboratory tests. The resulting estimates of wave-induced loads likely are not as well-understood or as precisely-known as would be required to form the basis of design. The purpose of this assessment was to use available technology to evaluate adaptation options for planning-level purposes.

Step 10. Select a Course of Action

The results from the economic assessment (Step 8) indicate that losing the bridge would result in significant costs to daily users of the bridge as well as the region-wide economy of the greater Mobile area. For this reason, it would be prudent for ALDOT to continue researching the effectiveness of the adaptation strategies. As indicated in Step 9, there is significant uncertainty associated with the methods used to determine the impact of wave-induced loads which limits the ability to select a final course of action at this time. However, elevating the Bayway (potentially in conjunction with one of the mechanical adaptations) appears to be the only course of action that would guarantee survival of the bridge during future storms.

Step 11. Develop a Facility Management Plan

This step was not completed for this analysis; however, it is recommended that ALDOT develop a facility management plan for this bridge to ensure that the Bayway continues to perform as designed. The plan should include ongoing monitoring as the climate changes and require that corrective actions be considered, if necessary.

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[42] AASHTO "Guide Specifications for Bridges Vulnerable to Coastal Storms."

Lessons Learned

Lessons learned in the process of this assessment include:

- High-resolution storm surge and wave modeling is a valuable tool for quantifying the vulnerability of coastal infrastructure. Such modeling is recommended in HEC-25: Highways in the Coastal Environment: Volume 2: Assessing Extreme Events. In this case, modeled surge and waves were available at the start of this assessment because of previous work in the Gulf Coast 2 Study.
- Following the load path implications through the entire structure is required in the design of engineering adaptations for coastal bridges exposed to wave-induced loads on storm surge.
    - Retrofit adaptations which strengthen the connections between the decks and substructure can be designed to avoid the primary, historical damage mechanism (separation of the decks from substructure).
    - However, such adaptations alone may lead to destruction of the bridge due to other, secondary, failure mechanisms (deck-girder damage due to negative bending and pile damage).
    - Strengthening the connections may only provide a limited increase in resilience, less than one additional foot of storm surge elevation, for this bridge.
- None of the adaptation options, with the exception of increasing the deck elevation, were found to be adequate in ensuring the survivability of this structure in this storm scenario.
- Further research is required and justified prior to engineering design of bridge decks for accommodating wave-induced loads.
- In this location, approximately one quarter of the economic costs associated with a bridge closure can be attributed to the loss of non-business (leisure) trips. The remaining three quarters of the losses come from direct loss of economic activity, including increased commuter travel time and distance, and increased freight travel time.
- A major US bridge, the I-10 Bridge near Pensacola, Florida, which was destroyed by Hurricane Ivan in 2004:
    - May have been destroyed by the increase in wave-induced loads due to the sea level rise which occurred during the life of the structure.
    - May have survived that storm with any of the adaptations evaluated in this study.
- The findings of this study address a key gap in our knowledge related to engineering solutions for preparing for climate change - including incorporating sea level rise and changing storm surge into design practices.

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