Impacts of Climate Change and Variability on Transportation Systems and Infrastructure:
The Gulf Coast Study, Phase 2

Task 3.2: Engineering Assessments of Climate Change Impacts and Adaptation Measures

U.S. Department of Transportation
Gulf Coast Study, Phase 2

August 2014
FHWA-HEP-15-004

Impacts of Climate Change and Variability on Transportation Systems and Infrastructure

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Engineering Analysis and Assessment

Final Report, Task 3.2

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Date: **August 2014**

Contract No.: **GS-10F-0124J**

Report No.: **FHWA-HEP-15-004**

Acknowledgements

The authors gratefully acknowledge the assistance from the following:

- The Mobile Metropolitan Planning Organization's Climate Change Work Group
- The U.S. Department of Transportation's Gulf Coast Phase 2 Technical Advisory Committee
- *Local Experts on Highways:* Kevin Harrison and Tom Piper of the South Alabama Regional Planning Commission; Lee Reach, Don Powell, David Ramsey of the Alabama Department of Transportation; Ricky Mitchell and Tina Sanchez, Mobile County; Nick Amberger of the City of Mobile
- *Local Experts on Ports*: Jerald Kichler

We also gratefully acknowledge the time and effort provided by the peer reviewers of this report (in alphabetical order): Eric Brown, Federal Highway Administration (FHWA); Gary Carr, Federal Railroad Administration; Andrea Hendrickson, Minnesota Department of Transportation; Casey Kramer, Washington State Department of Transportation; Jeffrey Knaub, Maryland State Highway Administration (MDSHA); Khalid Mohammed, FHWA; Cynthia Nurmi, FHWA; Karuna Pujara, MDSHA; and Laurel Radow, FHWA.

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1. Executive Summary

1.1 Background and Purpose

Facility managers, transportation leaders, and elected officials are increasingly concerned about the resilience of transportation infrastructure to a range of threats, including the threats posed by climate change and extreme weather. However, the information and tools necessary to understand, evaluate, and rank vulnerabilities remain scarce. This is particularly true as it relates to specific facilities and assets; even more scarce is information and data regarding adaptation (i.e., risk mitigation) measures, their efficacy in reducing risks, and the returns on investment that might be expected if adaptation strategies are adopted. While some resources exist for evaluating transportation system vulnerability at a broad level, there is little guidance on how to do so at the facility-level. Likewise, general information on adaptation options is known, but there have been few asset-level analyses evaluating the potential effectiveness of those options.

Acknowledging the importance of establishing systematic, transferrable approaches for assessing and addressing vulnerability to climate- and weather-related risks, the U.S. Department of Transportation's (USDOT) Center for Climate Change and Environmental Forecasting commissioned a comprehensive, multiphase study of climate change impacts on transportation in the Central Gulf Coast region. This study, formally known as *Impacts of Climate Change and Variability on Transportation Systems and Infrastructure: Gulf Coast Study* (hereafter, "the Gulf Coast Study").[1] Phase 1 (completed in 2008) examined the impacts of climate change on transportation infrastructure at a regional scale. Phase 2 (nearing completion) provides a more detailed assessment of the vulnerability of the most critical components of the transportation system in Mobile, Alabama to weather events and long-term changes in climate.

This report discusses a series of engineering assessments on specific transportation facilities in Mobile that evaluated whether those facilities might be vulnerable to projected changes in climate, and what specific adaptation measures could be effective in mitigating those vulnerabilities. The purpose of the engineering assessments was twofold:

(1) Develop and test a detailed climate impact assessment process (*The Process*) that both evaluates the climate vulnerabilities of specific transportation assets, and evaluates possible adaptation strategies that could be implemented. The methodologies developed for these assessments could be applied to similar facilities elsewhere. This report represents one of the few resources available to transportation practitioners that include engineering methodologies for evaluating climate change vulnerabilities and adaptation measures at the facility level.

[1] For background information on the Gulf Coast Study, please see Section 2.

(2) Explain and document Mobile-specific findings for each facility-climate stressor pair, including any findings that may apply more generally to engineering design practices, operations and maintenance practices, or other lessons learned.

1.2 Overview of the General Process for Transportation Facility Adaptation Assessments

Climate and weather have always played an important role in the planning and design of transportation infrastructure. Facilities of all types are planned to avoid high hazard areas whenever possible, bridges must be designed to accommodate floods, pavement must be able to tolerate extremes in temperature, etc. However, the prospect of long-term changes in climate and more extreme weather presents a fundamental challenge to transportation professionals. Consideration of long-term climate change threats will increasingly be required when planning, designing, and in some cases, operating/maintaining new infrastructure. Due to the lack of standard approaches or models for attempting this, this project developed a *General Process for Transportation Facility Adaptation Assessments* (the *Process*).

The *Process* provides an 11-step framework for determining the vulnerabilities of an individual transportation facility to climate change, developing adaptation options to mitigate risks of anticipated changes, and selecting a course of action. The 11 steps are:

1. **Describe the Site Context** – Describe location-specific details, such as surrounding land uses, population, economic activities; performance characteristics; proximal historic or sensitive environmental resources; long-term transportation and land use plans, and whether they account for climate change impacts; function(s) the facility serves or will serve within the broader transportation network.
2. **Describe the Existing / Proposed Facility** – Describe facility-specific details, such as location, functional purpose, design type, dimensions, elevations, design life, age, condition, and design criteria.
3. **Identify Climate Stressors that May Impact Infrastructure Components** – Identify climate-related variables that are typically considered in planning and design of the type of facility being investigated (e.g., precipitation, temperature, sea level, storms).
4. **Decide on Climate Scenarios and Determine the Magnitude of Changes** – Describe climate model projections that are used to determine whether and how much each of the variables of concern may change in the future.
5. **Assess Performance of the Existing / Proposed Facility** – Assess whether the existing/proposed facility is performing as expected/modeled under current climate data and design assumptions and whether it will continue to do so under each of the possible future climate scenarios selected in Step 4.
6. **Identify Adaptation Option(s)** – Identify potential planning, design, and maintenance / operations options that could be used to address climate risks to the facility.

7. **Assess Performance of the Adaptation Option(s)** – Assess the performance of each adaptation option under each potential climate change scenario selected in Step 4. This analysis is similar to Step 5 except that it is performed on the adaptation options instead of the existing facility or, in the case of new facilities, the standard design.
8. **Conduct an Economic Analysis** – Evaluate how the benefits of undertaking a given adaptation option, defined as the costs avoided with adaptation, compare to its incremental costs under each of the possible future scenarios developed in Step 4.
9. **Evaluate Additional Decision-Making Considerations** – Identify and evaluate other (non-engineering, non-economic) factors that should be considered before a final decision is reached.
10. **Select a Course of Action** – Consider both economic and non-economic factors, weighing all the information presented, and select a course of action.
11. **Plan and Conduct Ongoing Activities –** Identify, plan for, and conduct ongoing activities (such as monitoring), using tools such as facility management plans.

The Process was developed to be general enough to be applied to multiple transportation modes and asset types. It can also be used both for existing facilities, where adaptive retrofits might be considered, and for proposed new facilities where adaptation measures can be incorporated into the design.

This *Process* was employed throughout each of the case studies discussed in this report. For each case study, the *Process* was used to evaluate vulnerability of a specific asset to a certain projected climate stressor (such as increased temperatures, sea level rise, storm surge), and also to evaluate potential adaptation measures.

The climate data used in the case studies was developed during previous tasks of this project. See the Task 2 and Task 3.1 reports at www.fhwa.dot.gov/environment/climate_change/adaptation/ongoing_and_current_research/gulf_coast_study/phase2_task2/sensitivity_report/ and www.fhwa.dot.gov/environment/climate_change/adaptation/ongoing_and_current_research/gulf_coast_study/phase2_task3 for more information on how the climate projection information was developed.

1.3 Overview of Case Studies Selected to Demonstrate the *Process*

Ten case studies were conducted to demonstrate the application of the *Process* to a range of design problems at 10 specific facilities in Mobile, Alabama. In addition, operations and maintenance (O&M) opportunities for adaptation were examined in a final case study. These case studies illustrate how engineering design processes may be augmented to incorporate climate change and extreme weather considerations. Table 1 lists the climate stressors and asset types chosen for study and the specific facilities that were investigated. Figure 1 shows the location of these facilities. Since the case studies are intended to demonstrate the application of the *Process*, each case study is structured using the 11 steps of that *Process*. Due to the nature of this project as a broad study across many facilities (rather than an in-depth study of a single project), none of the case studies represent a full application of every step in the *Process*. [2]

This report showcases engineering assessments across a range of transportation asset types and climate change stressors. The facilities chosen for the engineering assessment were not necessarily the most vulnerable assets; instead, they represent a broad range of facility types and climate stressors in hopes that methods developed here might be instructive, not only for Mobile but for transportation agencies nationwide.[3]

[2] There is variation in the degree to which each step was completed across case studies, due to resource and data constraints, and applicability of each step. For example, for assets not likely vulnerable to the climate stressor analyzed, Step 7 (Assess Performance of Adaptation Options) was not completed in detail. Additionally, there were not sufficient resources to complete the Step 8 (Economic Analysis) for all case studies, so the first case study is the only one that includes a detailed economic analysis; this serves as a possible methodology for analyzing economics of others as well.

[3] Note that neither the *Process* nor the facility-specific findings are intended to change specific design methodologies for assets or to serve as an alternative approach for designing projects. Findings are illustrative and represent a first attempt to systematize an approach for incorporation of climate and weather risks into engineering analyses using facilities in Mobile as case studies.

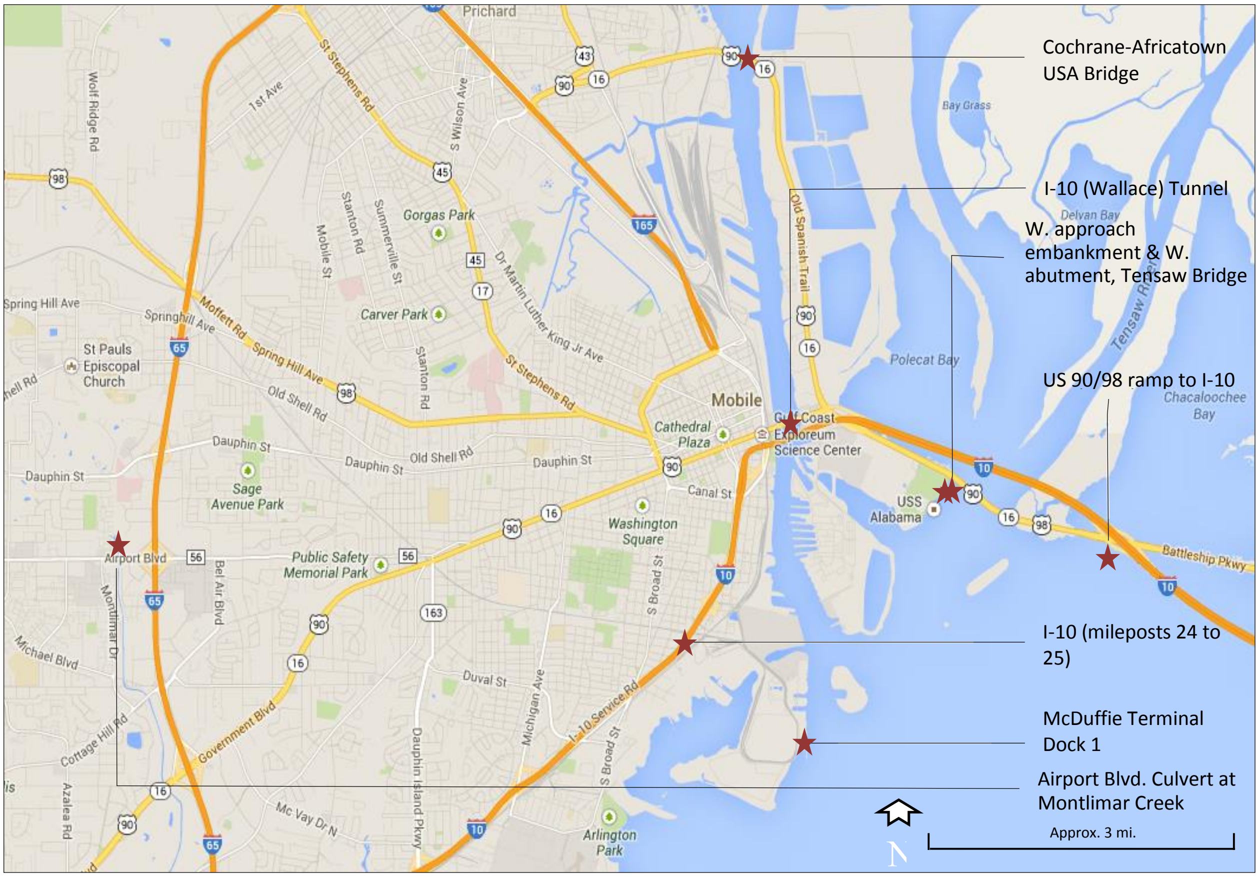
In presenting these illustrative examples, the authors are not suggesting that transportation agencies would apply such detailed analyses to all assets under their purview. Rather, transportation agencies may wish to conduct detailed analyses for specific assets considered to be particularly critical, vulnerable, expensive to replace, due for an upgrade/replacement, etc.

Table 1: Listing of Case Studies by Climate Stressor, Facility Type, and Damage Mechanism

Report Section	Climate Stressor	Asset Type	Damage Mechanism	Asset Location	**Indicator-Based Vulnerability***
4.4.1	Increased precipitation	Culvert	Flooding	Airport Boulevard culvert at Montlimar Creek	Low
4.4.2	Sea level rise	Navigable waterway bridge	Clearance for navigation	Cochrane-Africatown USA Bridge	Medium
4.4.3	Sea level rise	Bridge approach embankment	Slope erosion	West approach embankment of the US 90/98 Tensaw-Spanish River Bridge	High
4.4.4	Higher storm surge	Bridge abutment	Abutment scour	West abutment of the US 90/98 Tensaw-Spanish River Bridge	Medium
4.4.5	Higher storm surge	Bridge segment	Wave forces, bridge pier scour	US 90/98 ramp to I-10 eastbound at Exit 30	High
4.4.6	Higher storm surge	Road alignment	Overtopping / slope erosion	I-10 (mileposts 24 and 25)	Low
4.4.7	Higher storm surge	Coastal tunnel	Flooding	I-10 (Wallace) Tunnel	Medium
4.4.8	Higher storm surge	Shipping pier	Waves	McDuffie Coal Terminal, Dock 1	Low
4.4.9	Temperature changes	Roadway pavement	Rutting, concrete crackings	Generic location	Low
4.4.10	Temperature changes	Continuously welded rail	Buckling, pull-aparts	Generic location	Medium
4.4.11	Precipitation, wind, temperature, sea level rise, hurricanes	Operations and maintenance activities for various facilities	Varies based on climate stressor	Alabama Department of Transportation, City of Mobile, and Mobile County operations and maintenance practices	High

* An asset's estimated vulnerability was calculated previously in this project, as detailed in the report *Task 3.1: Screening for Vulnerability*. The vulnerability designations developed within the *Screening for Vulnerability* report provide initial estimates of an asset's vulnerability to a particular climate stressors based on a series of vulnerability "indicators" applied broadly across the transportation system. However, in this *Task 3.2 Engineering Analysis and Assessment* report, the more refined engineering analyses performed may result in different determinations as to an asset's actual vulnerability.

Figure 1: Case Study Locations[4]



1.4 Key Lessons Learned

The analyses conducted through the case studies yielded some important lessons learned regarding the applicability of the *Process* and developing input values for engineering designs.

1.4.1 Applicability of a General Process for Transportation Facility Adaptation Assessments

One of the important lessons learned from the engineering case studies is that the 11-step *Process* can be successfully applied across different types of assets and for a range of climate-change stressors. In fact, the *Process* was specifically developed to generate consistency among various engineering disciplines working on this project. The *Process* can therefore serve as an organizing framework for how engineering design can be undertaken considering the uncertainties associated with possible future environmental conditions.

Another lesson learned is that there is an important need for additional guidance on how engineering design can be undertaken given uncertainty about future conditions. As mentioned

[4] Source of base map: Google Maps (as modified)

above, there is very little engineering guidance for handling climate change-related uncertainty in the engineering design process. In developing many of the engineering case studies, considerable discussion and debate occurred among the designers representing different engineering sub-disciplines (including structural, hydraulic, geotechnical, and pavement) on the most appropriate approach for analyzing a particular asset given expected future loads and stresses. Engineering practice—and indeed engineering culture—is focused on research and statistical analyses of historical events (rainfall, extreme heat, etc.); these data provide the required input variables used in decision-making. The uncertainty associated with future input variables that are derived from climate model projections drives the need for new approaches to develop those input variables and consider their effects when making planning, design, and operations/maintenance decisions.

1.4.2 Developing General Input Values for Engineering Designs

A design process that reflects projected changes in climatic conditions has to account for possible changes in the input values of the design variables beyond simply relying on historical data. This is a significant shift from standard engineering design practice, and the case studies demonstrated example methodologies for doing so.

Engineering designs for transportation facilities rely on a determination of the stresses and loads that facility components will likely face. Identification and determination of the stresses and loads is, thus, critical to selecting designs that will provide durable and stable asset performance. The engineering profession has developed design procedures and methods that are based on years of experience and documentation of the relationships between load/stress input variables and the resulting design characteristics. Input variables used in these equations are taken from historical data. Future climatic conditions may result in changes to these variables that are not simply an extension of past trends. Thus, a design process that reflects projected changes in climatic conditions in the future would have to account for possible changes in the input values of the design variables beyond simply relying on historical data; this is not unlike designing structures to withstand seismic risks.

Appropriate Scale of Input Data

The engineering case studies illustrated the need to provide input data at a scale necessary for design purposes. This has been a challenge noted for many years and an identified gap in the application of climate scenario data in engineering design. For example, climate information is often presented in terms of ranges, or average changes over longer periods (seasons, years, decades), but engineering design often requires information on return periods (such as the 25-year event), short-term extremes (such as maximum temperature and precipitation experienced over a short timeframe), and for short timeframes such as daily or hourly events. This study developed data at the temporal and spatial scale needed to conduct engineering design at the project level. Such data were derived from the best climate modeling results available for the region, as well as from assumptions on the best approaches for providing that data that could be used in engineering design.

The Importance of Using a Range of Climate Input Data

It is important for a robust design process that a range of climate projection data be considered, simply to make sure that even the lower estimates do not require corrective design action, and that a reference alternative is presented for the scenario analyses of the higher stresses on the assets. Additionally, in some cases the lower scenario was actually found to be more damaging than the higher scenario.

In this and most similar studies, the analysis of future climate conditions was predicated on the emission scenarios offered by the Intergovernmental Panel on Climate Change (IPCC). The IPCC offers a range of emission scenarios that are then used as inputs to multiple global climate models. Importantly, the IPCC states that each of the emissions scenarios are equally likely to occur;[5] thus, while not providing a different conclusion on any one trajectory of future climate, the climate projections derived from those scenarios are useful in providing a range of outputs that can be considered as a sensitivity test in the planning and design of transportation facilities. A range of variables has been developed that can be used and parameterized in design decisions; some data may be found to have no bearing on design while other data may have wide-ranging consequences. The case studies demonstrate that the values of the design variable inputs can have strong influences on expected stresses and loads on transportation assets, as well as on appropriate adaptation response.

Earlier tasks of this study included development of climate information (see the Task 2 reports) from which climate narratives[6] were developed (see the Task 3.1 report).[7] "Warmer" and "Hotter" narratives were developed to describe ranges of temperature values, and "Wetter" and "Drier" narratives of precipitation projections were developed for the Mobile study region for use in the engineering case studies. These narratives provided the range of climate projection data used in this study for temperature and precipitation. Task 2 and Task 3.1 also developed climate narratives for sea level and storm surge, which provided the range for those climate stressors used in the case study analyses.

The scenario approach produces a range of values for the input design variables. Depending on the environmental stressors being considered, many of the engineering case studies showed that the scenarios defined by the lower ranges of design input values had either little or no impact on the current design of the asset, or that the impacts could lead to some corrective design action. For those with no impact, the original design of the asset provided enough strength and durability

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[5] IPCC, 2007
[6] Note that the term "narratives" was used in place of the term "scenario" to avoid confusion with the IPCC Special Report on Emission Scenarios; the narratives function like scenarios, as described in the Task 2 report.
[7] USDOT, 2012; USDOT, 2014

to withstand the forces that were likely to be placed on the asset assuming climate change-induced design values. In other cases, such as with storm surge, the assets were found to be vulnerable for all scenarios. In fact, with storm surge, in some cases the lower scenario was actually found to be *more* impactful than the higher scenario. This lends further credence to the importance of including lower end scenarios in an adaptation assessment.

Addressing the Design Storms vs. Modeled Future Storms

Rooting future scenarios in the experience of a single historical weather event and then altering characteristics to reflect possible future permutations, has the benefit of providing very relatable results to local stakeholders, especially if a severe storm event occurred recently. However, this approach does not allow for the calculation of a return period, which presents a challenge when comparing future asset performance against a design standard rooted in return periods (e.g., no overtopping is allowed up to the 100-year storm).

Engineering practice is based on the premise of "acceptable risk"; acceptable risk is addressed in design standards that hinge on the recurrence probability of storm events (e.g., a bridge designed to pass the 100-year storm with a 1% annual chance of occurrence). Selection of design storms or return period storms are indicative of an agency's risk tolerance and reflect a trade-off between the upfront cost of providing additional protection and the expected remedial cost in case of such an event occurring. Understanding performance relative to a return period storm is important to established engineering practice.

Scenarios marrying historical and future information were particularly important to local stakeholders in the context of developing sea level rise and storm surge projections. In the Gulf Coast Project, different sea level rise scenarios were combined with base and modified Hurricane Katrina storm surge scenarios. Modeling efforts to develop a series of storm surge scenarios enabled the consideration of increased wind intensity and modified storm tracks layered on top of historical data. This approach is novel in engineering design because "design storms" are generally defined by historical record. While these future scenarios were not analyzed for the purpose of establishing future return periods, they served as the basis for scenario-based engineering analyses. There is a need for additional research and dialogue to identify and refine approaches to considering future storm surge scenarios and the effects of climate change on the way engineering design inputs are established.

1.5 Facility-Specific Findings

The case studies yielded important findings about the vulnerabilities and adaptation options for the specific assets analyzed. The following table provides an overview of findings for each case study as well as a section reference so readers can readily find the full case study write-up. Short summaries of each case study are presented in the Lessons Learned section (Section 5).

Table 2: Case Study Locations

Case Study Name	Facility	Was it Vulnerable?	What Options are Viable?	Take-Away
Culvert Exposure to Changing Precipitation Patterns	Airport Boulevard Culvert over Montlimar Creek	The culvert is adequate for current conditions Runoff overtops roadway under projected conditions	Increasing the number of culvert cells Increasing the number and size of culvert cells Regional Drainage Area Management	Using 24-hour duration precipitation projections that are available from climate models is an appropriate approach for determining rainfall when designing large culverts for future conditions Benefit-cost analysis using the Monte Carlo process is a useful way to deal with climate uncertainties influencing major projects
Bridge Over Navigable Waterway Exposure to Sea Level Rise	Cochrane-Africatown USA Bridge	The navigational clearance is vulnerable to only the highest sea level rise projection	Restrict ship heights Re-configure seaward ports to handle more vessels Replace with higher bridge at the end of the life-span Monitor sea level and act accordingly	Port and transportation planners should monitor and consider sea level rise and its impacts on navigation constraints beginning as soon as feasible
Bridge Approach Embankment Exposure to Sea Level Rise	US 90/98 Tensaw-Spanish River Bridge – western approach	The embankment is temporarily vulnerable to wave run-up from the projected sea level rise Vulnerable to permanent inundation under highest sea level rise projection	Continue maintenance to ensure proper function of existing riprap slope protection Raise the elevation of the riprap slope protection, approach road, and bridge Extend the embankment slopes (20:1)	Any protection recommended for a facility of this type would need to address all potential stressors upon the abutment including storm surge and scour The general analytical methods demonstrated here can be applied to other coastal embankments

Case Study Name	Facility	Was it Vulnerable?	What Options are Viable?	Take-Away
Bridge Abutment Exposure to Storm Surge	US 90/98 Tensaw-Spanish River Bridge – western abutment	Not vulnerable	No action required for erosion from storm surge at this location; however, for other abutments: • Control the approach and departure flow to realign water passage though the waterway • Armor the bridge opening • Widen, lengthen and / or shift bridge • Control drainage from the embankment and roadway to avoid erosion in the abutment area	Formulas for estimating abutment scour are very conservative, especially for typical coastal conditions leading agencies to protect foundations rather than design foundations to resist the predicted scour Inspectors should be informed that even if the structural portion of an abutment is situated on "dry" ground, other components such as bulkhead, riprap, or other stability measures may play a key role in the overall scour resistance of the abutment and should likewise be monitored
Bridge Segment Exposure to Storm Surge	US 90/98 ramp to I-10 eastbound at exit 30	Superstructure at Bents 11 and 13 would likely have bolt failures at the bottom of the girders and lift off under all storm surge scenarios investigated. It is likely that this failure could occur at any of the spans within the study section of the ramp. The piles have sufficient axial capacity to resist the uplift force on the superstructure, but the piles are not able to resist the lateral forces (shear and moment) under any of the scenarios and may fail due to shear and / or bending	Design to break away (AASHTO) Strengthen the bolt connections Install open grid decks Design shallower girder sections Use open rail parapets Replace the lower bridge segments with protected embankment (shorten the bridge)	The worst case storm surge scenario does not necessarily translate to the worst effect on the facility

Case Study Name	Facility	Was it Vulnerable?	What Options are Viable?	Take-Away
Road Alignment Exposure to Storm Surge	I-10 (mileposts 20 to 25)	I-10 can overtop under the two highest projections Underpasses subject to erosion under all projections	Harden one or more of the underpasses Armor I-10 roadway embankment Raise the roadway	Roadway embankment breaching is an area with little research data on prediction methods Additional erosion protection should be considered when designing roadway crossings that could be subjected to reverse flow from storm surges
Coastal Tunnel Exposure to Storm Surge	I-10 (Wallace) Tunnel	The tunnel is vulnerable to flooding during storms of return periods of 75 years or greater	Raise the west portal wall to elevation 19 feet Raise all approach walls to elevation 19 feet Install “temporary” flood gates	When evaluating the impacts of storm surge, wave height must be included in the analysis The Saffir-Simpson Hurricane “Category” Scale was not particularly valuable for engineering decisions because it is wind-speed based, not surge based
Shipping Pier Exposure to Storm Surge	McDuffie Coal Terminal, Dock 1	No, not the pier itself	Adaptations should be considered to preserve critical equipment and ancillary services	Loads used to design piers are beyond expected lateral loads expected from even the most extreme storm surges

Case Study Name	Facility	Was it Vulnerable?	What Options are Viable?	Take-Away
Pavement Mix Design Exposure to Temperature Changes	Area-wide	Pavements constructed to ALDOT performance grades approach being vulnerable only to the highest temperature projections	Options to reduce rutting in hotter areas: • Use thicker pavement sections at the time of initial design • Consider PCC pavement in certain applications • Change frequency of maintenance Options to improve PCC paving in hot temps: • Minimize time to transport, place, consolidate, and finish the PCC • Use a PCC consistency that allows rapid placement and effective consolidation at high temperatures. • Protect the PCC from moisture loss at all times during placement and during its curing period ▪ Use cooled PCC, achieved by using chilled mixing water, or cooling the coarse aggregate	Monitor temperature changes, periodically update historical temperature records, and use climate projections where appropriate rather than existing historical data currently used by pavement design software
Continuous Welded Rail Exposure to Temperature Changes	Area-wide	Rail can be vulnerable under the "Hotter" narrative temperature projections	Plan and conduct on-going activities Perform regular maintenance and inspections Monitor temperature trends towards the "Hotter" narrative values Keep incident records to correlate with temperature trends Increase rail neutral temperature Ensure that ballasted tracks have sufficiently wide shoulders to support the ties	The neutral temperature currently used by the railroad would be inadvisable under the "Hotter" narrative Scenario at all future time periods. Continuing to use the adopted neutral temperature might increase the risk of sun kinks in the future.

Case Study Name	Facility	Was it Vulnerable?	What Options are Viable?	Take-Away
Operations and Maintenance Activity Exposure to Climate Change and Extreme Weather Events	Area-wide	O&M activities will be more stressed by the increases in temperature, precipitation, and extreme weather events	Consult with designers about more durable materials with consideration for likely future conditions (e.g., higher temperatures, increased rainfall intensities) Change equipment needs due to expected increases in emergency response Create stand-by contracts to increase response capacity and shorten reaction times Increase attention to erosion and sedimentation issues Improve weather information systems may be applied for year-round use to monitor precipitation and flooding. Increase cross-training of staff Stockpile materials	O&M personnel in the Gulf Coast region and elsewhere need to be prepared for the unique and continuing challenges of extreme weather particularly when it comes to cooperation between organizations

2. Background

2.1 Overview of Gulf Coast Project

Facility managers, transportation planners, and elected officials are increasingly concerned about the resilience of transportation infrastructure to a range of threats, including the threats posed by climate change and extreme weather. However, the information and tools necessary to understand, evaluate, and rank vulnerabilities remains scarce. This is particularly true as it relates to specific facilities and the assets that constitute a facility; even more scarce is information and data regarding adaptation (i.e., risk mitigation) measures, their efficacy in reducing risks, and the returns on investment that might be expected if adaptation strategies are adopted. Because many assets (e.g., rail lines, bridges, and piers) are expected to provide service for 100 years or longer, consideration of medium, and long-term climate threats is essential to ensuring safe and effective transportation services at all levels of authority (i.e., federal, state, county, and municipal) and for both publicly and privately managed assets.

Acknowledging the importance of establishing systematic, transferrable approaches for assessing and addressing vulnerability to climate and weather risks, the USDOT's Center for Climate Change and Environmental Forecasting commissioned a comprehensive, multiphase study of climate change impacts on transportation in the Central Gulf Coast region. This study, formally known as *Impacts of Climate Change and Variability on Transportation Systems and Infrastructure: Gulf Coast Study* (hereafter, the "Gulf Coast Study"), is the first such study of its magnitude in the United States and represents an important benchmark in the understanding of what constitutes an effective transportation system adaptation effort.

The Gulf Coast region was selected as the focal point due to its dense population and complex network of transportation infrastructure, as well as its critical economic role in the import and export of oil, gas, and other goods. The study is funded by the USDOT Center for Climate Change and Environmental Forecasting and managed by the Federal Highway Administration (FHWA). The Gulf Coast Study includes two phases:

- **Phase 1** (2008) – During Phase 1, USDOT partnered with the U.S. Geological Survey (USGS) and the U.S. Climate Change Science Program (CCSP) to investigate potential climate change risks and impacts on coastal ports, road, air, rail, and public transit systems in the region from Mobile, Alabama to Houston/Galveston, Texas. The study assessed likely changes in temperature and precipitation patterns, sea level rise, and increasing severity and frequency of tropical storms. Phase 1 then explored how these changes could impact transportation systems. It found that a local sea level rise of four feet would permanently inundate 27% of the Gulf Coast region's roads, 9% of its railways, and 72% of its ports; higher temperatures would likely lead to more rapid deterioration of infrastructure and higher maintenance costs; more intense precipitation events could overwhelm drainage systems and cause damage and delays; and increased hurricane intensity coupled with sea level rise would pose a significant threat to infrastructure.

- **Phase 2** (nearing completion) – The purpose of Phase 2 is to provide a more detailed assessment of the vulnerability of the most critical components of the transportation system to weather events and long-term changes in climate. This work is being conducted on a single metropolitan area, the Mobile, Alabama region (see Figure 2), with the intention of making the processes used in the study replicable to other areas. USDOT is conducting Phase 2 in partnership with the Mobile Metropolitan Planning Organization, part of the South Alabama Regional Planning Commission (SARPC).

Phase 2 Study Area

While Phase 1 took a broad look at the entire Central Gulf Coast region (between Houston/Galveston, Texas and Mobile, Alabama) with a 'big picture' view of the climate-related challenges facing infrastructure, the current effort in Phase 2 focuses on Mobile, Alabama. The area of the study includes Mobile County (including Dauphin Island) and the crossings of Mobile Bay to the east to landfall in Baldwin County (Figure 2).

Phase 2 is divided into the tasks below. The first three tasks form the basis of a vulnerability screen and assessment of the Mobile transportation system, while the other tasks focus on tool development, coordination with stakeholders, and communication of project results.

- **Task 1: Identify critical transportation assets in Mobile.** This task (completed) served as a first level screen for the vulnerability assessment, by identifying which transportation assets are highly critical to Mobile. The results were published in the report *Assessing Transportation for Criticality in Mobile, Alabama*.[8]
- **Task 2: Develop climate information.** Task 2 (completed) focuses on characterizing how temperature, precipitation, streamflow, sea level, and storms and storm surge in Mobile could change due to climate change. This task also investigated the sensitivities of different transportation assets to each of these climate stressors, which is discussed in the companion report *Assessing the Sensitivity of Transportation Assets to Climate Change in Mobile, Alabama*.[9]
- **Task 3: Determine vulnerability of critical assets.** This task (partly covered in this report) evaluates how the highly critical assets identified in Task 1 could be vulnerable to the climate information developed under Task 2. Activities under this task led to a clearer, more systematic understanding of the key vulnerabilities of Mobile's transportation system to climate and weather factors. The methodology and findings of a high level vulnerability assessment of the transport system are covered in *Screening for Vulnerability*.[10] This report provides engineering-oriented analyses of selected vulnerable assets.
- **Task 4: Develop risk management tool(s).** Based on the findings and lessons learned during the first three tasks, Task 4 is culminating in a suite of tools and resources to assist other transportation agencies in conducting similar assessments and in managing their identified risks.[11]

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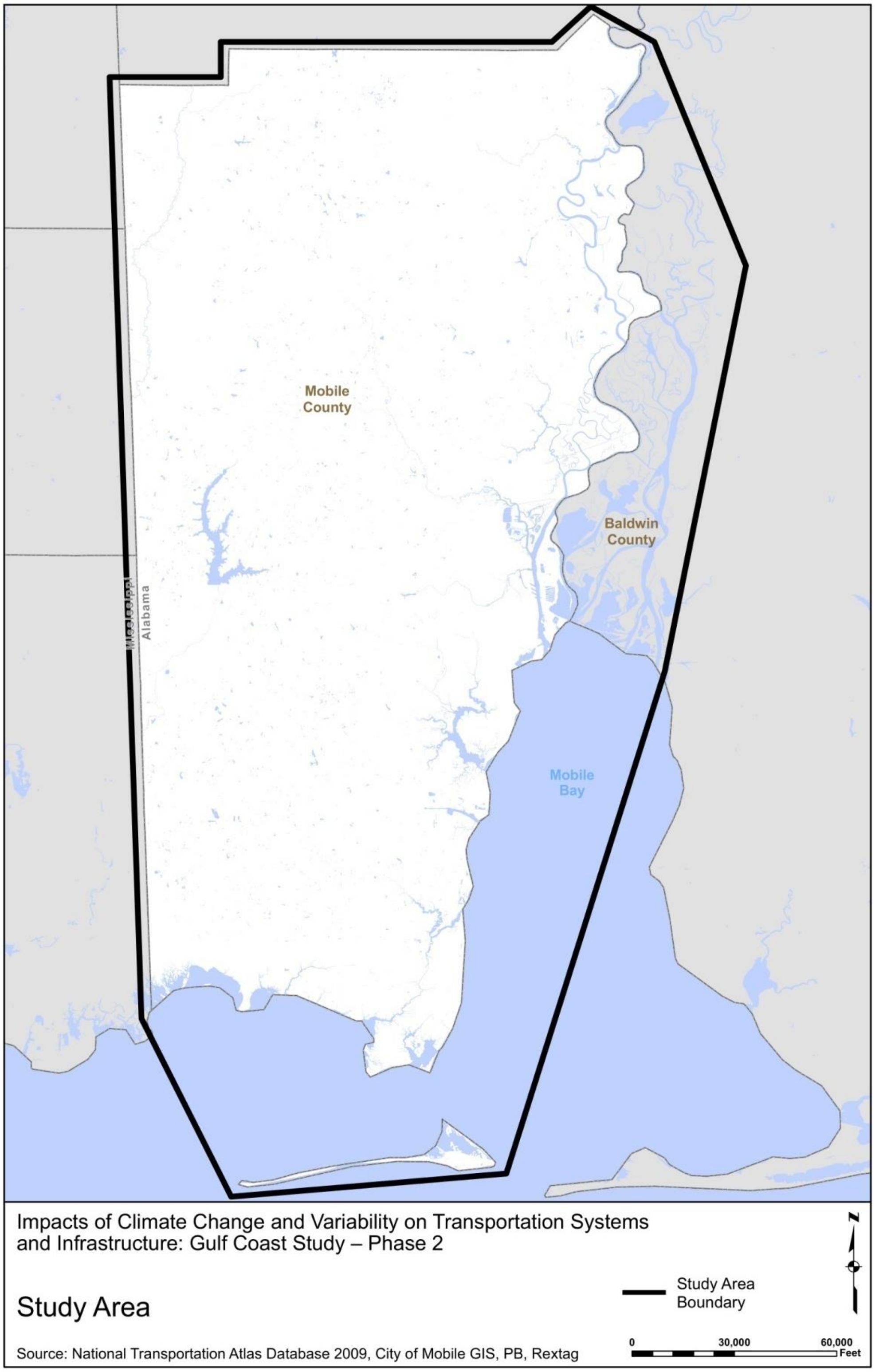
[8] The Task 1 report is available at http://www.fhwa.dot.gov/environment/climate_change/adaptation/ongoing_and_current_research/gulf_coast_study/phase2_task1/index.cfm.
[9] The Task 2 report is available at http://www.fhwa.dot.gov/environment/climate_change/adaptation/ongoing_and_current_research/gulf_coast_study/phase2_task2/sensitivity_report/.
[10] The *Screening for Vulnerability* report is available at www.fhwa.dot.gov/environment/climate_change/adaptation/ongoing_and_current_research/gulf_coast_study/phase2_task3.
[11] These tools will be housed at http://www.fhwa.dot.gov/environment/climate_change/adaptation/adaptation_framework/.

- **Task 5: Coordinate with planning authorities and the public.** Ongoing throughout the project, this task focuses on engaging key local transportation stakeholders, as well as members of the public.
- **Task 6: Disseminate and publish results.** High-level findings derived from Tasks 1 through 5 will be summarized in a brief synthesis report, as well as associated presentations of the findings.

Figure 2: Gulf Coast Study Phase 2 Project Study Area



2.2 Purpose of the Engineering Assessments

This report describes the methodology and results of engineering assessments conducted on selected transportation assets in Mobile. After inventorying transportation assets in Mobile for all five modes (i.e., highway, port, airport, rail, and pipeline), conducting an assessment to prioritize critical assets (Task 1), developing Mobile-specific climate data (Task 2), and using a screening approach to rank vulnerability of critical assets to specific climate stressors (Task 3.1), the project team embarked on a series of engineering assessments.

The facilities chosen for the engineering assessment were not reflective of the most vulnerable assets, but rather they were chosen to represent a broad range of facility types and climate stressors in hopes that methods developed here might be instructive, not only for Mobile but for transportation agencies nationwide.

The purpose of the engineering assessment was twofold:

(1) Develop and test a detailed climate impact assessment process that both quantifies climate risk to a particular facility (or asset within a facility) and evaluate possible adaptation strategies that could be implemented.

(2) Explain and document Mobile-specific findings for each facility-climate stressor pair, including any findings that may apply more generally to engineering design practices, operations and maintenance practices, or other lessons learned.

A *General Process for Transportation Facility Adaptation Assessments* (the *Process*) is established and used throughout the case studies to consider a variety of climate threats for a range of asset types and modes in Mobile County. Climate threats considered in these analyses include: temperature, sea level rise, storm surge, and precipitation. Asset types and modes covered in these analyses include: rail, pavement, a shipping port pier, a culvert, bridges, a tunnel, a roadway embankment, as well as operations and maintenance.

Note that neither the *Process* nor the facility-specific findings are intended to change specific design methodologies for assets or to serve as an alternative approach for designing projects. Findings are illustrative and represent a first attempt to systematize an approach for incorporation of climate and weather risks into engineering analyses using facilities in Mobile as case studies. Also note that, although these assessments are meant to demonstrate methodologies that other transportation practitioners could use, it is not expected that any transportation agency would apply such detailed analyses to all assets under their purview. Rather, transportation agencies may wish to conduct detailed analyses on specific assets considered to be particularly critical, vulnerable, expensive to replace, due for an upgrade/replacement, etc.

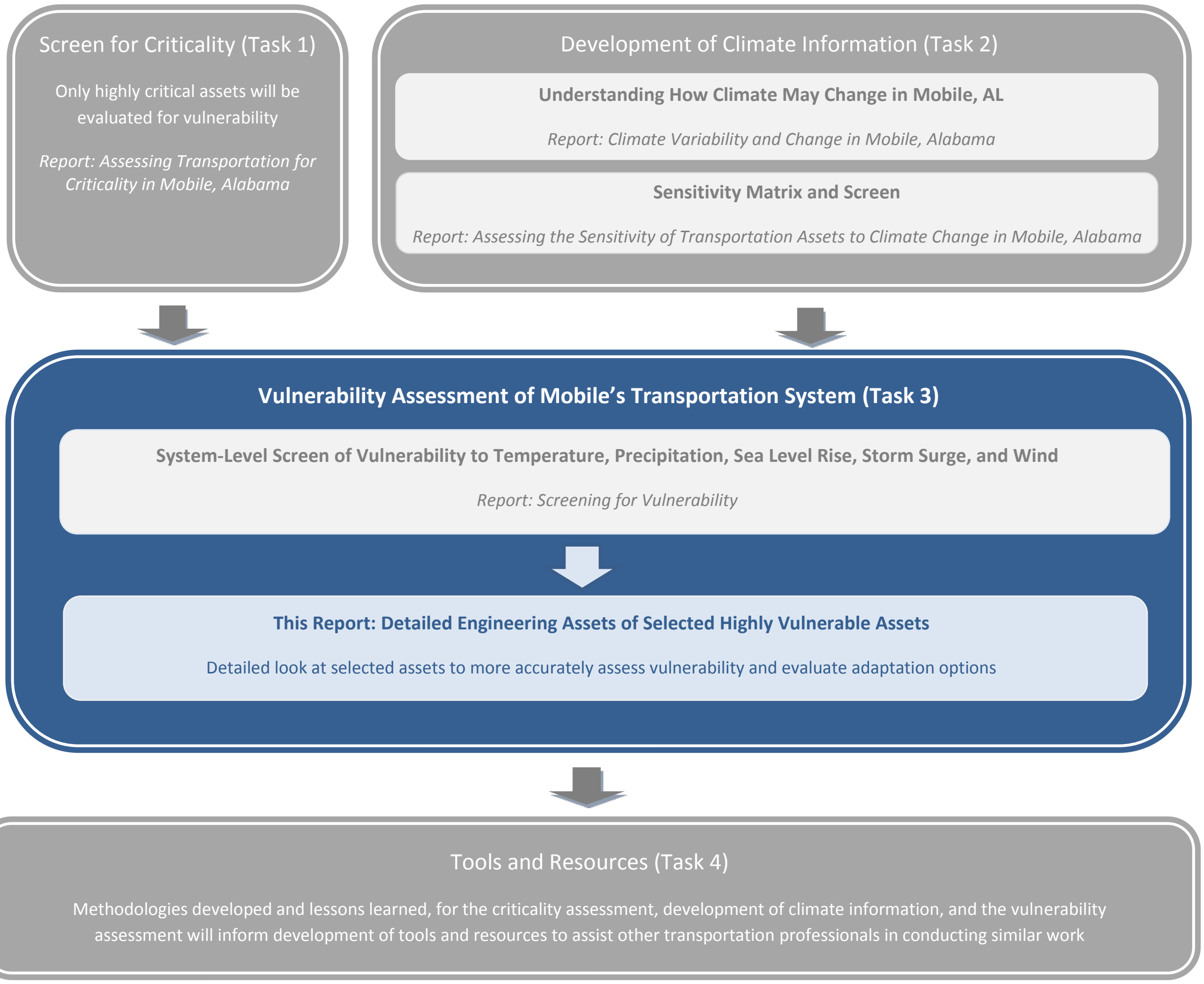
Figure 3 shows how the engineering assessments fit in with the other tasks under Phase 2.

2.3 Report Roadmap

The main body of this report is structured as follows:

- Overview of the *General Process for Transportation Facility Adaptation Assessments* used for the engineering assessments
- Case studies of engineering assessments for a variety of climate impacts on various asset types and modes in Mobile
- Overarching lessons learned
  - Key findings on the approach and defining key variables, which may be of particular interest to transportation officials considering engineering assessments in their region, and
  - Key findings of the engineering assessments on the assets in Mobile's transportation system
- Future research needs

Figure 3: Roadmap for Phase 2 of the Gulf Coast Project[12]



[12] Note: The components covered by this report are indicated with blue shading. The gray shading indicates other outcomes of the Phase 2 study that are presented in other reports and online resources.

3. A General Process for Transportation Facility Adaptation Assessments

3.1 Overview

Climate and weather[13] have always played an important role in the planning and design of transportation infrastructure; facilities of all types are planned to avoid high hazard areas whenever possible, bridges must be designed to accommodate floods, pavement must be able to tolerate extremes in temperature, etc. Traditionally, the transportation profession has relied upon historical records and statistical analyses of past climate data to inform decisions on the location and design of transportation facilities. In using historical climate data to design for future conditions, there is an inherent assumption that climate is stationary (i.e. unchanging over time).[14] Changes in climate and the frequency of extreme weather events have the potential to alter these fundamental assumptions. Climate model projections show that there could be significant changes to key environmental variables within the planned life-spans of many transportation facilities; this is particularly true for long-lived assets. These climate changes may require adaptations to the way transportation facilities are planned, designed, constructed, operated, and maintained in order to maximize functionality, safety, and the anticipated return on infrastructure investments.

Thus, the prospect of long-term changes in climate and more extreme weather presents a fundamental challenge to transportation professionals. With potential changes in climatic conditions, transportation professionals will have to grapple with new questions. For example, how much might environmental conditions change during and by the end of an asset's lifespan? At what rate will climate change take place? How can defensible, cost-effective decisions be made given the often large uncertainties involved in projections of future climate? These questions will need to be addressed on a case-by-case basis for each facility and climate change adaptation considerations may need to become standard practice when planning and designing new infrastructure.

Due to a lack of standard approaches or models for attempting this, USDOT developed a *General Process for Transportation Facility Adaptation Assessments* (the *Process*). The *Process* was developed for application on facilities found to be potentially vulnerable to climate changes

[13] Weather and climate are related in the sense that they both capture information on atmospheric conditions, however, they differ based upon the time periods referred to. Weather refers to short term atmospheric conditions as measured over time periods ranging from seconds to days and weeks. Climate, on the other hand, refers to long term patterns in atmospheric conditions; essentially, the average weather patterns for a given time period as statistically compiled from 30 years or more of weather observations.

[14] Stationarity refers to the lack of change in probabilities in a data series over time. Engineers do not assume stationarity with respect to all factors involved in design. For example, non-stationarity in watershed runoff values (e.g., due to anticipated land use changes in a drainage area) are typically considered in the design of bridges and culverts. However, with respect to the climate variables themselves (e.g., in the case of a bridge or culvert, the return period of various precipitation amounts), designers typically assume these values will remain stationary and there will be no change in the frequency of certain weather events in the future. Olson, Kiang, and Waskom (2010) provides an in-depth discussion of stationarity in the context of hydrology and water management which has broad applicability to the planning and design of transportation facilities as well.

in the *Screening for Vulnerability* report[15] produced as an earlier component of this project. That said, although the *Process* was developed in the context of this study, it is sufficiently generalizable such that it can be applied in other locations as well. The *Process* provides an 11-step framework for determining the vulnerabilities of an individual transportation facility to climate change, developing adaptation options to mitigate risks of anticipated changes, and selecting a course of action. The steps are generally as follows:

1. Describe the Site Context
2. Describe the Existing / Proposed Facility
3. Identify Climate Stressors that May Impact Infrastructure Components
4. Decide on Climate Scenarios and Determine the Magnitude of Changes
5. Assess Performance of the Existing / Proposed Facility
6. Identify Adaptation Option(s)
7. Assess Performance of the Adaptation Option(s)
8. Conduct an Economic Analysis
9. Evaluate Additional Decision-Making Considerations
10. Select a Course of Action
11. Plan and Conduct Ongoing Activities

Each of these steps is described in detail below.

The *Process* was developed to be general enough to be applied to multiple transportation modes and asset types. It can also be used both for existing facilities, where adaptive retrofits might be considered, or proposed new facilities where adaptation can be incorporated into the design. For new facilities or major upgrades to existing facilities, the *Process* can be followed during the planning and preliminary engineering phase of work so that it can influence project design decisions. For new projects, the *Process* assumes that a preliminary plan / design based on traditional practice has been developed before starting the *Process*. This preliminary plan would serve as a basis for comparison with plans that include adaptation options.

The *Process* is not intended to change specific design methodologies. The general approach to designing a culvert, for example, remains unchanged with the *Process*. What the *Process* potentially does change, however, are, (1) the climate-related inputs used in the design methodology, (2) the number and type of design options one develops, and (3) how the final option is chosen to provide a cost-effective and resilient improvement to the transportation network.

The remainder of this section provides a detailed discussion of each step in the *Process*. Next, Section 4 demonstrates how the *Process* can be applied to a variety of facility types using case study examples of potentially vulnerable facilities in the Mobile region.

[15] USDOT, 2014

3.2 The Process

3.2.1 Step 1 – Describe the Site Context

The first step in the *General Process for Transportation Facility Adaptation Assessments* involves developing a thorough understanding of the site context. The site's context is key to determining the appropriateness of various adaptation options considered in subsequent steps. Important questions that should be answered in this step include:

- What are the characteristics of the surrounding land uses, population, economic activities and significant community resources?
- For existing assets, what are the performance characteristics such as volumes / ridership, fleet mix, and role in network continuity?
- What are the characteristics of the surrounding topography and hydrography?
- Are there recognized historic resources and / or sensitive environmental resources in the area?
- What is the long-term transportation and land use plan for the area, does it account for climate change impacts, and, if so, how?
- What function does (or will) the facility serve within the broader transportation network, both in the near term and in the future (e.g., is it an evacuation route or does it provide access to an important community resource such as a hospital?)?

3.2.2 Step 2 – Describe the Existing / Proposed Facility

This step involves developing detailed knowledge on the existing or proposed facility to be studied. This knowledge is critical to developing appropriate and effective adaptation options in subsequent steps. Key information that should be gathered includes:

- Location
- Functional purpose
- Design type (e.g., box-girder bridge versus suspension bridge)
- Dimensions
- Elevations
- Proposed / remaining design life
- Age and condition (for existing assets)
- Design criteria

3.2.3 Step 3 – Identify Climate Stressors that May Impact Infrastructure Components

This step involves documenting the climate-related variables typically considered in the planning and design of the type of facility being investigated.[16] The design standards associated with these variables, if applicable, should also be noted (e.g., a state transportation department may have a policy that all bridges and their approaches must be designed to pass the 100-year storm without overtopping). Sometimes these variables affect the facility directly (e.g., temperature's influence on asphalt mix design) and in other cases they trigger other events that are the cause for concern. For example, more frequent and intense precipitation events could lead to more flooding which would need to be accounted for in bridge design. Furthermore, warmer temperatures could lead to more parasitic insects (such as the mountain pine beetle) being present in an area. The insects may kill trees in the watershed, contributing higher amounts of debris during floods that increase the chance of culverts or bridges becoming clogged.

For most facilities, several climate stressors will be relevant to designers. This can be handled in a couple of different ways when conducting facility-level adaptation assessments using the *Process*. The most thorough approach involves considering all of the climate variables as part of the same assessment. This provides the most holistic picture of climate change impacts (including their potential interactions). That said, considering all impacts can add significant effort to the assessment and may not be practicable or necessary in all cases. One alternative approach (and the one taken in this report) is to look only at the climate stressor that is likely to be most significant to the facility. This requires some degree of engineering judgment up front to determine which stressor might be most impactful. It should be noted that while this approach is appropriate in many cases, there are likely to be some instances where multiple climate stressors should be considered together, particularly if interactions are anticipated amongst the impacts. The development of adaptation options might also benefit from a holistic approach in some cases as there may be synergies (or dis-synergies) among the options that could significantly affect which options are most cost effective.

3.2.4 Step 4 – Decide on Climate Scenarios and Determine the Magnitude of Changes

After the climate-related variables that affect the facility have been identified in Step 3, the next step is to use climate model projections to determine whether and how much each of the variables of concern may change in the future. The information gathered for each variable should, if possible, relate to the design standards identified in Step 3. However, due to the limitations of current climate models and the difficulties in developing future projections, this

[16] A series of tables in the *Task 2: Climate Variability and Change in Mobile, Alabama* report (USDOT, 2012) relate specific climate variables to various transportation modes. Other useful references include (Armstrong, Keller, Flood, Meyer, and Hamlet, 2011) and the Engineering Options for Climate Stressor Mitigation tool developed as part of the *National Cooperative Highway Research Program (NCHRP) 20-83(05): Climate Change and the Highway System* project (Meyer, Flood, Keller, Lennon, McVoy, Dorney, Leonard, Hyman, and Smith, 2014).

may not always be possible in which case proxy variables may have to be substituted. Section 4.4.1 of this report provides a case study for a culvert illustrating the use of both a 100-year precipitation depth from climate models and a possible proxy for this value, the National Oceanographic and Atmospheric Administration (NOAA) Atlas 14 90% upper confidence limit, since similar climate projections are not yet available.

The future values and probability of occurrence for many climate variables are uncertain due to the inability to predict the quantity of greenhouse gases that will be emitted in the future and how precisely the Earth's climate system will respond. The *General Process for Transportation Facility Adaptation Assessments* incorporates a scenarios approach to deal with this uncertainty, an established practice in the climate change impacts assessment field. A scenarios approach involves generating a variety of climate scenarios to capture the range of possible future values of each climate variable.

Climate scenarios are typically based on assumptions regarding the amount of greenhouse gases that will be emitted into the atmosphere, a key driver in the ultimate level of climate change that will occur. Scenarios can also consider different scientific assumptions on the climate system's response to a given amount of greenhouse gas emissions (i.e., climate sensitivity) and the behavior of the climate system as captured by different climate models. Within the parameters of a scenario and the model of Earth's climate used, it is possible to develop the conditional probability[17] of a future climate event occurring, a key input to the economic analysis in Step 8. See the *Task 2: Climate Variability and Change in Mobile, Alabama* report[18] for a detailed example of how climate scenarios can be acquired or generated for a wide range of climate stressors.

As few as two scenarios showing high and low amounts of change in climate may be sufficient to capture the range of possible futures, but any number of scenarios can be utilized depending on the needs of the project. The more scenarios that are used, the more confidence one can have that all future possibilities have been considered. Practitioners should keep in mind that each scenario that is added will necessitate that additional time and resources be spent on the analysis during execution of the *Process*.

At a minimum, climate projections for each scenario should be gathered for the period extending out to the anticipated end of the facility's design life. Scenarios can also be analyzed at multiple points over the facility's design life; for example, if a facility is designed to last through 2100, it would make sense to develop scenarios for both the mid-century and end of century timeframes.

After gathering the climate projections and considering the full range of potential climate changes, it might be determined that none of the climate variables are expected to change

[17] Conditional probability refers to a probability subject to conditions, in this case, the conditions included in the chosen climate scenario.
[18] USDOT, 2012

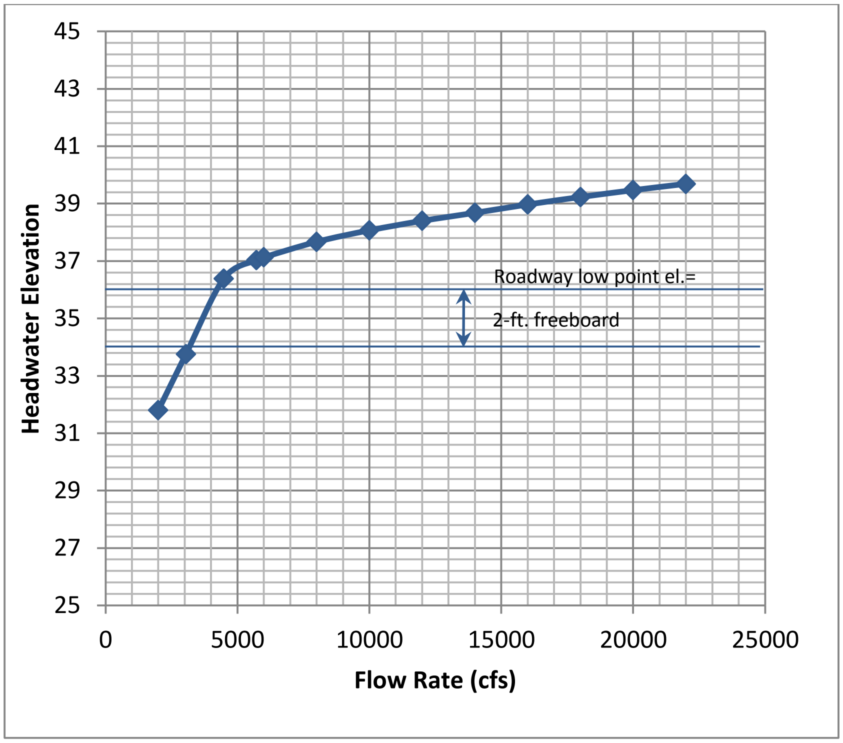
significantly or in a way that would threaten the facility. If this is the case, then the assessment is complete and no further climate adaptation analysis is required at this time. This conclusion should be reassessed periodically as new understandings and projections of future climate develop. If, however, the magnitudes of the projected change are significant, then the analysis should continue to Step 5.

3.2.5 Step 5 – Assess Performance of the Existing / Proposed Facility

The purpose of this step is to ascertain whether the existing / proposed facility is performing as expected and whether it will continue to do so under each of the possible future climate scenarios selected in Step 4. For existing facilities, the asset should be assessed with respect to current climate data to determine if the asset meets present-day design standards. Any analysis should state assumptions made regarding design standards if they are not available. This can be an important factor when deciding on whether to do adaptation or not; if the facility does not even meet present climate-related design standards, then that is further reason to undertake an upgrade.

The standards for which performance is assessed can vary depending on the asset being studied. Whenever possible, however, performance should be assessed against the design standards tied to the climate variables of interest that were noted in Step 3. For example, if a bridge and its approaches were required not to overtop during the 100-year storm, one would test each scenario's 100-year storm to determine if it overtops the facility. A graph, such as a stream stage-discharge curve for culverts or bridges (see Figure 4 for an example), can be a useful way to summarize performance.

Figure 4: Example Stage-Discharge Curve for a Culvert



In assessing impacts, one should broaden the analysis beyond narrowly looking only at the implications to the facility being studied and consider ancillary impacts caused by the subject facility's failure. For example, an undersized culvert might lead to flooding of properties upstream: these should be noted and studied. One should also determine what the implications would be (if any) on surrounding (e.g. downstream) facilities if the asset failed. These considerations are important to understand and can make a large difference in the economic analysis.

The takeaway of this step should be an understanding of which scenarios the facility performs adequately under and which scenarios it does not. One should be on the lookout for possible tipping points where damage greatly increases when the climate stressor reaches a certain level. To the extent that multiple timeframes were tested for each scenario, some sense of the timeframe around when performance is likely to fail to meet expectations will also be possible. It should be noted that the results will likely be subject to uncertainties in both the exact nature of how the climate changes will manifest themselves and precisely how the facility will respond.

At the conclusion of Step 5, it is possible that the facility is found to perform adequately under the full range of potential climate changes that it could experience throughout its intended design life: if this is the case, no further analysis is necessary at this time and the assessment is complete until new climate projections are released requiring a revisiting of this conclusion.

3.2.6 Step 6 – Identify Adaptation Option(s)

Adaptation options should be identified for each scenario that does not meet design expectations as determined in Step 5. The adaptation options could be planning or design-oriented; in many cases, the best adaptation may be to avoid a hazardous area altogether rather than to design an engineered solution. Examples of adaptive design solutions are presented within the case studies of this report. These are not meant to cover the full range of options available, but rather serve as examples of the types of actions one might consider for similarly situated facilities in other communities. A more complete list of general adaptation options for bridges, culverts, pavement, drainage systems, and slopes can be found in the Engineering Options for Climate Stressor Mitigation tool developed as part of the *National Cooperative Highway Research Program (NCHRP) 20-83(05): Climate Change and the Highway System* project.[19] It should be noted that the development of adaptation options is still in its infancy and this is an area worthy of much additional research and innovation.

In general, at least one adaptation option should be identified for each climate scenario selected. These options then become the basis for analyzing performance and decision-making. Adaptation options could consist of either one action (raising a bridge) or a package of actions that address a climate stressor or set of climate stressors (e.g., raising a bridge and armoring the approach embankments). Each option should be developed so that applicable design standards are met under the given scenario realizing that, as is the case with such standards generally, some exceptions may be necessary based on unique site constraints. Alternately, a flexible design option, whereby the design can be changed over time to respond to observed changes in conditions, may also be considered. Such an approach has the advantage of managing some of the uncertainty inherent in climate projections by avoiding committing to a certain solution until it becomes clearer that the scenario is becoming a reality. Note that there are likely to be multiple possible ways to achieve design standards under any given scenario (e.g., to accommodate higher flows through a culvert, one could add additional culverts or convert the culvert to a bridge): it is up to the project team to decide on how many options to develop and test, keeping in mind that each additional option will add additional work effort to the assessment.

Whatever approach is chosen, a high-level cost estimate to construct and maintain each adaptation option should be developed. This will be used in the economic analysis in Step 8. Although low-cost no-regrets adaptation options might be available, this is not always the case. Given the uncertainties of future climate, when adaptation involves much higher costs, some agencies might be tempted to consider a “roll-the-dice” strategy whereby proactive adaptations are not taken for existing facilities and adaptations are not made until damage occurs or planned major rehabilitation or replacement is required. The benefits and costs of such an approach can

[19] Meyer et al., 2014

be compared to those of a more proactive adaptation approach in the economic analysis discussed later.

3.2.7 Step 7 – Assess Performance of the Adaptation Option(s)

This step involves assessing the performance of each adaptation option under each potential climate change scenario selected in Step 4. This analysis is similar to Step 5 except that it is performed on the adaptation options as opposed to the existing facility or, in the case of new facilities, the standard design without adaptations. The key determination is whether each adapted facility satisfies its mandated performance standard (e.g., a 50-year design storm for a culvert) under each scenario. As in Step 5, the development of graphs such as stage-discharge curves (see Figure 4) can be a useful way to summarize this information.

3.2.8 Step 8 – Conduct an Economic Analysis

An economic analysis is of great value to informing decision-making on project level adaptation assessments. The analysis enables one to determine how the benefits of undertaking a given adaptation option, defined as the costs avoided[20] with adaptation, compare to its incremental costs under each of the possible future scenarios developed in Step 4.

The level of effort for the benefit-cost analysis should be scaled to match the magnitude of the project. Many techniques are emerging for conducting economic analyses of climate risk management strategies. Examples include Monte Carlo analysis (see Step 8 in Section 4.4.1 for an example) and the more simplified methods for scenario-based risk assessment planning discussed in Kirshen et al. (2012)[21] and *What Will Adaptation Cost? An Economic Framework for Coastal Community Infrastructure*.[22] The Federal Transit Administration (FTA) has released a *Hazard Mitigation Cost Effectiveness (HMCE) Tool* with an associated User Manual and trainings that facilitates the documentation of costs and benefits and calculates the final cost benefit ratio[23]. The approach used and the level of detail will be dependent upon the total value of the facility being studied, the technical abilities of the project team, and other resource constraints.

Whichever approach is utilized, the basic technique involves estimating the expected impact costs from climate or weather events over the life of the facility and discounting them to determine the present value of these expected costs. This is done for the base case situation of the existing facility or standard new design and repeated for each adaptation option under each climate change scenario selected in Step 4. The (lower) costs with the adaptation options in place

[20] Costs avoided might include the costs of damage to the facility, clean-up costs, costs to the traveling public due to detours and delays, death and injury costs, costs to businesses and others dependent on the transportation facility, potential costs to surrounding land uses from impacts generated by the facility (e.g., an undersized culvert resulting in upstream flooding that affects neighboring properties), and, potentially, environmental impacts generated by the facility (e.g., a coastal causeway that prevents marsh migration inland as sea levels rise).
[21] Kirshen, Merrill, Slovinsky, and Richardson, 2012
[22] Eastern Research Group, 2013.
[23] FTA, 2014.

can then be compared to the base case costs to determine the cost savings expected as a result of adaptation. The net present value and/or the benefit-cost ratio of each adaptation option can then be computed and compared among the adaptation options. The results can be presented in tables showing each adaptation option's cost-effectiveness under each scenario (see Table 3 for an example).

Table 3: Example Benefit-Cost Analysis Outputs for an Adaptation Option

Climate Scenario	1	2	3	4	5	Average (mean) of All Scenarios
Description of Scenario	Observed 1980–2009 with Current Land-use	Observed 1980–2009 with Future Land-use	NOAA 90% Upper Confidence Limit	"Wetter" Narrative	"Drier" Narrative	
Present Value of Costs	$2.5m	$2.5m	$2.5m	$2.5m	$2.5m	$2.5m
Present Value of Benefits	$10.3m	$11.8m	$18.5m	$97.5m	$8.9m	$29.4m
Net Present Value	$7.8m	$9.3m	$16.0m	$95.0m	$6.4m	$26.9m
Benefit-Cost Ratio	4.1	4.7	7.4	38.9	3.6	11.7
Probability that Benefit-Cost Ratio will be over 1	38%	44%	71%	99%	35%	N/A

Decision-makers can then consult the tables and look for (1) adaptation options that have benefit-cost ratios greater than one and (2) the adaptation option that performs best across the full range of scenarios tested (the robust option). It should be noted that the economic analysis does not in and of itself always provide an answer as to whether an adaptation option makes financial sense. There is no guarantee that an adaptation option that performs cost-effectively under each scenario will exist: an option may be cost-effective under one scenario but not another. Likewise, there may be no one adaptation option that is the most robust economic performer across all scenarios. In every case, but in these cases especially, trade-offs will have to be made and the community's and / or facility owner's risk tolerance evaluated to help choose the "best" option from a financial standpoint. Ultimately, uncertainties associated with projected changes in climate tend to magnify subjective factors during decision-making, including risk tolerance of customers and funders of transportation assets.

3.2.9 Step 9 – Evaluate Additional Decision-Making Considerations

As in other areas of transportation decision-making, the cost-effectiveness of adaptation options is not the only factor important to making wise investment decisions. Other factors that can be difficult to monetize (for benefit / cost analysis) should also be considered before a final decision is reached. These may include:

- Broader project sustainability beyond just climate change impacts (i.e., the "triple bottom line" of social, environmental, and economic concerns)
- Project feasibility and practicality
- Ongoing maintenance needs
- Maintenance funds availability
- Capital funds availability
- Stakeholders' (public and government agencies) tolerance for risk of service interruption and associated costs of all types (note: this affects how the economic analysis is perceived as well)
- Stakeholders' expected quality or level of service

Much of this information will need to be gathered through community outreach and stakeholder engagement processes.

3.2.10 Step 10 – Select a Course of Action

Once as much information as possible has been gathered on both economic and non-economic factors, decision-makers should weigh the information presented and decide on a course of action. Those involved should keep in mind that adaptation does not always make sense from a financial feasibility or community acceptance standpoint and a decision to take no action may be justified in some cases.

3.2.11 Step 11 – Plan and Conduct Ongoing Activities

Once a decision has been made on a course of action, a management plan for the facility should be developed. At a minimum, the management plan should contain an element of monitoring to determine if the facility is performing as expected over time. If an adaptation option was used, estimates of the costs saved from implementing the adaptation could be developed so that the benefits of the adaptation are documented and compared to its costs. This information could prove beneficial in future years as the community continues to make decisions on which adaptations, if any, make sense in various situations.

4. Case Studies

4.1 Introduction

Ten case studies were conducted to demonstrate the application of the *General Process for Transportation Facility Adaptation Assessments* (the *Process*) to a range of design problems at 10 specific facilities in Mobile. In addition, operations and maintenance (O&M) opportunities for adaptation were examined in a final case study. These case studies illustrate how engineering design processes may be augmented to incorporate climate change and extreme weather considerations.

The primary criteria for selecting an asset or facility in the case studies included:

- The facility's relative vulnerability ranking as determined in the *Screening for Vulnerability* report.[24] These screening-level results include the magnitude of the direct and secondary consequences of failure or interruption as a factor.
- Diversity in the range of modal assets and facilities being examined
- Degree to which lessons learned and processes illustrated for a given asset/facility could be applied elsewhere in the Gulf Coast or the nation
- Availability of non-proprietary information (existing plans and design data) for the subject asset
- Available resources

4.2 Selection of Cases

Table 4 lists the climate stressors and asset types chosen for study and the specific case study facilities that were investigated. Figure 5 shows the location of these facilities.

4.3 Approach

Since the case studies are intended to demonstrate the application of the *Process*, each case study is structured using the 11 steps of the *Process* (see Section 3.2 for a detailed discussion of each step). The step-by-step summaries thus provide detailed examples of how the *Process* can be applied to a variety of climate stressors and facility types.[25]

The projected climate data used in the case studies were developed during earlier tasks of this project. For more information on how the climate information was developed and packaged into climate "narratives" for purposes of vulnerability assessments, please see the Task 2 and Task 3.1 reports at www.fhwa.dot.gov/environment/climate_change/adaptation/ongoing_and_current_

[24] USDOT, 2014
[25] Detailed calculations and model outputs are not included in this report, but are available upon request by contacting the FHWA authors of this report.

research/gulf_coast_study/phase2_task2/sensitivity_report/ and www.fhwa.dot.gov/environment/climate_change/adaptation/ongoing_and_current_research/gulf_coast_study/phase2_task3.

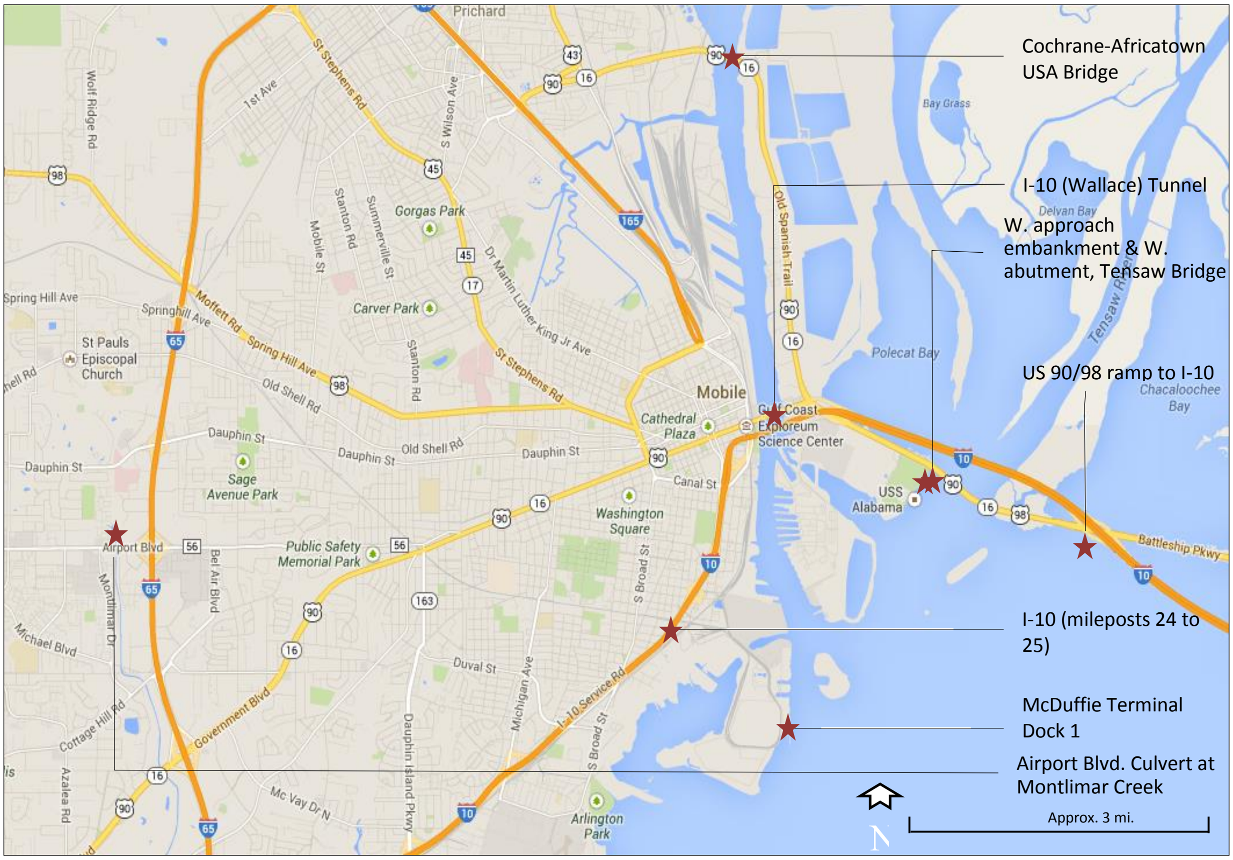
Due to the nature of this project as a broad study across many facilities, none of the case studies represent a full application of every step in the *Process*. For example, none of the cases included a full evaluation of additional decision-making considerations in Step 9, as public outreach related to specific adaptation measures was beyond the scope of this project. In addition, only the study of a culvert exposed to changes in precipitation patterns (Section 4.4.1) included an economic analysis. Recognizing these limitations, the selected courses of action discussed in Step 10 of each case study are preliminary recommendations, subject to additional findings from completion of full economic analyses and outreach with local stakeholders.

Table 4: Listing of Case Studies by Climate Stressor, Facility Type, and Damage Mechanism

Report Section	Climate Stressor	Asset Type	Damage Mechanism	Case Study Facility	**Indicator-Based Vulnerability***
4.4.1	Increased precipitation	Culvert	Flooding	Airport Boulevard culvert at Montlimar Creek	Low
4.4.2	Sea level rise	Navigable waterway bridge	Clearance for navigation	Cochrane-Africatown USA Bridge	Medium
4.4.3	Sea level rise	Bridge approach embankment	Slope erosion	West approach embankment of the US 90/98 Tensaw-Spanish River Bridge	High
4.4.4	Higher storm surge	Bridge abutment	Abutment scour	West abutment of the US 90/98 Tensaw-Spanish River Bridge	Medium
4.4.5	Higher storm surge	Bridge segment	Wave forces, bridge pier scour	US 90/98 ramp to I-10 eastbound at Exit 30	High
4.4.6	Higher storm surge	Road alignment	Overtopping / slope erosion	I-10 (mileposts 24 and 25)	Low
4.4.7	Higher storm surge	Coastal tunnel	Flooding	I-10 (Wallace) Tunnel	Medium
4.4.8	Higher storm surge	Shipping pier	Waves	McDuffie Coal Terminal, Dock 1	Low
4.4.9	Temperature changes	Roadway pavement	Rutting, concrete crackings	Generic location	Low
4.4.10	Temperature changes	Continuously welded rail	Buckling, pull-aparts	Generic location	Medium
4.4.11	Various	Operations and maintenance activities for various facilities	Various	Alabama Department of Transportation, City of Mobile, and Mobile County operations and maintenance practices	High

* An asset's estimated vulnerability was calculated previously in this project, as detailed in the report *Task 3.1: Screening for Vulnerability*. The vulnerability designations developed within the *Screening for Vulnerability* report provide initial estimates of an asset's vulnerability to a particular climate stressors based on a series of vulnerability "indicators" applied broadly across the transportation system. However, in this *Task 3.2 Engineering Analysis and Assessment* report, the more refined engineering analyses performed may result in different determinations as to an asset's actual vulnerability.

Figure 5: Case Study Locations[26]



Also, the case studies are very specific to one type of asset for one type of climate stressor at a particular location. It was beyond the scope of this analysis to include all components of a facility design when considering adaptation strategies, or to consider the entire range of climate stressors that might affect each case study facility over time. Instead, the climate stressor that is likely to be most impactful was chosen for study. Actual project-level applications of the *Process* may, however, attempt to be holistic in their consideration of climate impacts. All relevant components of a facility that could be impacted by climate change may be considered in these studies, including an assessment of how climate change may impact the land uses being served by that facility and vice versa. Complex interactions between land use, demographics, and changes in climate will all influence the demand on the facility; understanding the local context and the relative influence of these factors is critical to making informed adaptation decisions. Similarly, considering the possible interrelationships of various climate stressors may be important when undertaking actual assessments.

Lastly, it should be noted there are several key facilities in the transportation system that were not included as case studies. This includes highway and railway signs and signals,

[26] Source of base map: Google Maps (as modified)

communication networks, power supplies to all major transportation facilities, and other key components of the system. Additional assessments will be needed to determine how best to adapt the design of these facilities to a changing climate.

4.4 Individual Case Studies

4.4.1 Culvert Exposure to Precipitation Changes – The Airport Boulevard Culvert over Montlimar Creek

Introduction

Culverts are an important and sometimes underappreciated component of the highway system that can be highly sensitive to climate change. This section of the report illustrates how the *General Process for Transportation Facility Adaptation Assessments* can be applied to culverts by using the Airport Boulevard culvert over Montlimar Creek (see Figure 6) as a case study. The goals of this assessment are to (1) determine whether projected changes in precipitation patterns associated with climate change will pose a flood risk to the facility and, if so, (2) to develop and evaluate adaptation options for managing that risk. This case study does not go into all the details of the culvert design process, but rather focuses on one particular design consideration that is affected by climate change; precipitation and its impact on streamflow.

Case Study Highlights

Purpose: Evaluate whether culvert design is sufficient under projected levels of 24-hour precipitation

Approach: Using projected 24-hour rainfall values and NOAA temporal rainfall distributions, peak flows to the culvert were modeled using the Win TR-20 Program, considering both existing and future land use conditions. Then, hydraulic analyses were conducted to determine the performance of the culvert under current and future flows, using the HY-8 Version 7.2 program. Performance was assessed by determining whether at least 2 feet of freeboard would be achieved during a 25-year event, which is the standard used by the city of Mobile for this type of culvert.

For each adaptation option, flooding impacts of a 100-year event on surrounding areas were evaluated. An economic analysis of adaptation options was conducted using a Monte Carlo process.

Findings: Culvert design is sufficient for current conditions, but the roadway could be overtopped under projected future conditions

Viable Adaptation Options:

- Increase number of culvert cells
- Increase size of culvert cells
- Implement Regional Drainage Area Management practices

Other Conclusions: An economic analysis could be done using a Monte Carlo process, which is effective in managing the uncertainties associated with the climate projections. For this case study, the economic analysis indicates that increasing the number of the culvert cells is more likely to be cost-effective under various climate futures than increasing the size of the culvert cells.

Figure 6: Photo of the Airport Boulevard Culvert over Montlimar Creek, Upstream Side

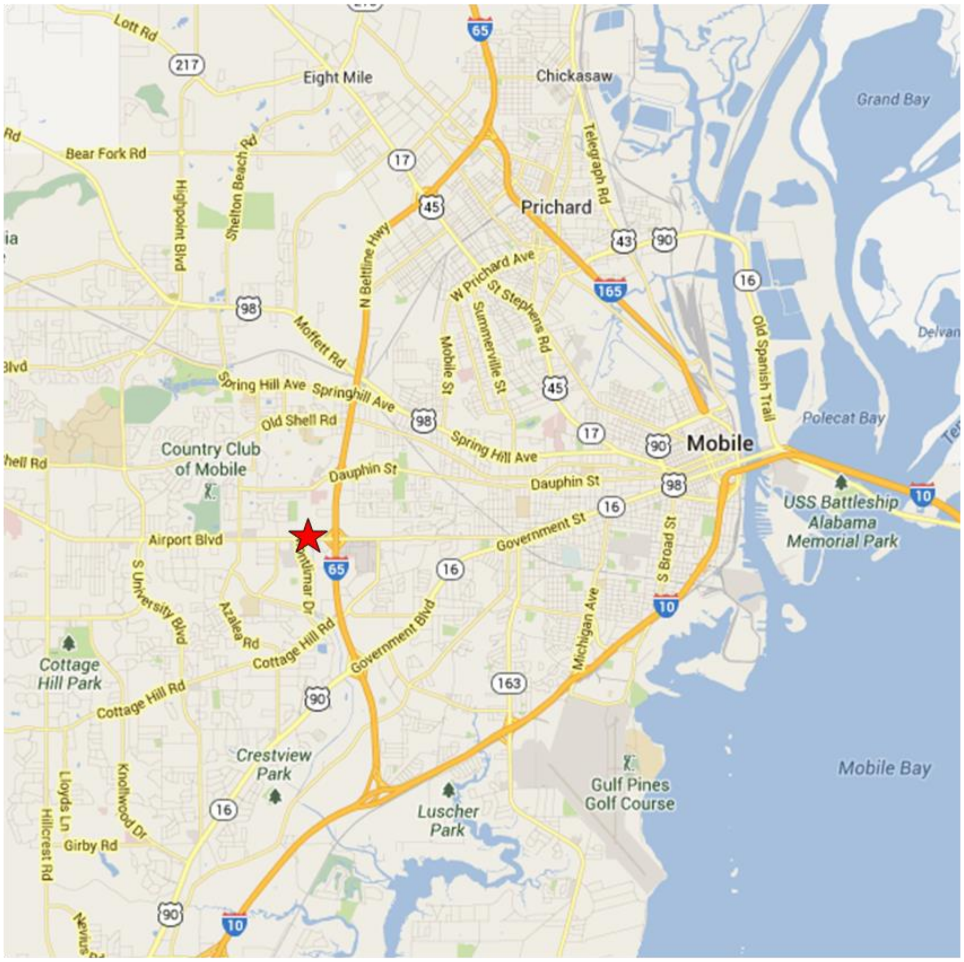
Adaptation options considered for handling projected increases in streamflow include adding two additional cells to the existing culvert or raising the road and replacing the existing culvert with one that has larger cells. Unlike the other case studies, this case study also includes an economic analysis to show how one might use the financial performance of each adaptation option developed to aid in decision-making. Based on this analysis, the preliminary recommended action is to expand the culvert by adding two additional cells to the existing facility. The case study concludes with lessons learned about the application of the *General Process for Transportation Facility Adaptation Assessments* to culvert design. As with any culvert design, the designer must consider the culvert as an element within the drainage *system* and determine the effectiveness of and the influence on performance of all of the drainage system elements.

Application of the General Process for Transportation Facility Adaptation Assessments

Step 1 – Describe the Site Context

The Airport Boulevard culvert over Montlimar Creek is located on the west side of the City of Mobile (see Figure 7) immediately to the west of Airport Boulevard's intersection with I-65. Airport Boulevard is a major six-lane east-west arterial linking downtown Mobile with its western suburbs and the regional airport (located approximately seven miles to the west). The main road is flanked by a pair of two-lane bi-directional frontage roads that also utilize the culvert. The two primary regional shopping centers, the Springdale Mall and the Bel Air Mall, are immediately to the east of the I-65 interchange, adding to the traffic loads on the facility. In addition to personal and commercial vehicles, Wave Transit System bus routes 1 and 18 use the segment of Airport Boulevard passing over the culvert.

Figure 7: Location of the Airport Boulevard Culvert within the Mobile Metropolitan Area[27]



The roadway network in the vicinity of the culvert is fairly dense with Dauphin Street providing an alternative crossing of Montlimar Creek about 0.6 miles to the north and Michael Boulevard and Pleasant Valley Road crossing the creek approximately one mile to the south. Alternative stream crossings are an important consideration when evaluating possible detour routes if Airport Boulevard is closed due to inundation.

Hydrologic Setting

Montlimar Creek is a man-made feature that was designed as a drainage canal through Wragg Swamp, a wetland that once existed in the area surrounding the I-65 interchange with Airport Boulevard. Figure 8 shows two historical topographic maps illustrating the swamp and the major changes to the area’s hydrology that came with the construction of I-65 and urbanization of the area. The figure shows that the Eslava Creek once flowed eastward through Wragg Swamp prior to the construction of I-65, at which time the creek was bifurcated into separate branches west

[27] Source of base map: Google Maps (as modified).

and east of the interstate. The east branch of Eslava Creek continued in a southeast direction to the Dog River; the west branch was channeled southward into what is now the Montlimar Creek, keeping all these waters to the west of I-65 before draining into the Dog River.

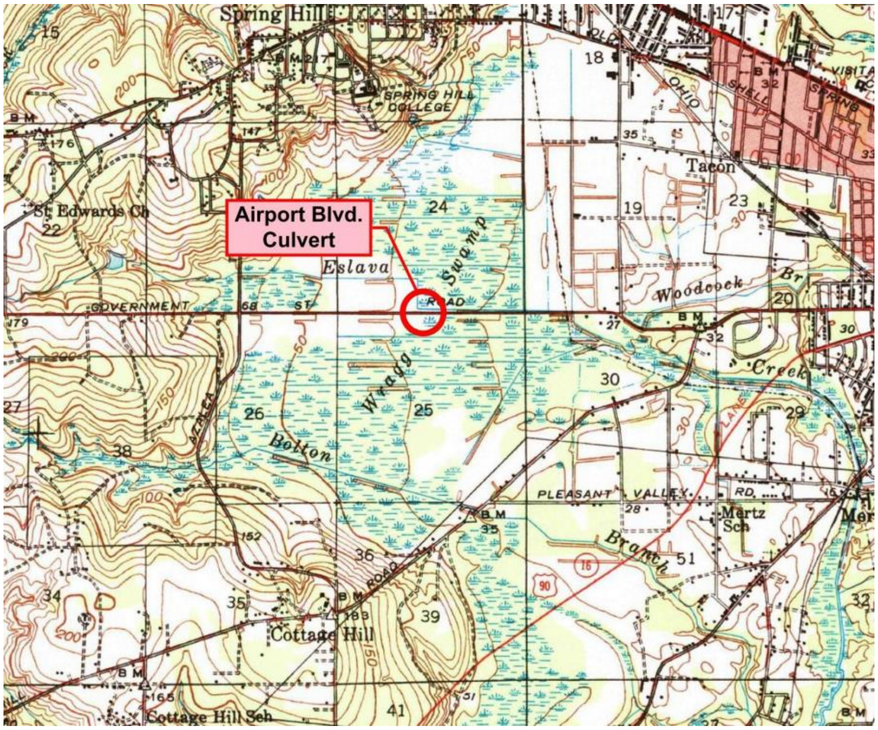
Figure 9 shows the present-day drainage area for water flowing through the Airport Boulevard culvert. The total drainage area to the culvert is 3.3 square miles (8.5 square kilometers).

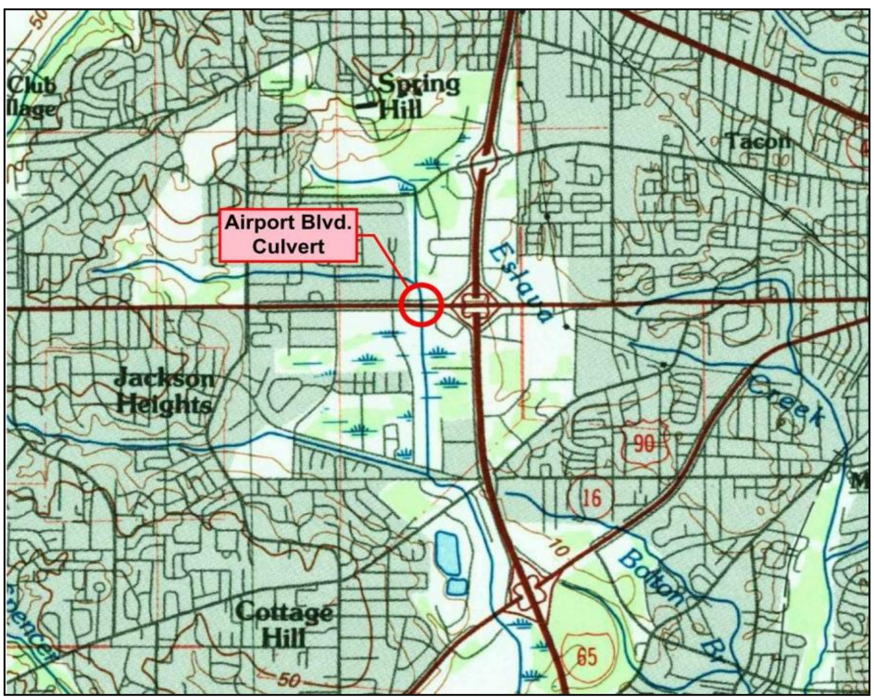
Today's Montlimar Creek consists of a series of man-made channels surrounded by urban development. The segment just upstream and downstream of the Airport Boulevard culvert is a trapezoidal channel with grassy vegetation on the side slopes. The surrounding area consists almost entirely of paved commercial areas. The channel has an approximate top width of 120 feet (36.6 meters) and an approximate depth of 10 feet (3 meters). The bottom width is approximately 40 feet (12.2 meters) and side slopes are 25%.

The Airport Boulevard culvert over Montlimar Creek lies within a Federal Emergency Management Agency (FEMA) regulated floodplain. Figure 10 shows the 100-year (1% annual chance) and 500-year (0.2% annual chance) floodplain boundaries developed based on the FEMA flood insurance study for Mobile County.[28] Importantly, the flood boundaries presented in the FEMA flood map do not account for the area east of the culvert. According to the flood insurance study, the flood elevations for Montlimar Creek at Airport Boulevard were determined to be approximately 37 and 38 feet (11.3 and 11.6 meters) for the 100-year and 500-year floods, but the floodplain boundaries do not include the area to the east of the culvert which sits at a lower elevation. Further investigation into flood elevations and extents using hydrologic analysis methods is discussed in subsequent sections.

[28] FEMA, 2010d

Figure 8: Topographic Maps from 1940 (Top) and 1994 (Bottom) Illustrating the Significant Changes to Local Hydrology with the Construction of I-65 and Urbanization[29]





[29] Source of basemaps: USGS, 2012 (as modified). Note: Maps not to scale.

Figure 9: Drainage Area to the Airport Boulevard Culvert
Showing LIDAR Contours and Time of Concentration (Tc) Path

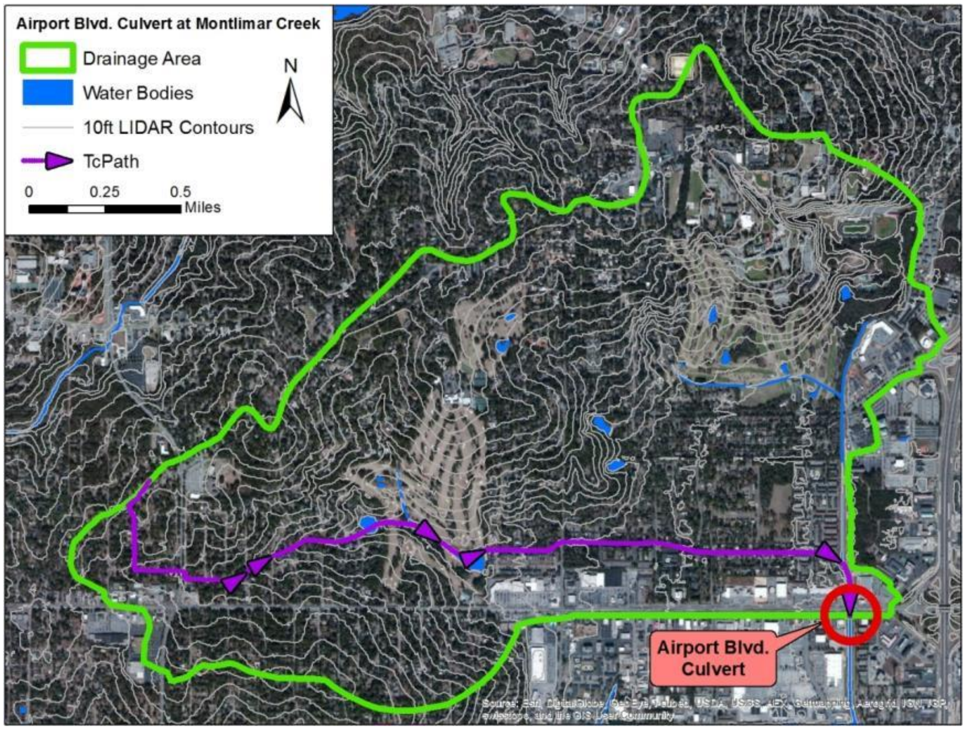
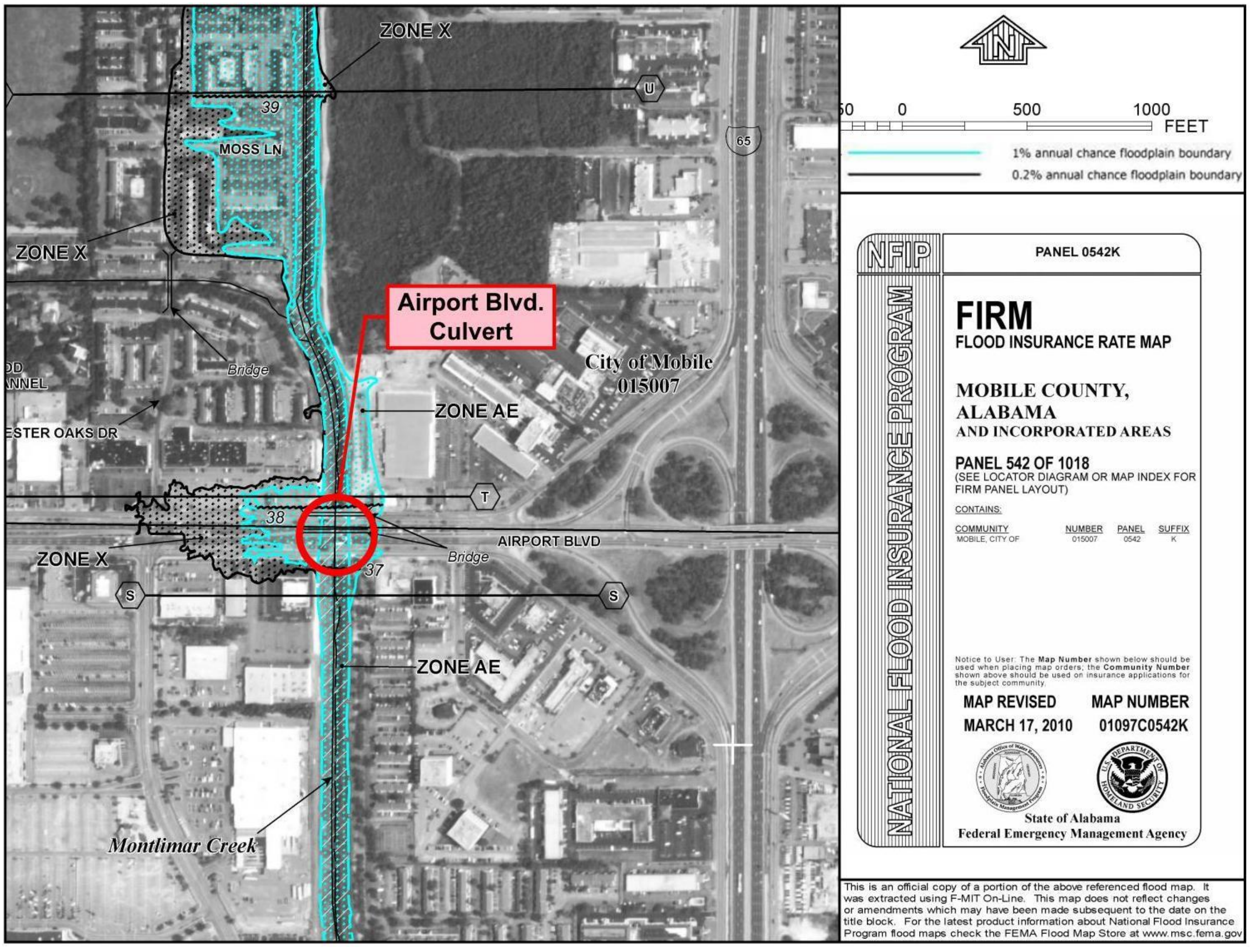


Figure 10: FEMA 100-Year (1% Annual Chance) and 500-Year (0.2% Annual Chance) Floodplains near the Airport Boulevard Culvert [30]



Step 2 – Describe the Existing Facility

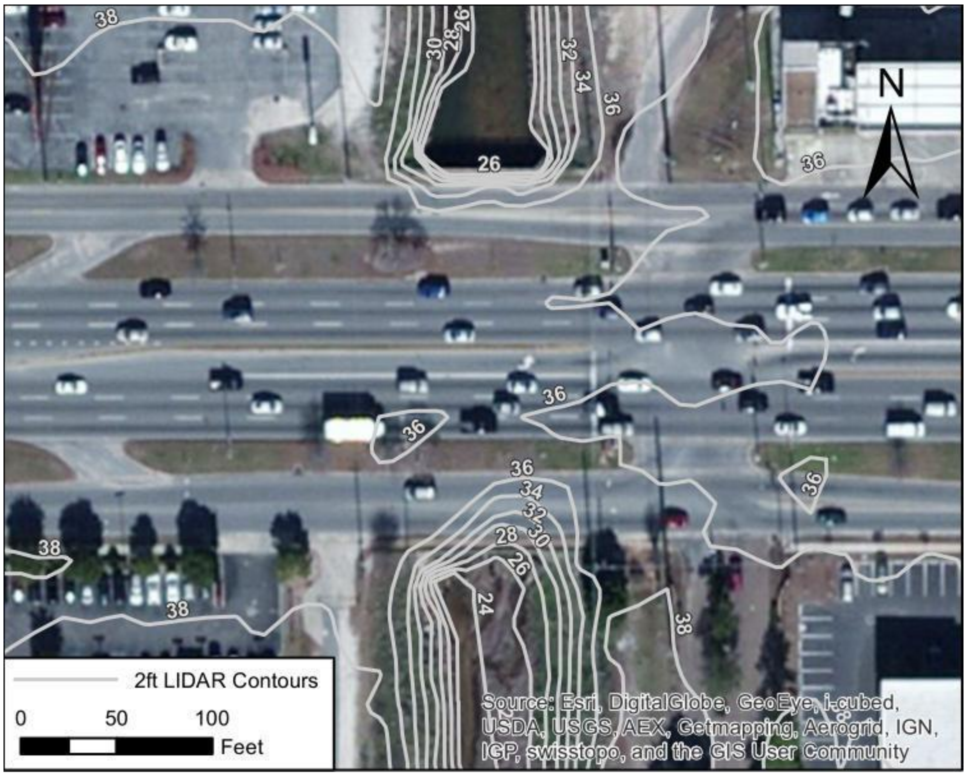
The Airport Boulevard culvert is owned and maintained by the City of Mobile. According to 2009 field inspection data provided by the City,[31] it was built in 1960. The longitudinal length of the culvert was originally approximately 70 feet (21.3 meters). A subsequent widening of Airport Boulevard lengthened the culvert by 65 feet (19.8 meters) north and 65 feet (19.8 meters) south to today's total length of 200 feet (61 meters). Figure 11 shows a plan view of the culvert crossing. Figure 12 and Figure 13 are photos of the culvert taken in June 2013. The culvert was designed for a 25-yr. storm with the maximum allowable headwater 2 feet (0.61 meters).

Figure 14 provides a diagram of the facility showing its current dimensions.

[30] FEMA, 2010a
[31] City of Mobile, 2009

The structure is a four cell concrete box culvert with each cell having a 12 foot (3.7 meter) span (width) by eight foot (2.4 meter) rise (height). The fill depth to the roadway was measured to be two feet two inches (0.7 meters). The slope of the culvert from its inlet to its outlet was not available from project data but was measured to be approximately 1% based on LIDAR contours. Based on the 2009 field inspection (performed at some time subsequent to the lengthening), the culvert was in generally sound condition. It was determined at the time that there were no deficiencies that would affect the load bearing capacity of the culvert, but there was some spalling[32] of the concrete and exposed rebar in multiple locations at the top face.

Figure 11: Plan View of the Airport Boulevard Culvert Crossing Showing LIDAR Contours

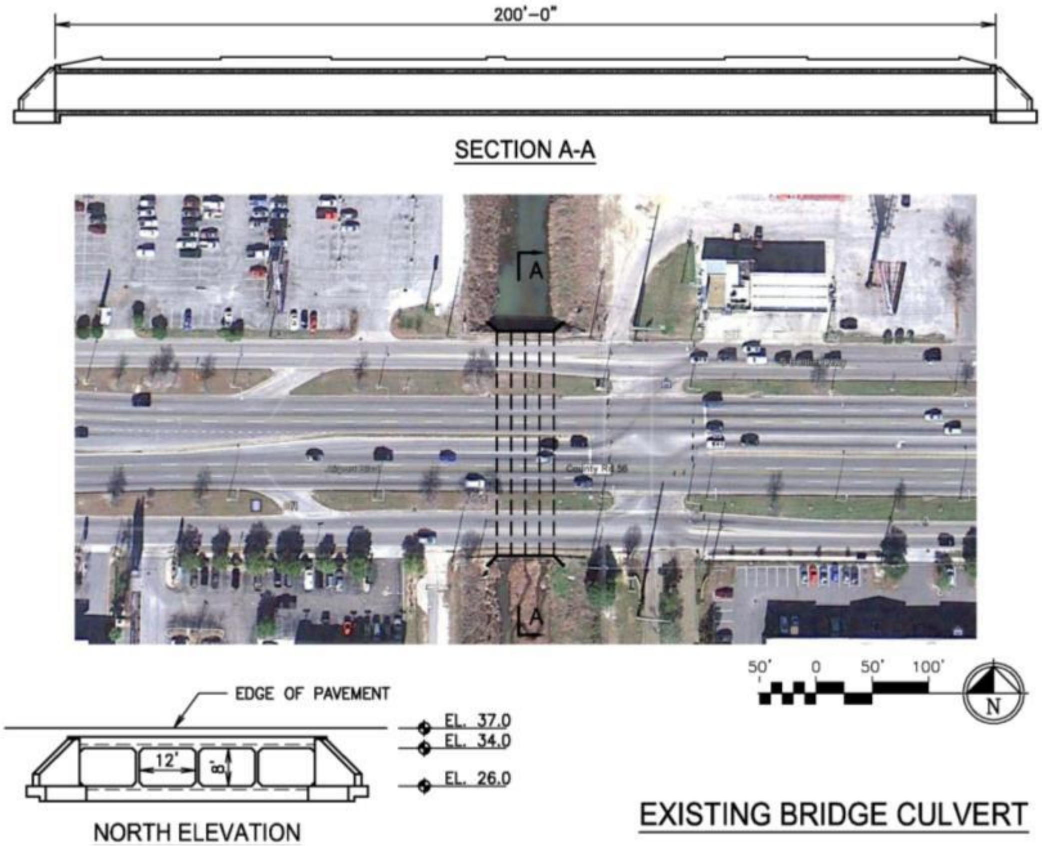


[32] Spalling refers to a deterioration condition of older concrete where fragments of the concrete material face have broken off. Spalling is usually caused by corrosion of the steel reinforcement bars embedded in the concrete.

Figure 12: Photo of the Airport Boulevard Culvert over Montlimar Creek, Upstream Side

Figure 13: Photo of the Airport Boulevard Culvert over Montlimar Creek, Downstream Side

Figure 14: Plan for the Airport Boulevard Culvert over Montlimar Creek



Step 3 – Identify Climate Stressors That May Impact Infrastructure Components

In this culvert study, precipitation (and the resulting flow) is the primary environmental factor affecting the design that is expected to be affected by climate change. Accordingly, precipitation is the environmental factor selected for this analysis.

Step 4 – Decide on Climate Scenarios and Determine the Magnitude of Changes

Four precipitation scenarios were considered for this adaptation assessment based on current climate and projected climate changes:

- Observed precipitation values (1980-2009).
- The NOAA Atlas 14 90% Upper Confidence Limit.
- The "Wetter" narrative precipitation ranges developed under this project using downscaled climate data for Mobile County and a range of climate scenarios.[33]
- The "Drier" narrative precipitation ranges developed under this project using downscaled climate data for Mobile County and a range of climate scenarios.

[33] For more information on the climate information referenced here, please refer to *Climate Variability in Change in Mobile, AL* (USDOT, 2012) and *Screening for Vulnerability* (USDOT, 2014).

All four scenarios considered 24-hour precipitation depths as this is the shortest storm duration reported from the available climate models for future conditions. Return periods[34] analyzed include the two-, five-, 10-, 25-, 50-, and 100-year storm. The range of 24-hour precipitation values for each scenario and return period is shown in Table 5, along with the NOAA Atlas 14 Average Baseline value for comparison. Table 6 shows how the projected precipitation values would translate to present date return periods.

A standard culvert design project would typically rely upon the average precipitation data as presented by the NOAA Atlas 14 Average Baseline. However, to provide consistency in this study with the work performed under prior project tasks and the modeled climate projections, the Atlas 14 data was not utilized as the current baseline condition. Instead, the Observed precipitation values represent the model baseline precipitation values as documented in the *Climate Variability in Change in Mobile, AL* report.[35] The observed data comprise the average modeled values across the five weather stations in the Mobile area. The project team considered using the observed data for the station closest to the project site (the Mobile station) but found that the average across all five stations more closely matched the NOAA Atlas 14 estimates for the site location. The use of the five station aggregate also provides the advantage of having a larger sample data set to draw from over the course of the observation time period (1980-2009): 145 peak yearly precipitation values, versus 29 peak yearly precipitation values (one for each year) for a single site. The compilation of a larger data set allows for improved accuracy in the prediction of return period storm events, particularly for rarer, more severe storm events.

The NOAA Atlas 14 90% Upper Confidence Limit scenario was chosen to provide a contrasting increased precipitation intensity scenario that relies on the use of historical observations. This scenario is not derived from climate models and does not represent theoretical or projected climate conditions. Instead, it represents the upper range of precipitation depths based on statistical analysis of historical observations. NOAA included these confidence intervals in its Atlas 14 to acknowledge the fact that there is uncertainty even in precipitation depth estimates derived from historical data. Typical engineering practice makes use of the average estimate values from NOAA Atlas 14 and does not consider the upper bound of the 90% confidence interval, which provides a greater precipitation depth than the point estimate for the average. Use of the upper bound estimate from NOAA Atlas 14 is a potential alternative proxy approach, consistent with engineers' traditional use of historic data (as compared to use of climate model projections), for considering changes in precipitation intensity and frequency under various future climate scenarios. Note that in this scenario, the probabilities of each return-period event are necessarily assumed to remain constant throughout the 21st century, something climate modeling may show is not necessarily the case in many areas.

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[34] A return period, or recurrence interval, is defined as the inverse of the probability of occurrence for a flood event in a given year; i.e., a 100-year storm would be a storm that has a 1% chance of occurring during any given year.
[35] USDOT, 2012

Table 5: 24-Hour Precipitation Depths Used in the Airport Boulevard Culvert Hydrologic Analysis

24-hour Storm Event Return Period	Observed (Model Baseline) 1980–2009 (inches)[36]	NOAA Average Baseline (inches)[37]	NOAA 90% Upper Conf. Limit[38]	"Wetter" Narrative			"Drier" Narrative		
				2010–2039 (inches)	2040–2069 (inches)	2070–2099 (inches)	2010–2039 (inches)	2040–2069 (inches)	2070–2099 (inches)
100-yr storm	13.5	14.9	18.9	21.0	20.4	22.3	12.6	14.2	13.4
50-yr storm	12.5	12.8	15.9	19.1	18.5	20.2	11.7	13.1	12.5
25-yr storm	10.1*	10.9	13.4	15.7*	15.2*	16.7*	9.3*	10.4*	9.9*
20-yr storm	9.5	Unavailable	Unavailable	14.8	14.4	15.8	8.8	9.9	9.4
10-yr storm	8.5	8.6	10.1	12.9	12.5	13.7	7.9	8.8	8.4
5-yr storm	7.1	7.1	8.3	10.5	10.3	11.1	6.6	7.3	7.0
2-yr storm	4.8	5.3	6.2	6.7	6.7	7.1	4.4	4.8	4.6

* Asterisks denote interpolated values[39]

[36] These values represent the baseline precipitation values from the climate models averaged across the five weather stations in the Mobile area. The source data is from Table 40 in USDOT, 2012.
[37] Based on NOAA Atlas 14 annual maximum time series (NOAA, 2013a)
[38] Based on NOAA Atlas 14 annual maximum time series (NOAA, 2013a)
[39] The 25-year storm is the design storm for the culvert facility but these values were not generated for the *Climate Variability in Change in Mobile, AL* report (USDOT, 2012) or the *Screening for Vulnerability* report (USDOT, 2014). As a workaround, the 25-year storm values were linearly interpolated from the 20-year and 50-year storm information available from these reports.

Table 6: Equivalent Present Day Return Periods for Projected Future Precipitation Values[40].

Existing NOAA 24-hour Storm Event Return Period	Wetter Narrative			Drier Narrative		
	2010–2039 (year storm)	2040–2069 (year storm)	2070–2099 (year storm)	2010–2039 (year storm)	2040–2069 (year storm)	2070–2099 (year storm)
100-yr storm	325	292	408	47	74	59
50-yr storm	227	201	281	36	55	46
25-yr storm	108	96	137	15	23	19
20-yr storm	87	78	111	12	19	16
10-yr storm	52	46	65	8	12	10
5-yr storm	24	22	29	4	6	5
2-yr storm	4	4	5	1	1	1

[40] Return periods for predicted future precipitation events are based upon current NOAA Atlas 14 return periods and precipitation totals versus the projected future precipitation totals as presented in Table 1.

The "Wetter" narrative uses the outputs of the climate models employed in earlier stages of the Gulf Coast Project to estimate how precipitation patterns might shift with future climate change. In keeping with the definition of this scenario developed in the *Screening for Vulnerability* report, the "Wetter" narrative represents the 95th percentile (mean+1.6 SD[41]) precipitation depths of all the climate model outputs under the range of climate scenarios considered. In other words, 95% of the model outputs across all the emission scenarios and climate models were less than the amounts shown for this scenario; thus, this narrative can be viewed as approaching a reasonable upper boundary for future climate changes. This narrative includes shifting probabilities of projections for three future time periods (2010-2039, 2040-2069, and 2070-2099), indicating the dynamic evolution of climate changes over this century. The values shown in Table 5 were derived by applying the projected changes for the Mobile station, the one nearest the study site, to the observed baseline scenario (described above) for each future time period. Note that the 25-year return period storm was not available directly from the model outputs provided. Instead, the 25-year storm was estimated by linearly interpolating between the model outputs provided for the 20-year and 50-year storms. Note from Table 5 that across all storm events and future time horizons, the projected precipitation amounts exceed the upper 90% confidence interval for the NOAA Atlas 14 data, thus demonstrating significance beyond the current uncertainty in the historical climate record.

The "Drier" narrative is calculated similarly to the "Wetter" narrative except that it represents the 5th percentile (mean-1.6 SD) of precipitation depths. The "Drier" narrative was not considered for testing in the hydraulic analysis of the culvert because precipitation values under this scenario are generally lower or nearly equal to historical values. Lower precipitation values mean lower peak flow rates at the facility; therefore, adaptation measures would not be needed to handle additional flows. The "Drier" narrative will, however, be a consideration in the economic analysis when developing recommendations for adapting this facility, as this scenario has the same likelihood of occurring as the "Wetter" narrative and this needs to be factored into decision-making.

Step 5 –Assess Performance of the Existing Facility

Assessing the performance of a culvert first requires detailed hydrologic and hydraulic modeling of the watershed in the vicinity of the facility to understand expected peak flows. These peak flows can then be used to evaluate the culvert's performance relative to its design standards.

Hydrologic Modeling

Peak flows through the culvert were modeled for various storm events and climate scenarios using the U.S. Department of Agriculture (USDA)-Natural Resources Conservation Service (NRCS) WinTR-20 program.[42] The WinTR-20 program utilizes NRCS hydrologic analysis

[41] Standard deviations
[42] USDA-NRCS, 2009

methodology to calculate runoff using the following inputs: drainage area, land use, soils, time of concentration,[43] and precipitation.

As previously discussed, the climate inputs for precipitation are limited to a minimum duration of 24-hours, due to limited output options from the available climate models. In consideration of this limitation, the selection of which culvert to study in this project focused on watersheds with a size and time of concentration that fell within a reasonable range of applicability for a 24-hour storm and the TR-20 model. The TR-20 development manual suggests an upper watershed limit of 25 square miles (64.7 square kilometers).[44] As for the lower limit, research results produced by Fennessey, Miller, and Hamlett (2001) suggest that the TR-20 computer model reasonably predicts discharges for urban watersheds down to a 1.2 acre (0.5 hectare) size (20 acre [8.1 hectare] minimum for wooded watersheds).[45] The model documentation for TR-20, does document that the main time increment of the models should be less than 30% of the watershed time of concentration. Since the TR-20 model has a computational limitation of six minutes in the main time increment, this results in a practical lower limit of 18 minutes for the time of concentration. To assess the time of concentration to the Airport Boulevard Culvert, a time of concentration path was developed (see Figure 9). The time of concentration along this path was calculated to be 1.5 hours.[46] Thus, the Montlimar Creek watershed at Airport Boulevard, with a drainage area of 2,137.6 acres / 3.3 square miles (865.1 hectares / 8.7 square kilometers) and a time of concentration calculated to be 1.5 hours, falls within the criterion for use of the TR-20 model with a 24-hour storm duration.

For watershed sizes smaller than the 1.2 acres (0.5 hectares) suggested by Miller and Fennessey's work, additional research will be necessary into precipitation intensity-duration-frequency (IDF) curves for sub-one hour durations with climate change. The climate change IDF curves will allow for the use of common small catchment hydrologic methods, including the Rational Method, to be utilized in these types of analysis.

Analysis of both existing and future land use conditions was necessary to evaluate current flows and predicted future flows at the Airport Boulevard culvert. Derivation of both existing and future land use began with consideration of the City's GIS-based zoning map. Existing land use conditions were developed from the zoning map and aerial photos.[47] Large, easily identifiable areas such as woodlands, golf courses, and lower density residential neighborhoods were located on the aerial photos and compared to the zoned land use. If the actual land use shown on the aerials differed from the zoning, corresponding edits were made to the zoning layer to create a map approximating

[43] Time of concentration is the time needed for water to flow from the most hydrologically remote point of the drainage area to the discharge point of the drainage area.
[44] USDA-SCS, 1992
[45] The study utilized the 24-hour precipitation curves from NOAA's Technical Paper 40.
[46] The time of concentration was calculated using the segmental method.
[47] Detailed current land use data was not made available at the time of initial analysis. This data was received at a later date and a comparison was made between the approach used here and this more detailed data. The comparison showed a less than 5% change in discharges with use of the more detailed data so it was decided to continue using the outputs from the initial approach.

existing land uses (see Figure 15). Future land use conditions for the drainage area were identified from build-out of the zoning layer (see Figure 16). If the tributary area is governed by stormwater management regulations that restrict flow to existing runoff conditions, the full-build-out runoff conditions may not apply, depending upon the effectiveness of the runoff control measures.

Approximately 70% of the drainage area consists of residential neighborhoods, 16% open pervious spaces (mostly comprised of golf courses), 2% woods, and 12% commercial and institutional land uses. In the future, based on build-out of the City's zoning, all of the open pervious spaces are changed to residential uses and there are no wooded areas. Thus, the future land use composition is slated to be 88% residential and 12% commercial and institutional.

The TR-20 model as developed by the NRCS, utilizes curve numbers[48] to represent the hydrologic properties of the watershed. Curve numbers themselves are determined based on the above discussed land uses along with the hydrologic soil group[49] types for the different soil types in the watershed. The hydrologic soils group data was obtained from the USDA-NRCS 2010 Soil Survey Geographic Database for Mobile, AL.[50] As a cautionary note, the analyst and designer must coordinate the datum planes used by the various sources of information to be certain that all elevations are referred to the same bench mark elevation.

[48] Curve numbers are a numeric approximation of a soil and land use combination's ability to produce overland runoff. The numbers range from a high end of 100, where all precipitation will be transferred to overland runoff, to a low end of near zero, which represents a condition where no overland runoff will be created.
[49] Hydrologic soil groups refer to the sorting of commonly occurring soil types into groups of A, B, C, or D dependent on the soil's infiltration and overland runoff performance.
[50] USDA-NRCS, 2010

Figure 15: Existing Land Use Conditions within the Airport Boulevard Culvert Drainage Area[51]

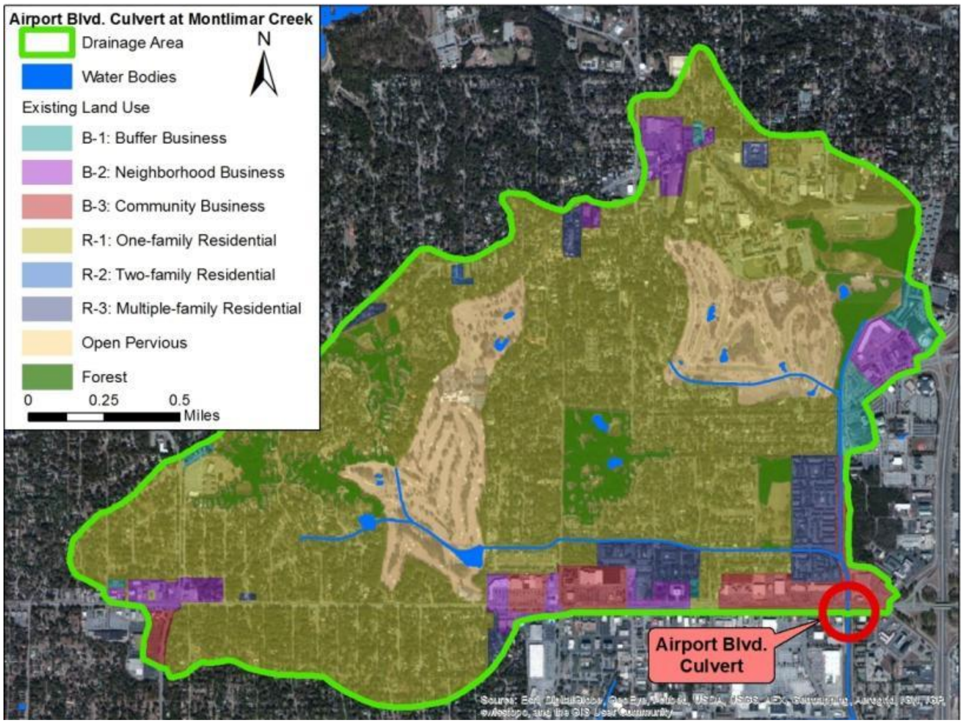
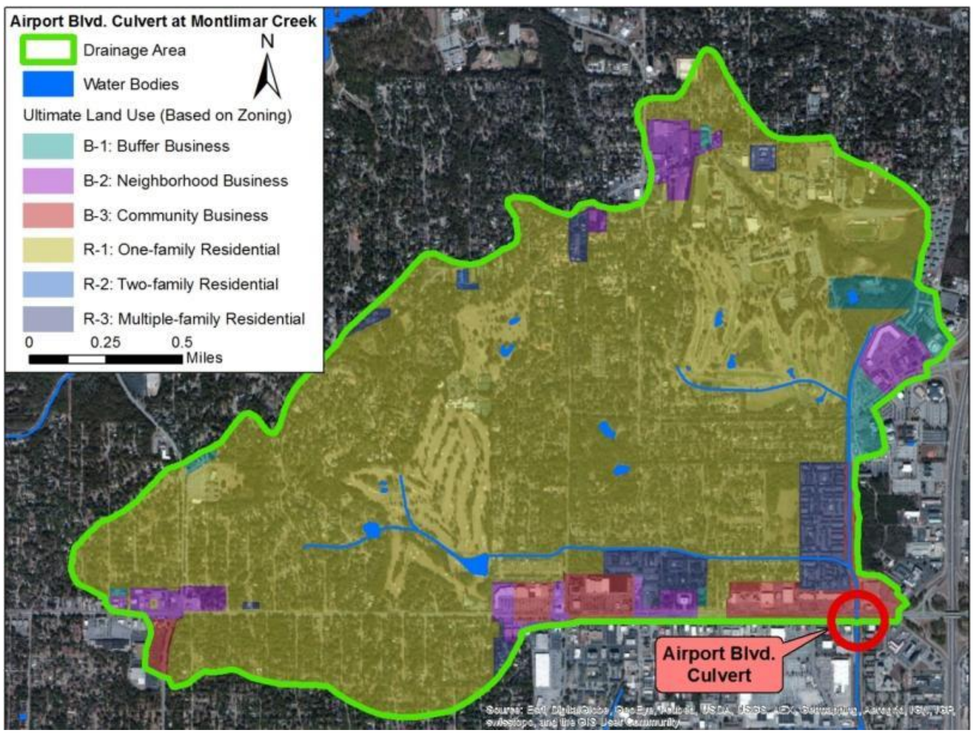


Figure 16: Future Land Use Conditions within the Airport Boulevard Culvert Drainage Area[52]



[51] Source of zoning data: City of Mobile GIS
[52] Source of zoning data: City of Mobile GIS

The hydrologic analysis also considered a range of temporal rainfall distributions for the evaluation of flows at the culvert to determine the appropriate values. Among the rainfall distributions considered were Soil Conservation Service (SCS) Type II and Type III rainfall type curves[53] as well as 24-hr temporal distributions developed by NOAA based on empirical rainfall data collected for the Southeastern Region of the United States. NOAA has developed detailed temporal distributions based on actual gauge data for a range of storm types depending on cumulative percentages of precipitation at various time steps. It was determined that the recently developed (as of June 2013) NOAA distributions for the Southeast region would be the most accurate representation of the storm distributions in the Mobile area.

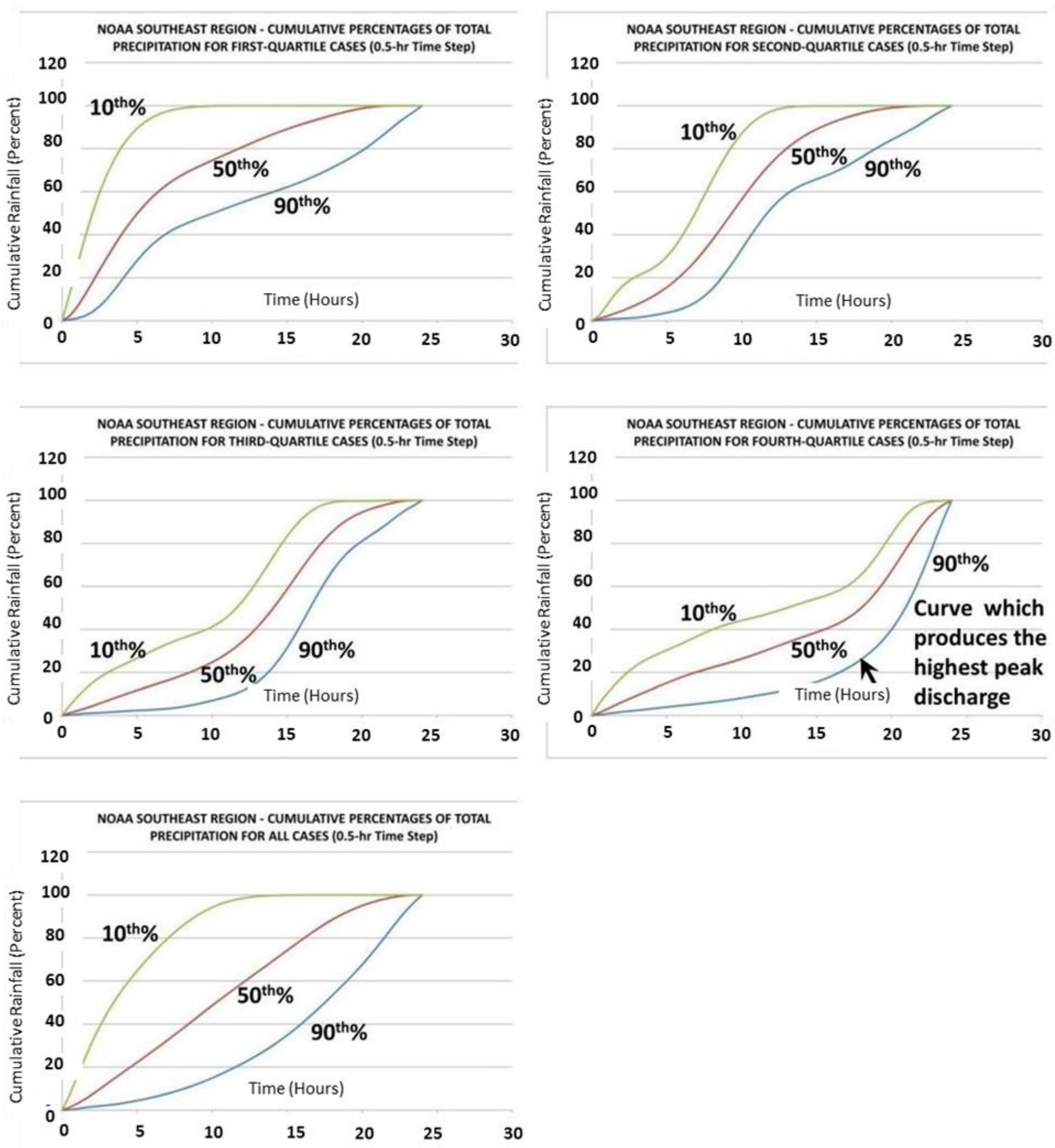
The NOAA temporal distributions used in this study were for the 24-hour duration storm. Separate temporal distributions were developed by NOAA for four precipitation cases defined by the time during the storm (first, second, third, and fourth quarters) in which the greatest percentage of the total precipitation occurred. For example, for 24-hour duration storms, the first quartile case refers to storm events where the largest amount of rain fell within the first six hours of the duration, the second quartile in the second six hours, and so on. Figure 17 shows graphical plots of the NOAA temporal distributions that were considered: the 10th to 90th percentile of first-, second-, third-, and fourth-quartile cases, as well as the overall case. For this analysis, the NOAA rainfall distribution that produced the greatest peak discharge (the 90th percentile of the fourth-quartile case) at the model output was chosen. This distribution was chosen because it validated well against regional regressions curves (discussed below), whereas more moderate distributions did not validate well. This distribution was also used for the future scenarios because it is the most extreme distribution that current climate shows is possible in the region.

Peak flows through the Airport Boulevard culvert were determined for the two-, five-, 10-, 20-, 25-, 50-, and 100-yr storm events under various climate scenarios. The Observed precipitation depths were run utilizing both existing and future land use conditions, while the NOAA 90% Upper Confidence Limit, “Wetter” narrative, and “Drier” narrative scenarios utilized future land use conditions.

A comparison was performed between the existing condition TR-20 model discharges and the regional regression estimates for urban streams in Alabama,[54] an alternative approach to estimating flows developed by USGS, to validate the model conditions. These regional regression curves are applicable to urban areas statewide with drainage areas between one and 43 square miles. An exception includes regions with impervious chalk and marl which does not pertain to the Mobile area. The use of these equations is therefore recommended for the Mobile area.

[53] USDA-SCS, 1986
[54] USGS, 2007

Figure 17: NOAA Atlas 14 Temporal Rainfall Distributions for the Southeast Region[55]



55 NOAA, 2013b

The two-year discharge fell within the regression estimate and the lower standard error of prediction, while the higher storm event discharges fell within the regression estimate and the upper standard error of prediction. The regression equations were utilized as a basis for calibration / validation of the TR-20 model. Since the regression equations are empirically derived and regionally specific, they provide a reasonable basis for calibration of the theoretical model. In the theoretical model, the resultant discharges can be highly influenced by development factors such as channel 'n' values,[56] sheet flow lengths, and precipitation temporal distributions that can be somewhat arbitrarily chosen. For a basis of comparison, the first cut uncalibrated existing condition TR-20 model produced a 100-year storm in excess of 10,000 cubic feet per second (283.2 cubic meters per second), while once calibrated against the regional regression, this value dropped to 4,300 cubic feet per second (121.8 cubic meters per second).

Table 7 shows the existing condition TR-20 model peak discharges compared to the USGS regional regression peak discharge estimates and the discharges with lower and upper standard error. This table also shows a comparison to the peak discharges presented in the FEMA flood insurance study for Mobile County[57] for a location on Montlimar Creek just downstream of Airport Boulevard. The FEMA discharges were calculated based on regression equations presented in USGS (1985) and USGS (1974). These discharges are provided for a point of comparison and were not used for calibration because the methods predate the USGS regression equations (2003) described above. Table 8 shows the final model outputs comparing current peak flows to projected future peak flows.

[56] Manning's 'n' value is a frictional resistance factor that approximates the effects of various channel and floodplain conditions on the conveyance rate of flows. An increase in this n value will cause a decrease in the velocity of water flowing across a surface.
[57] FEMA, 2010d

**Table 7: TR-20 Peak Flows Compared to
Regional Regression Peak Flow Estimates at the Airport Boulevard Culvert**

24-hour Storm Event Return Period	TR-20 Observed 1980–2009 w/ Current LU (cfs)	FEMA Flood Insurance Study Estimate (cfs) 2010	Regional Regression Estimate (cfs)	Regional Regression Estimate at Lower Standard Error (cfs)	Regional Regression Estimate at Upper Standard Error (cfs)
100-yr storm	4,361.8	3,260	3,601.7	2,521.2	4,682.2
50-yr storm	3,975.0	2,900	3,204.7	2,371.5	4,037.9
25-yr storm	3,045.1*	Unavailable	2,816.7	2,140.7	3,492.8
20-yr storm	2,813.4	Unavailable	Unavailable	Unavailable	Unavailable
10-yr storm	2,424.8	2,090	2,298.6	1,815.9	2,781.3
5-yr storm	1,889.0	Unavailable	1,857.2	1,467.2	2,247.2
2-yr storm	1,030.8	Unavailable	1,222.3	904.5	1,540.2

* Asterisks denote flows derived from interpolated precipitation values

Table 8: TR-20 Projected Peak Flows at the Airport Boulevard Culvert

24-hour Storm Event Return Period	Observed 1980–2009 w/ Current LU (cfs)	Observed 1980–2009 w/ Future LU (cfs)	NOAA 90% Upper Conf. Limit w/ Future LU (cfs)	"Wetter" Narrative w/ Future LU			"Drier" Narrative w/ Future LU		
				2010–2039 (cfs)	2040–2069 (cfs)	2070–2099 (cfs)	2010–2039 (cfs)	2040–2069 (cfs)	2070–2099 (cfs)
100-yr storm	4,361.8	4,484.6	6,553.8	7,347.1	7,122.3	7,844.8	4,137.9	4,754.8	4,445.1
50-yr storm	3,975.0	4,097.9	5,404.3	6,626.9	6,396.8	7,047.2	3,789.3	4,330.8	4,097.9
25-yr storm	3,045.1*	3,170.4*	4,445.1	5,328.1*	5,138.7*	5,712.9*	2,899.1*	3,325.2*	3,131.6*
20-yr storm	2,813.4	2,938.1	Unavailable	4,984.9	4,831.7	5,369.3	2,664.8	3,092.6	2,899.1
10-yr storm	2,424.8	2,549.4	3,170.4	4,253.3	4,097.9	4,560.1	2,316.6	2,664.8	2,510.5
5-yr storm	1,889.0	2,008.1	2,467.1	3,325.2	3,247.6	3,557.5	1,817.4	2,086.9	1,968.4
2-yr storm	1,030.8	1,134.6	1,665.5	1,855.0	1,855.0	2,008.1	987.7	1,134.6	1,061.0

* Asterisks denote flows derived from interpolated precipitation values

Hydraulic Modeling and Performance of the Existing Culvert

Hydraulic culvert analyses were conducted to evaluate the performance of the Airport Boulevard culvert under current and future peak flows. The Federal Highway Administration's (FHWA) HY-8 Version 7.2 program was used for these purposes.[58] The culvert computations considered the discharge; culvert size, slope, and material; roadway data; and channel characteristics. The roadway profile data was obtained from a 1961 design plan for the resurfacing of Airport Boulevard.

One uncertainty of the hydraulic model was that the slope of the culvert was not available: as a workaround, it was estimated to be approximately 1% based on two foot (0.6 meter) LIDAR contours. To be specific, the contour elevation was 26 feet (7.3 meters) at the upstream end of the culvert and 24 feet (6.7 meters) at the downstream end (see Figure 11): based on the vertical difference of two feet (0.6 meters) and the culvert length of 200 feet (61.0 meters), the slope was determined to be 1%. Barrel slope is critical to the hydraulic analysis of a culvert so, to ensure the 1% estimate was reasonable, a sensitivity analysis was performed by running the HY-8 program using 0.5% and 1.5% culvert slopes as well. This was achieved by holding the inlet elevation constant while raising and lowering the outlet elevation by one foot (0.3 meters) in the model. Results showed that the headwater elevations[59] were the same for all three slopes for discharges less than or equal to 8,000 cubic feet per second (226.5 cubic meters per second). Headwater elevations only began diverging for discharges beyond this value, which is higher than the discharge calculated for the 100-year "Wetter" narrative end-of-century storm event. Although a reasonable estimate of the culvert slope was used in this analysis, an actual project would require field surveys of the site to gather more accurate information.

Another uncertainty was the exact height of the roadway above the invert (bottom) of the culvert. The 1961 design plan provided the roadway elevation but not the culvert invert elevation. However, a close approximation of 26 feet (7.9 meters) for the invert elevation was obtained based on LIDAR contours, culvert size, slab thickness, and fill depth. The peak flows developed through the hydrologic analysis were analyzed with the culvert model to determine the headwater elevation at the culvert for various climate scenarios.

The tailwater conditions at the culvert location are also a potential point of uncertainty in the analysis. As discussed, the modeling procedure used HY-8 with tailwater values based upon normal depth and channel slope. The potential exists for downstream structures to influence this backwater elevation or for other floodplain limiting factors to control the tailwater elevation. The detailed evaluation of any structure for replacement should include a hydraulic study that fully considers downstream and upstream channel hydraulic characteristics and the impacts of other

[58] FHWA, 2012b
[59] Headwater elevation is the level of water immediately upstream of the inlet (upstream end) of a culvert or any other conduit.

factors into the hydraulics of a particular site, however, this level of detail was outside of the scope of this current study.

Culvert performance was assessed according to Alabama Department of Transportation (ALDOT) standards, which have been adopted by the City of Mobile. Per the 2008 *ALDOT Bridge Bureau Structures Design and Detail Manual,*[60] the 25-year flood is to be used for design of waterway crossings on secondary routes such as Airport Boulevard. A minimum of two feet (0.6 meters) of freeboard[61] is required above the design flood stage (relative to the edge of pavement) to keep the water surface an adequate depth below the pavement base. As such, the 25-year storm peak flows for various climate scenarios were analyzed through the hydraulic model and adaptation options were developed based on a two foot (0.6 meter) freeboard requirement.

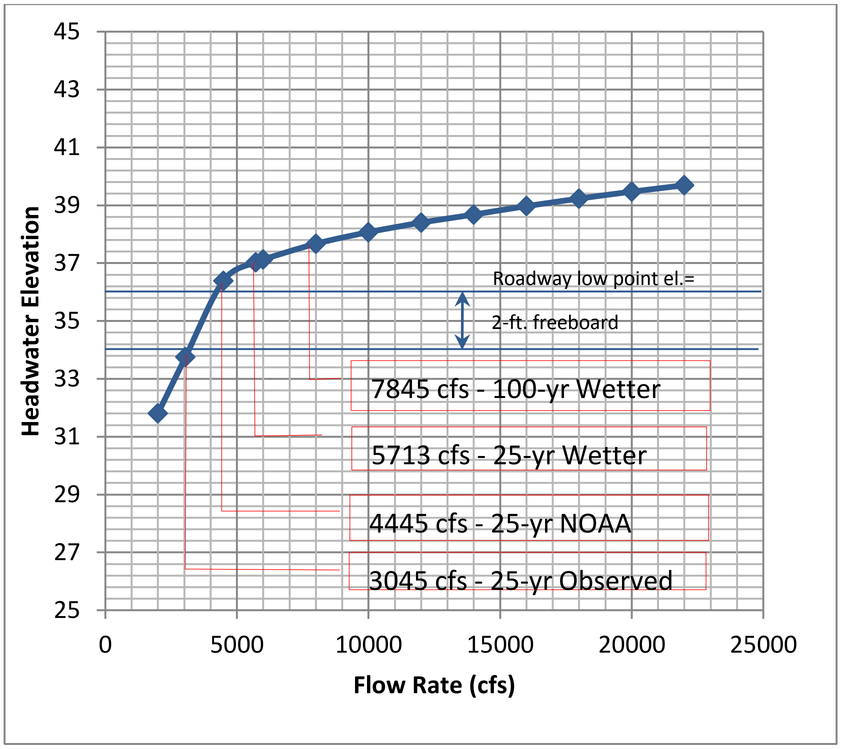
The 25-year storm peak flows for the Observed precipitation depths, the NOAA 90% Upper Confidence Limit, and “Wetter” narrative (end-of-century, 2070-2099 time period) scenarios were analyzed with the culvert model. The results, shown in Table 9, indicate that under existing precipitation and land use conditions, the current culvert meets ALDOT standards. The culvert also performs to standard with Observed precipitation amounts and future land use. However, the standard is not met for the other two scenarios tested. In the NOAA 90% Upper Confidence Limit scenario, the 25-yr flood stage overtops the low point of the roadway by 0.4 feet (0.1 meters) and in the “Wetter” narrative (end-of-century, 2070-2099 time period) the 25-yr flood stage overtops the low point of the roadway by one foot (0.3 meters). Figure 18 illustrates this information graphically.

Table 9: Airport Boulevard Culvert Modeled Performance under Various Climate Change Scenarios

	Observed 1980-2009 w/ Current LU	Observed 1980-2009 w/ Future LU	NOAA 90% Upper Confidence Limit w/ Future LU	“Wetter” Narrative End-of-century (2070–2099) w/ Future LU
Roadway Low Point Elevation (ft.)[62]	36.0	36.0	36.0	36.0
25-yr Flood Stage (ft.)	33.8	34.0	36.4	37.0

[60] ALDOT, 2008
[61] Freeboard is the excess height between a specific design water surface elevation and a given point of reference (e.g., roadway shoulder, low chord of a bridge, or crown of a culvert).
[62] The low point is located immediately to the east of the culvert location.

Figure 18: Existing Airport Boulevard Culvert Stage[63]-Discharge Curve

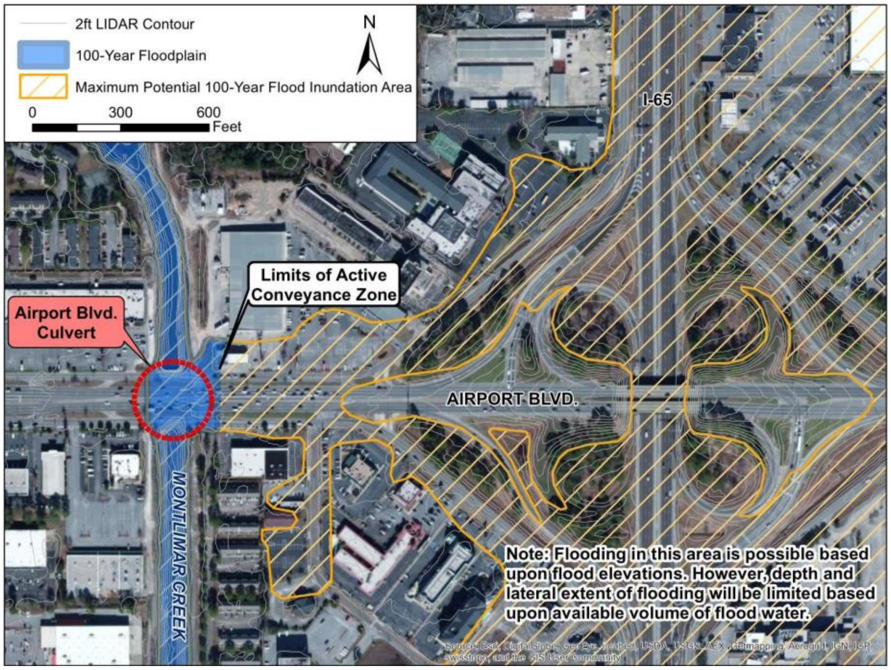


Flooding impacts on surrounding areas were evaluated based on the performance of the existing four cell box culvert. The degree of flooding was analyzed for the 100-year storm event under three scenarios: Observed conditions, the NOAA 90% Upper Confidence Limit, and "Wetter" narrative (end-of-century, 2070-2099 time period) scenarios. Figure 19, Figure 20, and Figure 21 show the potential inundation areas for each of the three scenarios. The areas shown are projected to be subject to inundation due to flood water overtopping the stream channel banks and the Airport Boulevard culvert. The lateral extent of the inundation areas are approximated based on the headwater elevation, noted in the figure footnotes, produced by the hydraulic model for each of the three scenarios. The area shaded in blue denotes the predicted floodplain limits adjacent to the flooding source. The potential floodplain is truncated at the limits of the active conveyance zone east of the culvert. The limits of the active conveyance zone as noted on the figures were delineated based upon a local ridgeline that is overtopped by the storm events. This limit is approximate and would require detailed 2-dimensional modeling to define in greater detail. The area beyond the limits of the active conveyance zone, shown by the hatching, represents the theoretical maximum area that may be vulnerable to inundation based upon the computed floodwater elevation relative to the ground topography. While the topographic contours show a clear flow path from the conveyance zone to this low-lying area, the depth and lateral extent of flooding in this area will be limited based upon the available volume of flood water from Montlimar Creek. A detailed two-dimensional unsteady flow analysis, beyond the

[63] Stage refers to water elevation.

scope of this case study, would be required to determine the volume of flood water that would flow eastward, the likely flow paths of that water, and, given these considerations, how much of the land to the east would actually become inundated.

Figure 19: Potential 100-year Inundation Area with Observed Precipitation Depths, Current Land Use, and the Existing Airport Boulevard Culvert[64]



[64] The headwater elevation of this scenario is 36.3 feet (11.1 meters). Note: Although the flood boundaries overlap nearby buildings on the map, the roofs are not overtopped by the 100-year flood event.

Figure 20: Potential 100-year Inundation Area under NOAA 90% Upper Confidence Limit Scenario with Future Land Use and Existing Airport Boulevard Culvert[65]

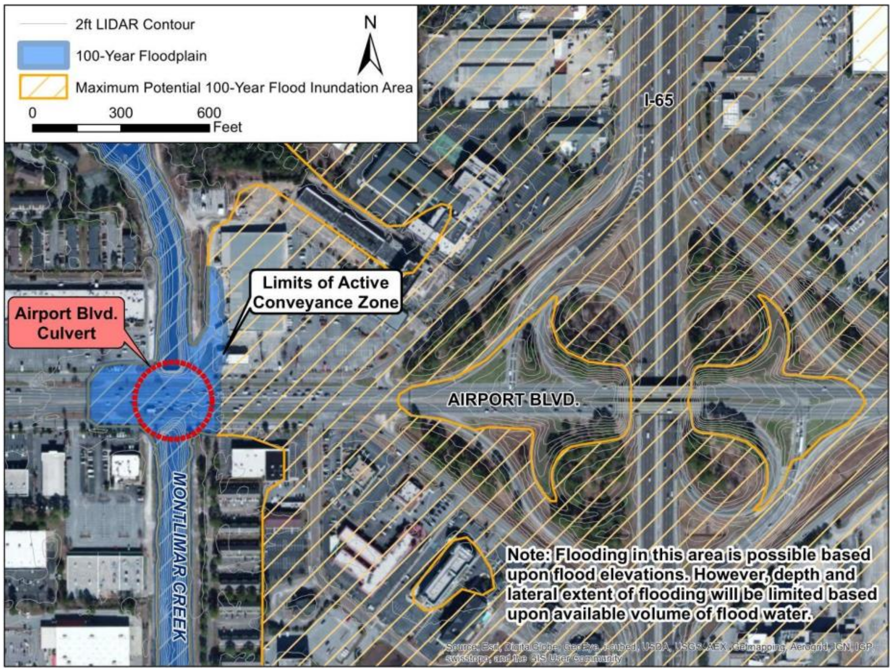
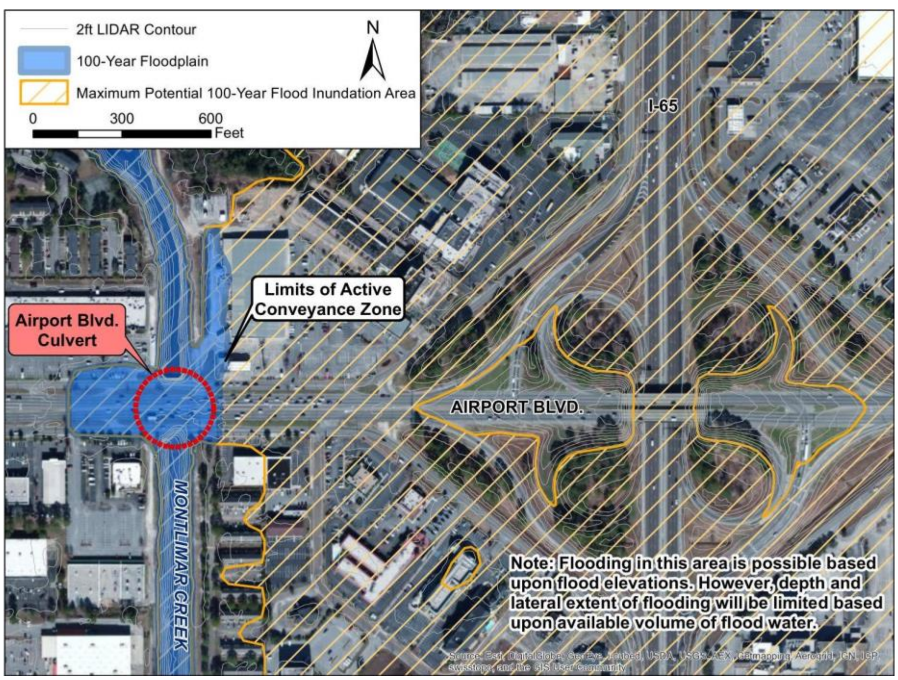


Figure 21: Potential 100-year Inundation Area under "Wetter" Narrative (End-of-Century, 2070–2099 Time Period) with Future Land Use and Existing Airport Boulevard Culvert[66]



[65] The headwater elevation of this scenario is 37.3 feet (11.4 meters). Note: Although the flood boundaries overlap nearby buildings on the map, the roofs are not overtopped by the 100-year flood event.
[66] The headwater elevation of this scenario is 37.6 feet (11.5 meters). Note: Although the flood boundaries overlap nearby buildings on the map, the roofs are not overtopped by the 100-year flood event.

Step 6 – Identify Adaptation Option(s)

The alternatives for adaption to meet design standards include expanding the existing crossing or controlling the runoff that reaches the crossing. The alternatives considered and their pros and cons are presented in Table 10. Expanding the capacity of the existing crossing is the most direct solution; however, it could have a negative effect downstream by increasing the flow rate, stage (water elevation), and downstream erosion because the existing culvert acts to attenuate the flow. The latter approach, controlling the runoff that reaches the crossing, would mitigate these affects and not require modification of the crossing. This approach, although not considered here due to this study's site-specific focus, is worthy of additional investigation. Such an approach would require a regional drainage plan with up-stream retention or detention facilities, potential condemnation of property, and possible zoning or development regulation changes.

To demonstrate how an adaptation analysis could proceed, this analysis focuses on the approach of increasing the capacity of the culvert crossing. In actual practice, the effects downstream mentioned above would be assessed to determine the impacts and the costs associated with them that the project would have to bear. As mentioned previously, the drainage system (upstream and downstream of the culvert) and its effect on culvert operations should be considered in an actual case. For example, widening the stream at the culvert entrance could result in sediment deposits because of lower flow velocity. This effect can require that inspection and maintenance be increased because the sediment accumulation would reduce culvert capacity. The idea of a bridge replacing the culvert was briefly considered but ruled out because the low allowable headwater height would not allow an adequate structure depth without major roadway reconstruction to raise its elevation.

Table 10: Adaptation Design Options Considered for the Airport Boulevard Culvert

Possible Alternatives	Description	Pros	Cons
Regional Drainage Area Management	• Perform drainage area analysis to determine best management procedures • Consider restrictions or constraints for future development • Acquire RW for and construct one or more detention / retention facilities to attenuate runoff AND volume to existing downstream capacity	• Reduces runoff rate and volume to existing values for selected design storm runoff at roadway crossing and farther downstream • No traffic delay on Airport Boulevard	• Large project undertaking compared to the culvert option • Possible zoning changes required that would restrict development (potential inverse condemnation) • Acquisition of large amounts of property required for facilities • Large capital and maintenance costs • Lengthy project development time
Bridge	• Replace culvert with a single-span bridge	• Increases crossing capacity • Increases hydraulic opening / decreases flow obstructions. • Provides increased protection to surrounding properties for existing and future runoff amounts	• Large project undertaking compared to the culvert option • Structure depth requires raising the roadway for a horizontal distance of about 600 feet (182.9 meters) on each side of the culvert • Long period of traffic disruption • Increases flow rate and volume downstream • Large capital and maintenance cost

Possible Alternatives	Description	Pros	Cons
Culvert Modification (Option One)	• Add one cell on each side of the existing crossing	• Increases crossing capacity • Headwater elevation meets criteria for the 25-year NOAA rainfall value • Provides increased protection to surrounding properties for existing and future runoff amounts • Keeps within the existing easement and right-of-way • Uses the existing facility (sustainable) • Smallest footprint • Lowest capital cost alternative • Shortest project development time	• Disrupts traffic • Increases flow rate and volume downstream • Increased potential for sediment aggradation / increased maintenance needs.
Culvert Modification (Option Two)	• Replace the existing crossing with largest crossing that will fit within the space available	• Increases crossing capacity • Keeps the 100-year "Wetter" narrative precipitation runoff at the edge of pavement—not overtopping the roadway • Provides increased protection benefit to surrounding properties for existing and future runoff amounts over Option One	• Longer period of traffic disruption than Option One • Increases flow rate and volume downstream more than Option One • Higher capital cost than Option One • Increased potential for sediment aggradation / increased maintenance needs

Other factors that may complicate the development of adaptation options are any pipes that are located near the culvert or discharge into it. At this study site, a 48 inch (121.9 centimeter) bituminous coated concrete drainage pipe and a 15 inch (38.1 centimeter) reinforced concrete drainage pipe discharge into the westernmost barrel of the culvert. A pressured water line may also be crossing the culvert. Although it was not evaluated in this study, adaptation solutions must take into account potential impacts to nearby pipes and utilities which may need to be relocated, modified, or temporarily taken offline to carry out the project.

With these limitations in mind, the selection of alternatives is based on the following guidelines:

- Keeping the solution within the existing easement and right-of-way to the extent practical
- Making use of the existing facilities to the extent practical
- Keeping the footprint of the alternative as small as practical

Plotting stage-discharge curves developed from the hydrologic and hydraulic models of various types of crossings helps visualize the actual performance against the design standard. For this project, we considered eight combinations of culvert cell sizes. Based on the guidelines above, we selected the narrowest combination with the lowest rise. Two levels of adaptation options are suggested for the Airport Boulevard culvert: one that is optimized for the NOAA 90% Upper Confidence Limit and a second that is optimized for the “Wetter” narrative (end-of-century, 2070-2099 time period). The goal of both adaptation options is to achieve the minimum two feet (0.6 meters) of freeboard from the 25-year flood stage to the edge of pavement at the low point of the roadway. Both adaptation options involve the addition of box culvert cells or expansion / replacement of existing box culverts in order to increase flow capacity.

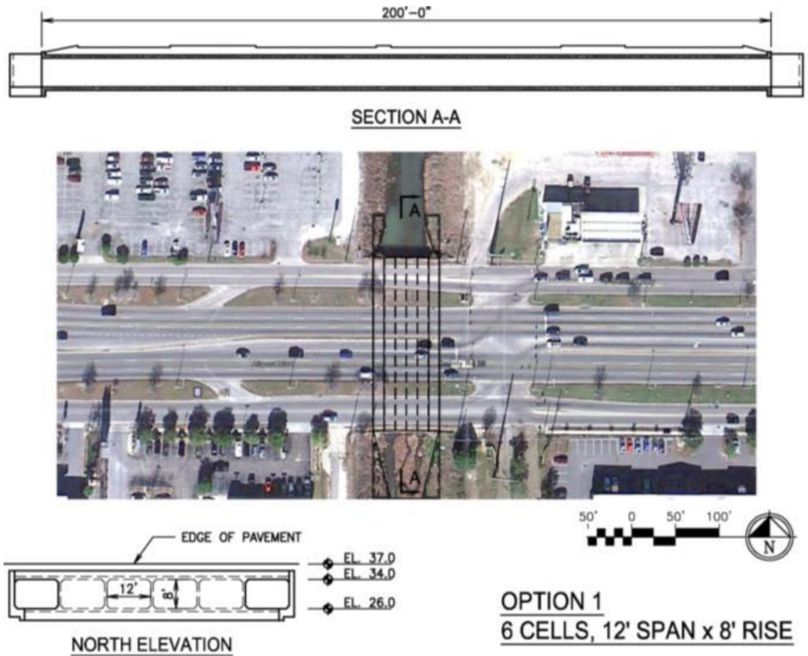
Option One

Option One meets the criteria for the Observed precipitation depths and the NOAA 90% Upper Confidence Limit scenario. It consists of adding one additional 12 foot (3.7 meter) span by eight foot (2.4 meter) rise box culvert on each side of the existing four cells of the same size. Transitioning the channel width to the wider entrance and from the wider exit will require concrete training walls. Figure 22 shows this configuration.

The work, estimated to cost $1.7 million, includes:

- Removing the existing wing walls on both sides (to stay within the right-of-way and minimize channel alignment changes)
- Excavating and installing one box culvert on each side of the existing crossing
- New headwall extensions
- New training walls
- Utility relocation (existing utilities appear to cross and run parallel and close to the existing crossing). The estimate includes a “plug number” for these relocations because they have not been identified and located in the field.

Figure 22: Plan for Adaptation Option One of the Airport Boulevard Culvert over Montlimar Creek



Option Two

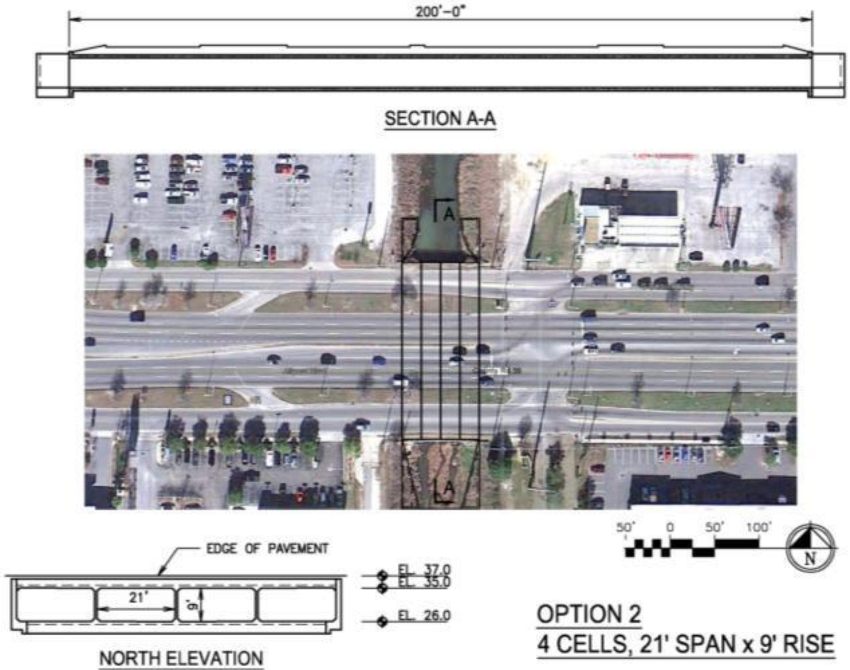
Option One meets the criteria for the Observed precipitation depths and the NOAA 90% Upper Confidence Limit scenario but not the higher flow of the 25-year “Wetter” narrative (end-of-century, 2070-2099 time period). Thus, the project team developed Option Two to address the possibility of the 25-year “Wetter” narrative occurring. This option is within practical limits of the aforementioned criteria, is about 2.4 inches (6.1 centimeters) above the design headwater elevation, and has the benefit of keeping the 100-year “Wetter” narrative runoff to below the elevation of the low point in the roadway so that it does not overtop the road. Option Two consists of removing the existing crossing and installing four cells, each with a 21 foot (6.4 meter) span by nine foot (2.7 meter) rise. Pre-cast spans this wide are available and alternatively could be cast in place which would increase construction duration. The spans would be designed to meet highway loadings as described in the AASHTO standards. The option of raising the culvert height by one foot (0.3 meters) was considered because the required expansion of culvert width is limited by channel width. Transitioning the channel width to the wider entrance and from the wider exit will require concrete training walls. Figure 23 shows this configuration. Widening the channel may increase sediment deposition which would require inspection and sediment removal. Sediment transport was not included in this analysis. An estimate of annual maintenance costs was carried forward in the economic analysis.

The work, estimated to cost $2.5 million, includes:

- Removing the existing culvert and wing walls
- Excavating and installing a culvert with four 21 foot (6.4 meter) by nine foot (2.7 meter) cells

- New headwall extensions
- New training walls
- Utility relocation (existing utilities appear to cross and run parallel and close to the existing crossing). The estimate includes a "plug number" for these relocations because they have not been identified and located in the field.

Figure 23: Plan for Adaptation Option Two of the Airport Boulevard Culvert over Montlimar Creek



Step 7 –Assess Performance of the Adaptation Option(s)

The degree of flooding was analyzed for each adaptive design option using the 100-yr storm event with the Observed precipitation depths and the NOAA 90% Upper Confidence Limit and "Wetter" narrative (end-of-century, 2070-2099 time period) scenarios. As shown on the following curves in Figure 24 and Figure 25, the ALDOT standard is met in Option One for the Observed precipitation depths and the NOAA 90% Upper Confidence Limit scenario and nearly met (within the margin of error) by Option Two for all three cases. Also, note that the 100-year "Wetter" narrative flood elevation exceeds the roadway low point elevation of Option One by one foot (0.3 meters) while it stays approximately at the Airport Boulevard road surface low point elevation under Option Two.

As an additional consideration, potential downstream impacts should be taken into account in the evaluation of adaptation alternatives. For example, the adaptation options propose a wider channel width at the culvert which would need to transition back to the existing channel width downstream. Although it was not conducted for this study, one may evaluate the impacts of the channel width transition on downstream water surface elevations. A U.S. Army Corp of

Engineers Hydrologic Engineering Center River Analysis System (HEC-RAS) model can be developed to determine if the discharges from the larger magnitude storms may overtop the existing 40 foot (12.2 meter) wide channel downstream. The results of this analysis may show that certain adaptation options are more suitable than others.

Figure 24: Airport Boulevard Culvert Adaptation Option One Stage-Discharge Curve

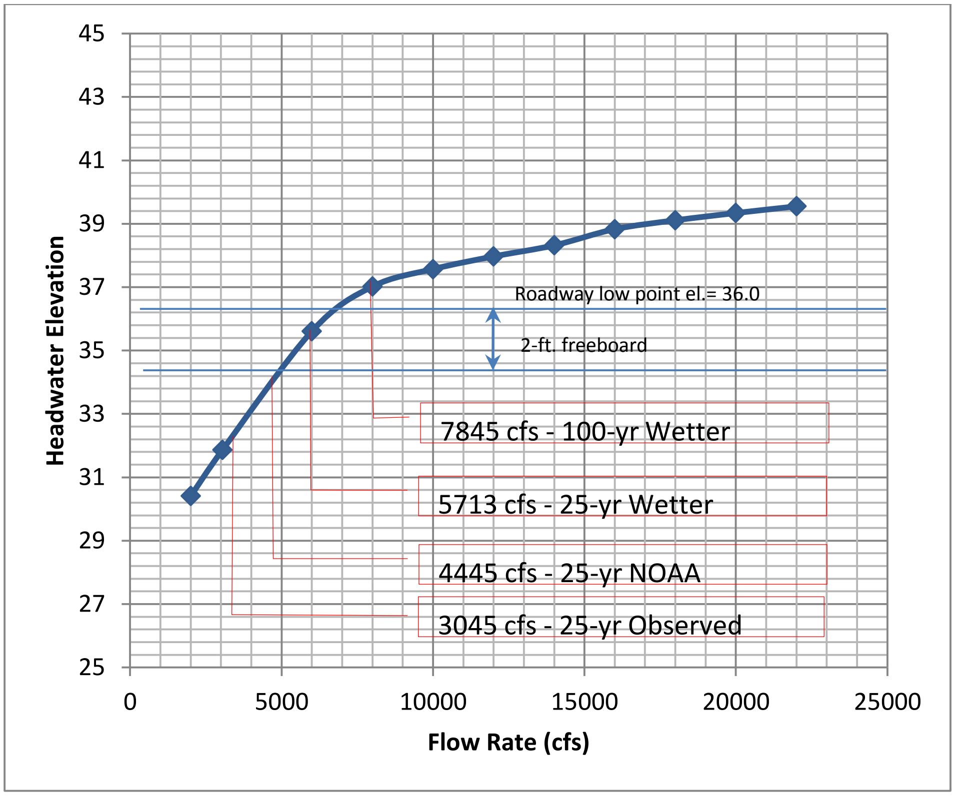
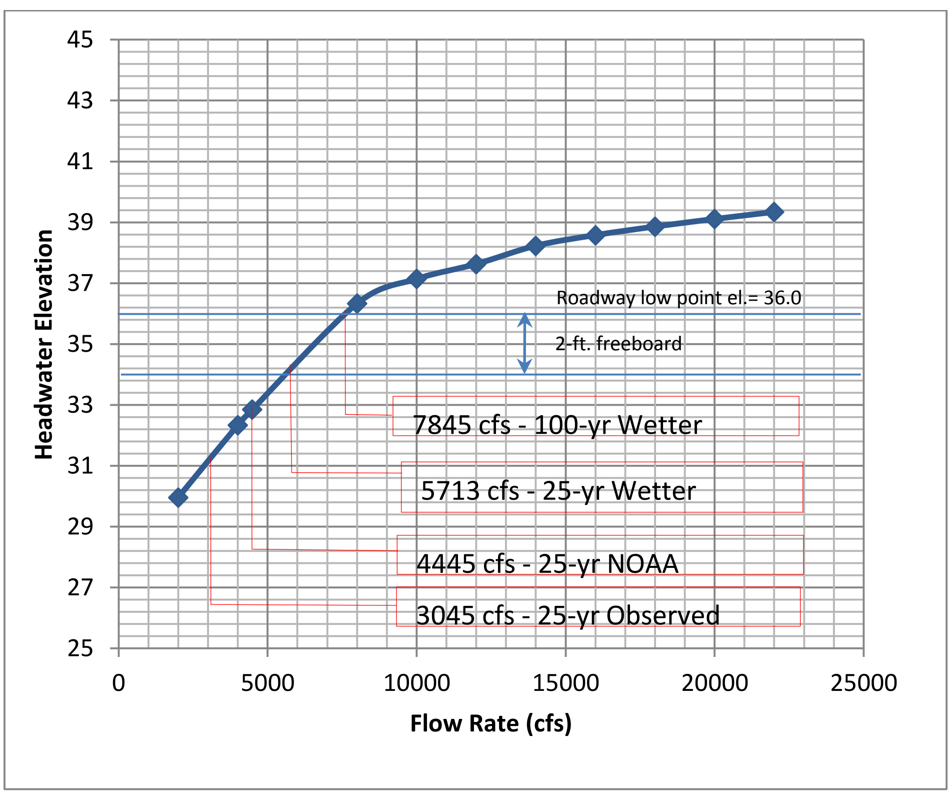


Figure 25: Airport Boulevard Culvert Option Two Stage Discharge Curve



Step 8 –Conduct an Economic Analysis

This step involves an economic analysis of the various adaptation options to aid in the selection of the most cost-effective adaptation measure. The section begins with an overview of the approach taken and the analysis results before proceeding to discuss the entire process in full detail.

Overview

A key feature of climate change is uncertainty. In particular, the timing, severity and frequency of future weather and climate events are not known with any degree of precision. Economic analyses provide valuable information for decision-making where there is a high degree of uncertainty about the future. The benefit-cost analysis model developed for this study uses a probabilistic framework to account for this uncertainty by testing thousands of different combinations of future storm events, each with its own peak discharge flow.

As in any economic study, the level of detail and rigor of the economic analysis should be commensurate to the scale of the project being studied. For climate change analyses, the consequences of that facility's failure should be considered as well when determining the level of effort to expend. The analytical framework used in this study employs Monte Carlo analysis[67] to simulate thousands of different combinations of storm events under five climate / land-use scenarios and then estimates the resultant flooding costs over a 30-year appraisal period. This type of economic analysis may involve a larger level of effort than is required for most culverts, especially smaller culverts, however, it is used in this case study as an illustration of an approach to benefit-cost analysis that can be used to aid in decision-making for a variety of facility types, including those with higher values.

In this analysis, three alternative courses of action are compared:

- **Base Case**: "Do Nothing", leaving the existing culvert as is
- **Option One**: Provision of additional cells adjacent to the existing culvert

[67] Monte Carlo analysis is a computerized mathematical technique widely used in industry to analyze problems where there is inherent uncertainty in predicting future events. It particularly useful when there are large numbers of input variables and hence many degrees of freedom affecting the outcome. Its origins lie with the Manhattan Project: 'Monte Carlo' was a code name coined by one of the lead scientists whose uncle had a penchant for gambling at the casinos in Monaco. Since the end of the war, the technique has been adopted within financial, insurance, oil and gas, and many other industries to assist with their decision-making processes. Despite the connotation of the name, the technique is not completely random; inputs to the models are sampled from pre-determined probability distributions. These inputs are substituted into the model, the output calculated, and the result saved. The process is then repeated using a different set of sampled inputs. Depending upon the number of uncertain variables, a simulation could involve thousands or tens of thousands of recalculations before it is complete. At the end of the simulation, the outputs are collated and analyzed and a probability distribution is generated showing the range of all possible outcomes and the likelihood that they will occur. Monte Carlo simulation is frequently used for applications such as analyzing extreme weather and natural disaster events. Whilst this may appear to be an advanced technique, software packages such as *@Risk* and *Crystal Ball* are widely available as 'plug-ins' to Microsoft Excel at prices which would be affordable to most local and state government agencies. The *@Risk* software tool has been used for this study.

- **Option Two:** Replacing the existing culvert with a new facility that has larger capacity cells. Option Two offers a higher level of protection to Option One as it can accommodate higher discharge flows through the culvert, but is around $800,000 more expensive.

The Monte Carlo approach employed in this case study calculates economic benefits (in terms of avoided flood costs) for Option One and Two relative to the base case using the precipitation depths and resulting flow of the Observed scenario. Discounted cash-flow analysis is then used to rank and select a preferred adaptation option for the culvert upgrade, by comparing the discounted economic benefits against the upfront capital cost of the two upgrade options. After all iterations of the Monte Carlo simulation have run, a probabilistic distribution of results is then generated.

In this analysis, the economic benefits comprise avoided traffic congestion costs resulting from Airport Boulevard and the adjacent I-65 freeway being closed due to flooding.[68] A second analysis was conducted where avoided flood damages to nearby buildings (from flooding attributable to the culvert) were included as benefits as well. These results are presented separately as a sensitivity test given the uncertainties of how much flooding will occur to the east of the site pending further detailed hydrological analysis. It should be noted that anticipated clean-up and repair costs from flood damage to the facility should also be considered in benefit-cost analyses, however, in this case it was assumed that such costs would be minimal given the relatively slow velocities of water at the site.

A summary of the results of the analysis (not inclusive of property damage costs) is shown in Table 11. This table shows the 90th percentile results (out of 1,000 iterations of the Monte Carlo simulation) along with the average economic benefits, Net Present Value (NPV), and Benefit-Cost Ratio (BCR) of all five climate scenarios. The range of benefits, NPVs, and BCRs across the five scenarios is provided in parentheses. The results show that whilst both options are economically viable, Option One is preferred as it has the higher BCR of the two options at 3.5.

Approach

There is considerable uncertainty when appraising climate change initiatives in that the timing, severity, and frequency of future weather and climate events are not known with any degree of precision. If scarce public funds are invested in infrastructure adapting to climate change and no weather events occur in the near-term, then the infrastructure adaptations will, in essence, "lie idle" and the funds could have been used for alternate more immediate needs. Conversely, if infrastructure investments are delayed but a number of significant flooding events occur, then widespread economic damage that could otherwise have been avoided will result.

[68] Although there are uncertainties in the location and amount of flooding expected to the east of the Airport Boulevard culvert due to spill over attributable to the culvert, a conservative assumption was made that I-65 would be affected by the resulting flooding. An initial evaluation of likely flow paths showed that much of the overflowing water from Montlimar Creek would likely pass through the drainage infrastructure of the I-65 interchange with Airport Blvd. and this infrastructure is known to have capacity challenges. However, further detailed hydrological analysis, beyond the scope of this case study, would be required to verify this assumption.

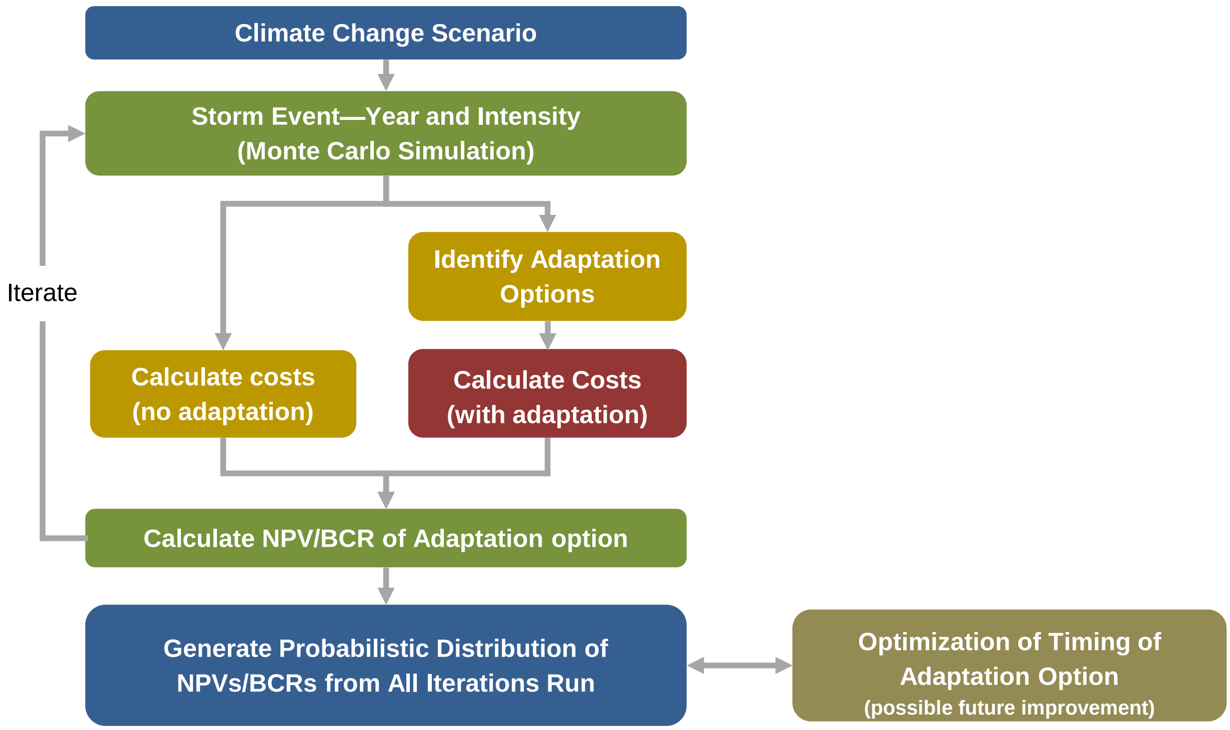
Table 11: Airport Boulevard Culvert Benefit-Cost Analysis Results: 90th Percentile, Average of All Climate Scenarios, No Property Damage Costs Included[69]

	Option One	Option Two
Scope of improvement	Six cell, 12' span x 8' rise culvert	4 cell, 21' span x 9' rise culvert
Present value of costs	\$1.7m	\$2.5m
Present value of benefits [avoided traffic congestion costs]	\$6.0m *(\$3m to \$12.7m)*	\$6.5m *(\$3.2m to \$14.4m)*
Net present value	\$4.3m *(\$1.3m to \$11.0m)*	\$4m *(\$0.7m to \$11.9m)*
Benefit-cost ratio	3.5 *(1.7 to 7.3)*	2.6 *(1.3 to 5.8)*

To address this uncertainty, a probabilistic benefit-cost analysis model has been developed which uses Monte Carlo analysis to simulate thousands of different combinations of flooding events of differing intensities under five climate / land-use scenarios over a 30-year period (a reasonable remaining design-life for the existing culvert and Option1 since it involves use of the existing culvert as well). A probability distribution is then generated from the results of all event combinations simulated. Figure 26 shows the model framework developed for the benefit-cost analysis, which is described in further detail in later sections.

[69] Costs and benefits presented in \$2013, discounted over 30 years at 7%. The ranges of possible values are shown in parentheses where pertinent.

Figure 26: Probabilistic Benefit-Cost Analysis Framework



The main model engine undertakes an analysis of each climate change scenario and each adaptation option using the following process. For each year in the 30-year appraisal period, the model:

- **Randomly generates a peak flow through the culvert:** This is done by using Monte Carlo simulation to sample from the appropriate probability function for that time period. A probability distribution for each climate change scenario has been developed using the annual return periods for each 24-hour storm event given in Table 8. The resulting function is continuous and so can not only generate 100-year, 50-year, and 25-year events according to the modeled data but intermediate values such as 23-year or 49-year events as well. Extreme events which have not yet been observed are also included: for example, if observations of storm events only go back 400 years, an estimate of a 1,000-year storm can be generated from the fitted curve as well.

  An example of a fitted distribution is shown in Figure 27. This shows the cumulative probability distribution for the input data of the "Wetter" narrative (2010-2039 time period) from Table 8 against the fitted distribution (estimated as an Extreme Value distribution[70]) generated by the @Risk software used for the Monte Carlo analysis.

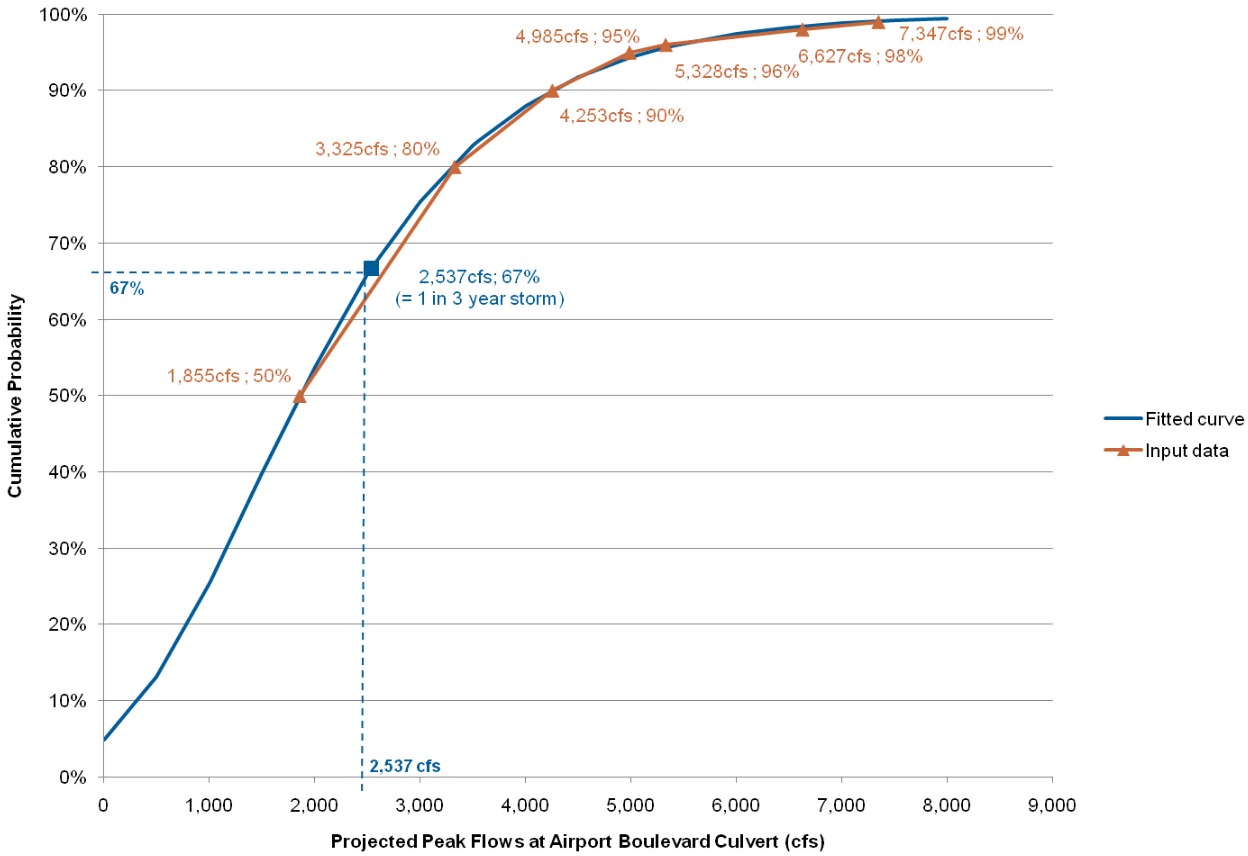
[70] In hydrology, the Extreme Value (or Gumbel) probability distribution is commonly used to describe the distribution of extreme event variables such as peak discharges. The distribution is skewed to the right to focus on maximum events, and can be used to estimate extreme events such as a 100,000-year event.

In Figure 27, the cumulative probability on the y-axis is a function of the annual return period: for a 100-year storm (peak flow 7,347 cubic feet per second [208 cubic meters per second]), in any given year, only one percent of storm events will generate a peak flow greater than this. For a five-year storm, 80% of annual storm events will generate peak flows lower than 3,325 cubic feet per second (94.2 cubic meters per second). The fitted curve also shows an example of how the peak flow through the creek can be estimated for a storm event which has not been provided in the input data: a three-year storm event (with a 66.6% probability of not being exceeded in any given year) is estimated to generate a peak flow through the creek of 2,537 cubic feet per second (71.8 cubic meters per second).

- **Determines the resultant flood elevation due to the generated flow**: This is done using the culvert stage-discharge curves provided in Figure 18, Figure 24, and Figure 25 for the existing culvert and each improvement option. Continuous functions which estimate the flood elevation for any flow rate through the culvert have been estimated using regression.
- **Determines the cost of the flood if the elevation overtops the low point of the roadway (36 feet [11 meters]):** This analysis considers the traffic disruption costs that would occur if Airport Boulevard and I-65 were closed for an assumed 24 hours[71] due to a flooding event. Using Vehicle Miles Traveled (VMT) and Vehicle Hours Traveled (VHT) outputs from the Mobile Area Transportation Study (MATS) regional travel demand model, the analysis calculates the additional vehicle hours traveled and vehicle miles traveled which would occur throughout the region if both Airport Boulevard and I-65 were closed. From this, FEMA parameters (for example, the value of travel time savings) are used to estimate the additional travel costs incurred (an avoidable economic cost). A sensitivity test has been undertaken (discussed later) whereby property damage costs are also included.
- **Calculates the flood cost savings for each of the improvement options**: Costs are relative to the flood costs of the base case (no build) with Observed precipitation depths.

[71] 24-hours is used to allow for water to recede, inspections for damage, and clean-up of the roadway.

Figure 27: Comparison of Input Data and Fitted Curve for the "Wetter" Narrative (2010-2039 Time Period)[72]



Once the impacts and costs are generated for each storm event in each year of the appraisal the model:

- Discounts the benefits (in terms of flood cost savings) for each year of appraisal back to present year values using the Office of Management and Budget rate of 7%.
- Discounts the capital costs of the improvement options (if incurred in future years)[73]
- Sums all discounted costs and savings across the entire 30-year appraisal period
- Calculates the NPV and BCR of each improvement option and records the result

The process is then repeated thousands of times, each time using a different set of randomly generated flood events. After all simulations have completed, a probability distribution showing the results of all iterations is generated.

[72] Note that the root mean square error of the fitted Extreme Value distribution versus the input data is less than 0.005 indicating a good fit. Also, whilst the fitted distribution curve does not pass through the origin, controls have been implemented in the simulation model to ensure that negative peak flows are not generated in the Monte Carlo simulation process.

[73] In this case study, the capital costs are assumed to be incurred in year zero, and therefore are not discounted. A possible improvement option would be to optimize the year of implementation of the improvement option. In this situation, the capital costs would also be discounted.

The Cost of Doing Nothing

There are likely to be substantial costs incurred if no adaptation actions are taken to address flooding at the culvert. This is because when water overtops the culvert at 36 feet (11 meters), travel will be impeded both on Airport Boulevard and I-65 (see Figure 19, Figure 20, and Figure 21). These costs can be expected to rise over time as traffic volumes increase. A sampling of the incurred costs due to traffic disruption owing to Airport Boulevard and I-65 being closed at two different time periods, 2007 and 2035, is shown in Table 12. These two time periods are the only years for which volumes are available from the MATS regional travel demand model. Disruption costs for intermediate years between 2007 and 2035 and years beyond 2035 have been calculated by using linear interpolation and extrapolation for use during the analysis.

Table 12: Traffic Disruption Costs (Per Flooding Event) With No Adaptation Actions[74]

	2007	2035
Flood elevation <36 feet	$0	$0
Flood elevation >=36 feet	$2.6m	$6.1m

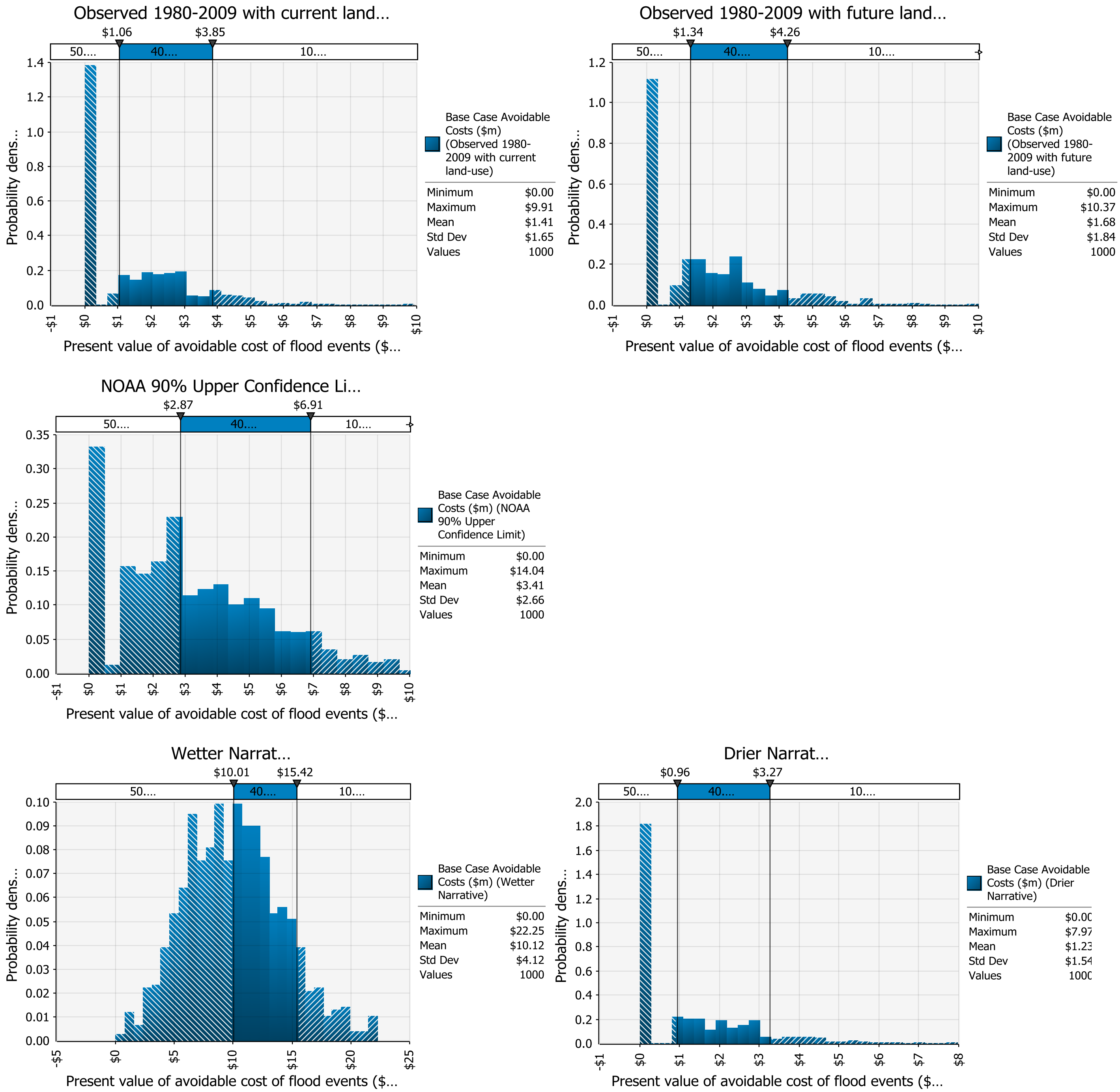
Figure 28 shows the probability distributions of cumulative traffic disruption costs[75] (discounted to present values) over 30 years for each of the climate change scenarios in the Base Case situation (no adaptation actions taken). The vertical delimiters show the 50th (median) and 90th percentile results. Thus, for the Observed conditions with current land use, over 50% of model runs had costs greater than $1.1 million and 10% had costs greater than $3.9 million. Under the "Wetter" narrative, the median disruption cost over the 30-year period was $10 million and the 90th percentile cost was $15.4 million. The maximum cost modeled was $22.3 million.

Understanding the Economic Analysis of the Adaptation Options

Economic viability of each option is assessed through its Benefit-Cost Ratio (BCR) and Net Present Value (NPV). The NPV (the discounted value of benefits, less the capital cost) gives an indication of the **magnitude** of net benefit to society. Positive NPVs indicate the project is desirable to society as a whole.

[74] Values are indexed to $2013 using the U.S. Consumer Price Index for all Urban Consumers (CPI-U)
[75] The incremental regional VMT and VHT due to shutting down Airport Blvd. and I-65 were converted into economic costs (additional travel time and mileage) using $2013 inflation adjusted (CPI-U based) FEMA unit values (FEMA, 2009). The value used for vehicle delay detour time was $41.20 per VHT and for additional vehicle mileage was $0.60 per VMT.

Figure 28: Probability Distributions of Avoidable Costs without Adaptation and Inclusion of Property Damage Costs[76]



[76] These histograms show the range of possible outcomes and their relative likelihood of occurrence. The x-axis represents the avoidable cost and the y-axis shows the probability density. In simple terms, the height of each bar gives an indication of how likely that result is to occur. The vertical delimiters show the 50th and 90th percentile results; the values at which 50% and 90% of results fall under.

The BCR (the discounted benefits divided by the capital cost) is a measure of **efficiency** or value for money, and is of principal consideration when Government is considering spending scarce funds. BCRs greater than one indicate the benefits outweigh the costs and hence a project is good value for money.

In analyzing the results of an economic analysis, the NPV and BCR should be considered together when comparing two options, particularly if the BCRs are similar in magnitude. In general, if an option has a higher BCR than its alternative, then it will also have a higher NPV. However, on occasions, an option may have a lower BCR but higher NPV than its alternative. In this situation, the incremental benefits versus the incremental costs of the two alternatives should be examined, to determine whether the additional investment of the more expensive option will deliver benefits greater than this incremental investment cost.

As this is a probabilistic analysis, where thousands of alternative flooding frequency / intensity combinations are modeled, the results also show the probability of an option's BCR being higher than one (i.e., the proportion of combinations modeled where the total value of flood damage cost avoided outweighed the investment costs). This is useful for assessing the "risk appetite" or the level of risk that Government agencies or decision-makers would be willing to take to mitigate damage costs. If an option only has a 20% chance of having a BCR over one, then this would indicate that there was a 80% chance of the asset lying idle or unused, and so on balance, the option might not be pursued by many agencies. However, some agencies may determine that even a 10% risk of damages occurring would be unacceptably high (particularly for issues affecting human safety), and so would still pursue this option. Thus, decision-makers with different risk tolerances could legitimately use the same results to justify different courses of action.

<u>Results of the Economic Analysis of the Adaptation Options (Traffic Disruption Costs Only)</u>

The results of the analysis for Options One and Two are shown in Table 13 and Table 14. The results show that both options are economically viable with positive NPVs and BCRs over one. On average, Option One is preferable to Option Two as it has a higher average BCR at 3.5 compared to 2.6 for Option Two. This is due to its lower capital cost and also that the additional flood protection provided (in terms of benefits) by Option Two are insufficient to outweigh its additional cost. The magnitude of the BCR and NPVs is highly dependent upon the climate scenario adopted. In this analysis, it was assumed that all five scenarios are equally likely to occur,[77] and hence the BCRs and NPVs shown in Table 11 earlier and the end column of Table 13 and Table 14 represent the mean values of all five scenarios. Table 13 and Table 14 also show the BCRs and NPVs for Options One and Two under each of the five climate and land use

[77] The Wetter and Drier narratives were developed based on climate model outputs that assumed certain IPCC greenhouse gas emission scenarios. Since the IPCC does not assign probabilities to these scenarios, probabilities cannot be assigned to the resulting climate projection information.

scenarios. Also shown is the probability that the BCR will be over one, representing the proportion of the 1,000 Monte Carlo simulations run where a BCR over one occurred.

Table 13: Airport Boulevard Culvert Adaptation Option One Economic Analysis Results under Each Climate Scenario: 90th Percentile Results, No Property Damage Costs Included[78]

Climate Scenario	1	2	3	4	5	Average (mean) of All Scenarios
Description of Scenario	Observed 1980–2009 with Current Land-use	Observed 1980–2009 with Future Land-use	NOAA 90% Upper Confidence Limit	"Wetter" Narrative	"Drier" Narrative	
Present Value of Costs	$1.7m	$1.7m	$1.7m	$1.7m	$1.7m	$1.7m
Present Value of Benefits	$3.5m	$4.0m	$6.8m	$12.7m	$3.0m	$6.0m
NPV	$1.8m	$2.2m	$5.0m	$11.0m	$1.3m	$4.3m
BCR	2.0	2.3	3.9	7.3	1.7	3.5
Probability that BCR will be over 1[79]	36%	39%	68%	97%	30%	N/A

[78] Note: Scenarios 3-5 include assumptions on future land use per the discussion in Step 5 of this case study. The costs and benefits are presented in $2013, discounted over 30 years at 7%. The present value of benefits represent the 90th percentile result from the Monte Carlo simulation (1,000 observations). These values differ slightly from the 90th percentile avoidable costs shown in Figure 28, since not all floods are avoided by the culvert upgrades; overtopping still occurs during the most extreme storm events.
[79] Based upon proportion of Monte Carlo simulations run where a BCR of 1 or above was achieved

Table 14: Airport Boulevard Adaptation Option Two Economic Analysis Results under Each Climate Scenario: 90th Percentile Results, No Property Damage Costs Included[80]

Climate scenario	1	2	3	4	5	Average (mean) of all scenarios
Description of scenario	Observed 1980-2009 with current land-use	Observed 1980-2009 with future land-use	NOAA 90% Upper Confidence Limit	"Wetter" Narrative	"Drier" Narrative	
Present Value of Costs	$2.5m	$2.5m	$2.5m	$2.5m	$2.5m	$2.5m
Present Value of Benefits	$3.9m	$4.2m	$6.9m	$14.4m	$3.2m	$6.5m
NPV	$1.3m	$1.7m	$4.4m	$11.9m	$0.7m	$4.0m
BCR	1.5	1.7	2.8	5.8	1.3	2.6
Probability that BCR will be over 1[81]	24%	29%	59%	97%	20%	N/A

The results show that:

- Under the "Drier" narrative and with Observed precipitation depths (using either existing or future land use), fewer extreme floods occur in the Base Case with no adaptations. Whilst the BCRs for these scenarios are above one for both Options One and Two, there is a lower probability that the benefits will outweigh the capital costs (i.e., around between 30 and 39% probability that the benefits would not be recouped).
- Under the NOAA 90% Upper Confidence Limit scenario and the "Wetter" narrative, storm events causing flooding are more frequent resulting in higher damage costs across the appraisal period. The BCRs for these scenarios are higher than those under the other scenarios with over 50% probability that the avoided damage costs would exceed the investment costs. In the "Wetter" narrative, there is around a 97% probability of achieving a BCR over one.

Sensitivity Test - Inclusion of Property Damage Costs

Background

The central results for this benefit-cost analysis only include the traffic disruption costs resulting from Airport Boulevard and I-65 being closed in the event of a flood. However, the flood contour boundaries presented in Figure 19, Figure 20, and Figure 21 of the main report show that many buildings east of the culvert could be affected by flooding attributable to the culvert if the water

[80] Note: Scenarios 3-5 include assumptions on future land use per the discussion in Step 5 of this case study. The costs and benefits are presented in $2013, discounted over 30 years at 7%. The present value of benefits represent the 90th percentile result from the Monte Carlo simulation (1,000 observations). These values differ slightly from the 90th percentile avoidable costs shown in Figure 28, since not all floods are averted by the culvert upgrades; overtopping still occurs during the most extreme storm events.
[81] Based upon proportion of Monte Carlo simulations run where a BCR of 1 or above was achieved

reaches an elevation over 36 feet (11 meters). Acknowledging the aforementioned uncertainties of the location and depth of flooding east of the culvert crossing, a sensitivity analysis was undertaken to determine the effects of incorporating the estimated damage to buildings into the analysis.

Flooding of buildings incurs a number of direct and indirect economic costs, including:[82]

- **Structural Damage**: Structural damage increases as floodwaters rise.
- **Content Damage**: For example, stock held within a shop or furniture within a residential property.
- **Displacement:** Costs incurred when the occupants of a property need to move to temporary accommodation as a result of flood damage.
- **Loss of Business or Rental Income:** For example, when businesses are forced to temporarily close after a flood.
- **Value of Service:** Loss of function of a facility, such as electricity or water treatment.

Approach

This sensitivity test makes a preliminary, conservative estimate of the direct and indirect costs to properties lying within the flood contour boundaries between Montlimar Creek and I-65[83] using a methodology that broadly follows that used within the FEMA *HAZUS-MH* software used to estimate losses from flood damage.[84]

Note that there are limitations to this analysis in that:

- Only the flood boundaries are known, and not the actual flood elevation at each individual building. This would require more detailed hydrologic analysis.
- GIS data on individual land parcels (detailing building dimensions, occupancy type, and construction materials) was not available for this study, and as such, the numbers of buildings affected and building type have been estimated from publicly available imagery.
- Downstream increases to flow and attendant land use impacts associated with increasing the culvert capacity were not included. Additional detailed hydrologic analysis would be required to study this. It is highly recommended that this consideration be included on an actual project.

As a result of these limitations, these results should be treated with some caution and we recommend that further analysis using appropriate FEMA software tools be undertaken to build upon the findings here.

Using the flood contour maps provided in the main report, the number of damaged buildings and resultant economic cost for three flood elevations was estimated, as shown in Table 15. In the

[82] FEMA, 2011
[83] Flooding of properties to the east of I-65 was not considered attributable to the culvert overtopping since, without further detailed hydrologic analysis, it is even less clear if there would be sufficient volume of water to reach and inundate these structures. Also, it is possible that during a severe storm event these properties may already be flooded by the nearby Eslava Creek that lies to the east of the interchange.
[84] FEMA, 2012

analysis, we have conservatively only considered these three discrete steps rather than developing a continuous flood elevation / damage cost function. Thus, for floods below 36.3 feet (11.1 meters), no damage costs are incurred. For floods between 36.3 and 37.3 feet (11.1 and 11.4 meters), four buildings are damaged and so forth.

Table 15: Estimated Numbers of Damaged Buildings and Economic Losses Occurring from Flooding at the Airport Boulevard Culvert

Flood Elevation (feet)	Damaged Buildings[85]	Estimated Economic Cost of Damage per Flood[86]
Less than 36.3 feet	0	$0m
36.3 to 37.2 feet	4	$11.6m
37.3 to 37.5 feet	18	$106m
Greater than or equal to 37.6 feet	20	$122m

Results and Analysis

Table 16 and Table 17 show the results of the sensitivity test where property damage costs are included within the benefit-cost analysis.

Table 16: Airport Boulevard Culvert Adaptation Option One Property Damage Cost Sensitivity Test Results under Each Climate Scenario: 90th Percentile Results, Property Damage Costs Included[87]

Climate Scenario	1	2	3	4	5	Average (mean) of All Scenarios
Description of Scenario	Observed 1980–2009 with Current Land-use	Observed 1980–2009 with Future Land-use	NOAA 90% Upper Confidence Limit	“Wetter” Narrative	“Drier” Narrative	
Present Value of Costs	$1.7m	$1.7m	$1.7m	$1.7m	$1.7m	$1.7m
Present Value of Benefits	$9.7m	$11.5m	$17.8m	$83.3m	$8.2m	$26.1m
NPV	$8.0m	$9.7m	$16.1m	$81.6m	$6.4m	$24.4m
BCR	5.6	6.6	10.2	47.9	4.7	15.0
Probability that BCR will be over 1[88]	44%	50%	76%	99%	41%	N/A

85 Estimated from aerial photos and LIDAR two-foot elevation contours.
86 Estimated using equations and data provided within the FEMA HAZUS manual. Values indexed to $2013 using US CPI-U.
87 Costs and benefits presented in $2013, discounted over 30 years at 7%. The present value of benefits represent the 90th percentile result from the Monte Carlo simulation (1,000 observations).
88 Based upon proportion of Monte Carlo simulations run where a BCR of one or above was achieved.

Table 17: Airport Boulevard Culvert Adaptation Option Two Property Damage Cost Sensitivity Test Results under Each Climate Scenario: 90th Percentile Results, Property Damage Costs Included[89]

Climate Scenario	1	2	3	4	5	Average (mean) of All Scenarios
Description of Scenario	Observed 1980–2009 with Current Land-use	Observed 1980–2009 with Future Land-use	NOAA 90% Upper Confidence Limit	"Wetter" Narrative	"Drier" Narrative	
Present Value of Costs	$2.5m	$2.5m	$2.5m	$2.5m	$2.5m	$2.5m
Present Value of Benefits	$10.3m	$11.8m	$18.5m	$97.5m	$8.9m	$29.4m
NPV	$7.8m	$9.3m	$16.0m	$95.0m	$6.4m	$26.9m
BCR	4.1	4.7	7.4	38.9	3.6	11.7
Probability that BCR will be over 1[90]	38%	44%	71%	99%	35%	N/A

It can be seen that in this test, both options have BCRs over one under Observed precipitation depths and all the other scenarios indicating that they are economically viable and demonstrate very good value for money even with future uncertainty. The BCRs for Option One are higher than those for Option Two on average (15 versus 11.7) indicating that this option is preferred. However, for the "Wetter" narrative, the average NPV for Option 2 is higher than that for Option 1. Incremental analysis shows that the additional benefits (avoided costs) outweigh the incremental capital costs and so Option 2 should be considered.

To summarize, the results show that:

- Under the "Drier" narrative and Observed historical precipitation depths (with existing or future land use), even though floods are less frequent, the damage costs incurred far outweigh the capital costs for the provision of infrastructure, resulting in BCRs over one. However, the probability that the BCR would be over one is less than 50% for both Options 1 and 2.
- Under the NOAA 90% Upper Confidence Limit scenario and the "Wetter" narrative, there is a higher probability of achieving a BCR over one. Under the "Wetter" narrative, whilst the BCR for Option One is higher than that for Option Two, the NPV of the latter is greater. The incremental benefit of Option Two over Option One ($14.1 million) outweighs the $800,000 additional capital costs. Therefore, Option Two would be preferred when considering land use implications (noting the aforementioned limitations of this analysis). Under the "Wetter" narrative, almost 100% of simulations run generated BCRs over one.

[89] Costs and benefits presented in $2013, discounted over 30 years at 7%. The present value of benefits represent the 90th percentile result from the Monte Carlo simulation (1,000 observations).
[90] Based upon proportion of Monte Carlo simulations run where a BCR of one or above was achieved.

Economic Analysis Conclusions

The benefit-cost analysis shows that on average, Option One with a BCR of 3.5 would be preferred over Option Two with a BCR of 2.6, as the additional flooding protection provided by Option Two would not outweigh the additional capital cost. However, under three of the scenarios tested, the probability of achieving a BCR over one was less than 50%.

A preliminary analysis into the avoidable damage costs to properties surrounding the culvert crossing has been undertaken as a sensitivity test, which revealed that the BCRs for both Options One and Two would be greatly enhanced under all climate change scenarios, reaching 47.9 for one scenario. Considering net present values, Option Two performs equal to or better than Option One when considering land use impacts. However, it should be noted that the land use impacts are very much high-level preliminary estimates, and further analysis is recommended to determine whether floodwaters would actually reach and inundate these structures.

Step 9 –Evaluate Additional Decision-Making Considerations

While the statistical analyses provide a starting point and documented basis for making decisions, the numerical results by no means represent the final recommendation in the decision-making process. Many other factors that reflect the reality of the economy, the environment, and the social implications of the adaptation options must be considered. While the economic analysis tends to address some of these issues if all costs are considered, the tolerance for risk, the other needs of the stakeholders and the ability to fund change are equally, if not more important than the bare numbers. Any decisions made must account for all of these impacts and come from a general consensus of the engineering, planning, operations, and maintenance staff along with representatives of the affected stakeholders.

Specific considerations include items of concern for any typical project such as:

- Broader project sustainability beyond just climate change impacts (i.e., the “triple bottom line” of social, environmental, and economic concerns)
- Project feasibility and practicality
- Ongoing maintenance needs
- Maintenance funds availability
- Capital funds availability
- Stakeholders’ (public and government agencies) tolerance for risk of service interruption and associated costs of all types (note: this affects how the economic analysis is perceived as well)
- Stakeholders’ expected quality or level of service

After considering all of the above, decision-makers should ask the question, “Is this project worth pursuing?” Adaptation of infrastructure in response to the potential for changing climate conditions is proposed to fit within the broader context of any transportation agencies’ capital improvement program and ongoing asset management efforts. Adaptation for the sake of adaptation is not expected to meet each of the special considerations noted above and is best viewed as a component of a larger decision-making process.

Step 10 –Select a Course of Action

The economic analysis showed that the adaptation options proposed may or may not have a high chance of being cost effective; depending on how climate changes in the future. If the "Wetter" narrative were to occur, undertaking either of the adaptation options is almost certain to be cost-effective; a 97% chance or greater that the BCR would be greater than one. However, if the "Drier" narrative were to occur or if climate did not change, there is a good chance that the adaptation options proposed would not be cost-effective; a 50% chance or less that the BCR would be greater than one.

That said there is uncertainty in the probability of various rainfall events occurring even with current climate. This can have a significant bearing on the cost-effectiveness of any adaptation project as shown by the analysis of the NOAA 90% Upper Confidence Limit precipitation value. If one considered the NOAA 90% Upper Confidence Limit for current precipitation, this analysis indicated that there is *at least* a 59% chance that either of the adaptation options would be cost effective. If land use impacts are also considered, there is *at least* a 71% chance of the adaptation options being cost-effective.

This observation, along with the strong performance of the adaptation options under the "Wetter" narrative, leads to a preliminary recommended course of action to undertake adaptation Option One. Of the two adaptation options considered, this option performed best in the economic analysis considering traffic impact costs alone[91] and also had a greater probability of being cost effective under all scenarios tested. Another observation is that Option One has the best average net present value in the analysis (not considering land use flooding impacts) and a reasonable average benefit-cost ratio regardless of whether land use flooding impacts are considered. Finally, Option One is the less costly of the two options – a very relevant factor in the decision-making process.

The recommended course of action should only be seen as preliminary and subject to change pending the additional detailed analyses suggested in this document. For example, if further hydrological analysis confirms extensive flooding of surrounding properties is attributable to the culvert, more consideration should be given to adaptation Option 2. Furthermore, this study did not include a component engaging local stakeholders in a dialogue over which design would be "best" and there is no way to predict what decisions such a discussion would lead to. This discussion should acknowledge the possibility that, under any of the scenarios, there is a possibility that no serious floods will occur and the adaptation will never be "used." The

[91] Given the previously noted uncertainties in the extent of the flooding impacts on surrounding land uses caused by the culvert, more weight was given to the analysis considering just the traffic impacts.

dialogue would no doubt be heavily shaped by local risk tolerance and other factors and may very well lead to a different decision than the one arrived at through this assessment.

Step 11 – Plan and Conduct Ongoing Activities

Regardless of which design option is chosen (if any), the effects of climate on the culvert cannot be expected to remain constant as has generally been assumed in the past. Thus, the climate stressors and the culvert's performance should be monitored after the project is constructed (or it is determined that the existing culvert be left in place) and the effects on the culvert must be revisited and periodically assessed to determine if the culvert's critical design thresholds are being reached. Such monitoring and periodic assessment can help indicate if it might be necessary to implement additional improvements, change design guidelines, and / or alter operation and maintenance practices.

For a culvert, monitoring would consist primarily of keeping tabs on the most recent rainfall statistics (is rainfall becoming more frequent or intense or both?). Other questions a monitoring regime could help answer may include:

- Is land use within the drainage area becoming more impervious?
- Are storm water management facilities performing as expected?
- Is flooding becoming more frequent and widespread?
- Are there increased impacts (e.g., erosion) to the downstream channel or properties downstream from the culvert?

Conclusions

This case study has, using the *General Process for Transportation Facility Adaptation Assessments*, demonstrated how a large culvert can be analyzed for climate change impacts resulting from a projected increase in precipitation depths. Adaptation options were identified and tested using a benefit-cost framework. Ultimately, this information must be shared with local stakeholders and discussed before any locally preferred decisions can be made on what adaptive actions (if any) would be appropriate for the community.

The process shown is broadly applicable to other large culverts across the country where use of 24-hour duration precipitation projections, a standard climate model output, is appropriate. For smaller culverts where 24-hour projections may not be applicable, further research into the development of climate change IDF curves is recommended to aid in the translation of climate model outputs into inputs useful for engineering design.

4.4.2 Bridge Over Navigable Waterway Exposure to Sea Level Rise – The Cochrane-Africatown USA Bridge

Introduction

Sea level rise is a potential threat to coastal bridges of all types including non-navigable bridges, navigable moveable bridges, and navigable non-moveable bridges. With sea level rise, vertical clearances can be reduced to the point that navigation is impeded, corrosion may be enhanced, and in some cases, the bridge itself (or its approaches) may become permanently inundated. This case study assesses whether a coastal bridge, the Cochrane-Africatown USA Bridge, could limit navigation on the tidal Mobile River as a result of projected sea level rise scenarios. The sea level rise analysis for the bridge was conducted using the 11-step *General Process for Transportation Facility Adaptation Assessments* and this serves as the organizing framework for this case study.

> **Case Study Highlights**
>
> **Purpose:** Evaluate whether sea level rise could impede the navigational clearance of the Cochrane-Africatown USA Bridge.
>
> **Approach:** The sea level rise scenarios developed previously for this study were compared to site-specific results from the USACE Sea-Level Change Curve Calculator to demonstrate an alternative approach for determining relative sea level rise for specific sites. Then, the sea level rise was evaluated against the current vertical clearance of the bridge to determine if minimum clearance requirements might be violated.
>
> **Findings:** Navigational clearance of the bridge will not be affected under lesser sea level rise scenarios, but it would be affected under the 6.6 foot (200 cm) sea level rise by 2100 scenario.
>
> **Viable Adaptation Options:**
>
> - Restrict ship heights passing under that bridge, which could affect the type of ports located on either side of the bridge
> - At end of bridge lifespan, replace with a higher bridge or movable bridge
>
> **Other Conclusions:** Since navigational clearance may not be affected until 2100, Mobile has time to consider adaptation options. Forward-looking planning on land-use might be part of the solution. If it is too costly to replace the bridge, restricting the type of vessels that can pass under the bridge is a viable option. Doing so would change the types of facilities that could be situated upriver over time, but the Mobile community has time to adjust to such changes in land use.

The analysis shows that projected sea level rise may present a navigation challenge at the Cochrane-Africatown USA Bridge in one of the three sea level rise scenarios tested. However, any impediments to navigation are not anticipated to occur until late in the facility’s design life. Given this finding, no immediate adaptation actions are recommended for this facility to address sea level rise although the situation should continue to be monitored over time.

Application of the General Process for Transportation Facility Adaptation Assessments

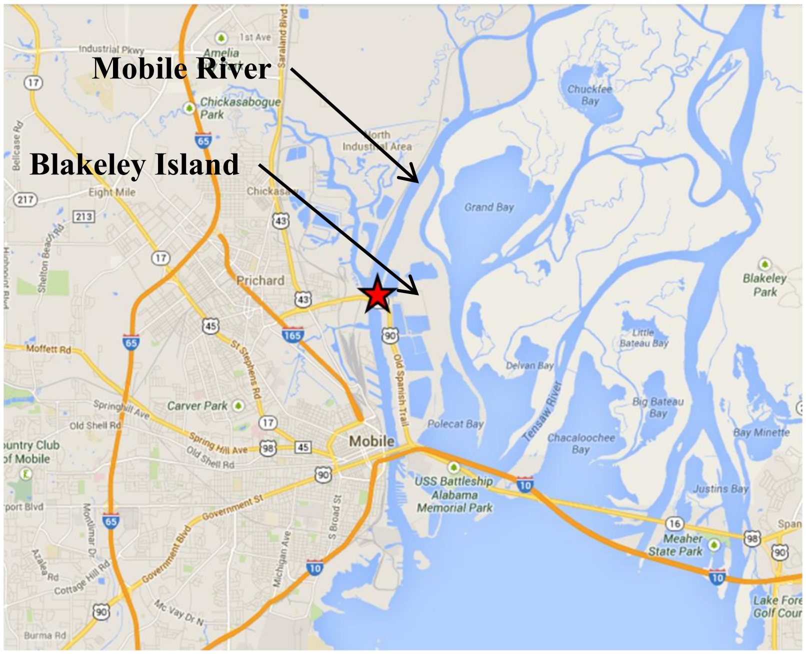
Step 1 – Describe the Site Context

The case study bridge, The Cochrane-Africatown USA Bridge, is located on the north side of the Mobile metropolitan area and carries Alternate US 90 over the tidally influenced Mobile River between Blakeley Island and the Magazine industrial area (see Figure 29). The bridge provides a link between the industrial land uses on Blakely Island and I-165. Alternate US 90 also functions as a bypass around downtown Mobile for travelers headed across Mobile Bay on I-10 (the

"Bayway") or US 90/98 (the "Causeway") to / from points north along I-165, I-65, US 43, or US 45.

The Mobile River is an important navigable waterway used both by ocean-going vessels destined for the industrial area north of the bridge and barge traffic accessing the Tennessee-Tombigbee Waterway, a canal system that connects Mobile to interior Alabama and the Tennessee River system. It should be noted that there are no other bridge crossings that would act as an impediment to navigation on the Mobile River south of the case study bridge: this is the first bridge that has the potential to limit coastal navigation for large ocean-going ships (the I-10 and US 90 crossings to the south are tunnel crossings). A future I-10 bridge over the Mobile River is currently being studied. If built, sea level rise impacts to navigation on the Mobile River should be accounted for in its design.

Figure 29: Location of the Cochrane-Africatown USA Bridge within the Mobile Metropolitan Area[92]



Surrounding land uses to the Cochrane-Africatown USA Bridge are heavily industrial (see Figure 30).

[92] Source of base map: Google Maps (as modified)

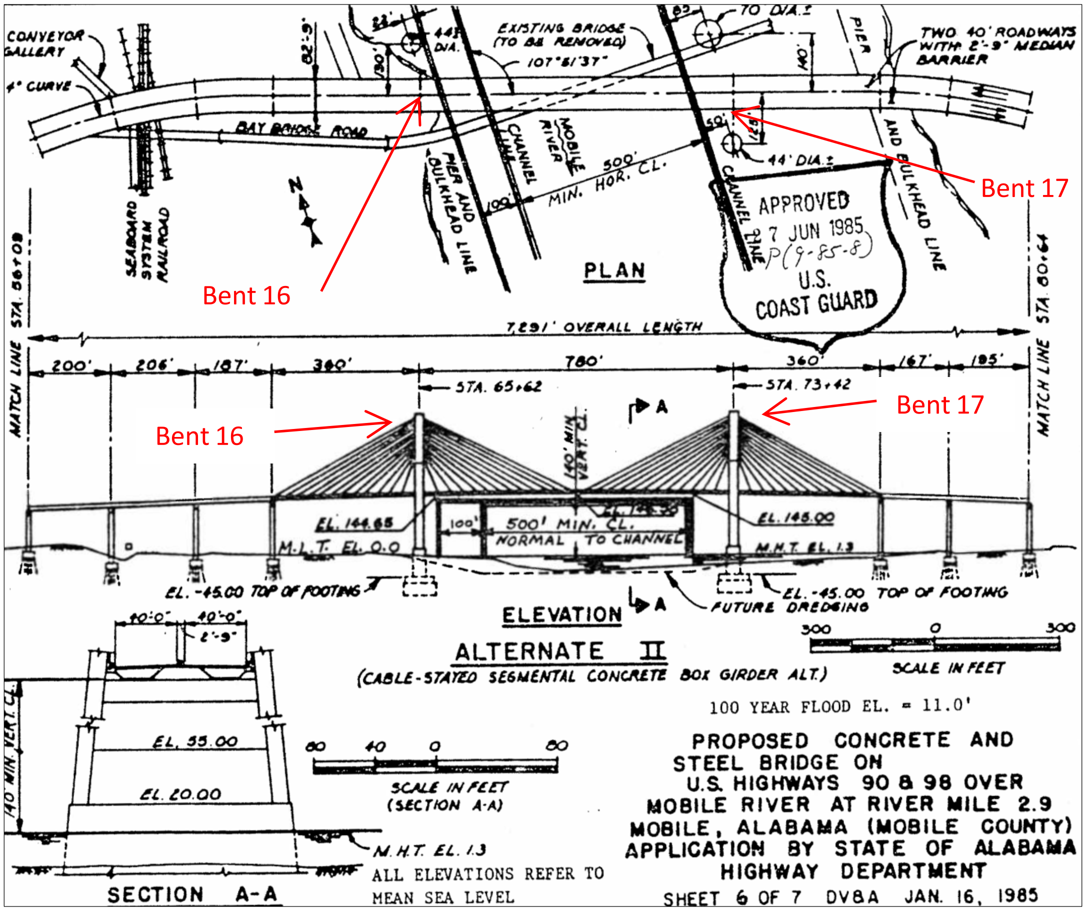
Figure 30: Land Use in the Vicinity of the Cochrane-Africatown USA Bridge[93]

Step 2 – Describe the Existing Facility

The bridge that is the subject of this case study was completed and opened in 1991. The bridge is 82.8 feet (25.2 meters) wide[94] and approximately 7,291 feet (2,222 meters) long from its western base at a location between Tin Top Lane and Paper Mill Road and its eastern base on Blakeley Island. A total of 32 bents,[95] each one assigned a unique identifying number, support the bridge from Bent 1 at the beginning of the ramp on the west end to Bent 32 on Blakeley Island. The case study analysis will focus on the portion of the bridge between Bents 16 and 17; this portion was chosen because the U.S. Coast Guard has enforced vertical clearance requirements for navigation on this portion of the bridge. Figure 31 highlights the relevant portion of the bridge in the plan (overhead) view and elevation (side) view; the same section is illustrated in a photograph in Figure 32.

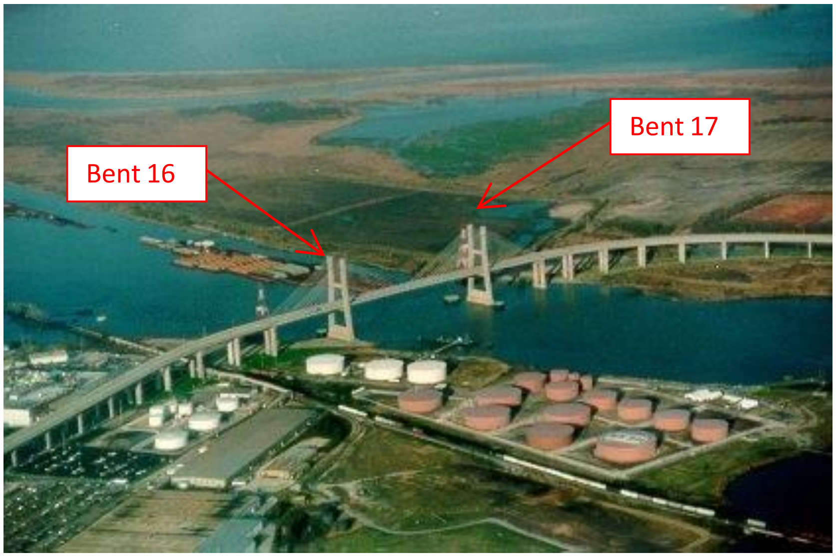
[93] Source of base map: Google Maps (as modified)
[94] Width as measured to the outside faces of the parapets. The parapets are the outside walls on either side of a bridge that are designed to prevent vehicles from careening off the structure.
[95] Bents, also known as piers, are the vertical columns supporting each bridge span along with the horizontal member, called a cap, which holds them together.

Figure 31: Plan and Elevation Views of the Cochrane-Africatown USA Bridge Showing the Section of Analysis[96]



[96] Source: USCG, 1985 (as modified). Note: All elevations shown in the image refer to mean sea level. Mean sea level is the average of the water elevations recorded at each hour of the day over the current National Tidal Datum Epoch. The tidal epoch is the specific 19-year period over which NOAA uses to obtain observations that are used to develop tidal statistics.

Figure 32: Image of the Cochrane-Africatown USA Bridge Showing the Section of Analysis between Bents 16 and 17[97]



The span between Bents 16 and 17 is 780 feet (237.7 meters) long. The superstructure[98] consists of two cable-stayed segmental concrete box girders[99] (see Figure 33). Most of the bents in the study segment consist of two square concrete piles[100] topped with a concrete pile cap.[101] Bents 16 and 17 constitute the main piers for the center span of the bridge and provide the support for two planes of high-tension steel cables. The bents extend 350 feet (106.7 meters) above mean low tide and the top of the footings for the bents are located 45 feet (13.7 meters) below mean low tide (see Figure 31). The U.S. Coast Guard bridge permit approved for the Cochrane-Africatown USA Bridge provides for a minimum 600 foot (182.9 meter) horizontal clearance between Bents 16 and 17 for passage of vessels where a minimum vertical clearance of 140 feet (42.7 meters) must be maintained between the bottom of the span and the mean high tide elevation.

Note that the elevations provided in the U.S. Coast Guard permit seen in Figure 31 are in reference to mean sea level.[102] No specific tidal epoch[103] is given in the permit. For consistency within this report, all elevations within the text are provided in reference to the North American Vertical Datum of 1988 (NAVD88) datum:[104] thus, the elevation values described in the text will not match the U.S. Coast Guard permit figures. With that in mind, the western and eastern spans have a bottom elevation of 145 and 145.3 feet (44.2 and 44.3 meters), respectively. The vertical clearance of 140 feet (42.7 meters) must be maintained between these spans and the mean high tide elevation of 1.6 feet (0.5 meters) as reported. The U.S. Coast Guard permit indicates a

[97] Source: Volkert Engineering, Planning, and Environmental Consulting, 2013 (as modified)
[98] The superstructure is the top part of the bridge and consists of the horizontal support girders, deck, and parapet walls preventing vehicles from falling off the structure.
[99] Girders are the main horizontal supporting members of the bridge.
[100] Piles are the vertical support structures extending from the bridge deck to the seabed below.
[101] The pile cap is the horizontal member that ties together the vertical piles.

provided mean high tide clearance of 143.3 feet (43.7 meters) at the western limit of the clearance envelope and 143.7 feet (43.8 meters) at the eastern limit of the clearance envelope.

Figure 33: Image of Typical Bent Configuration for Bents 1 to 15 and Bents 18 to 32 on the Cochrane-Africatown USA Bridge[105]

Step 3 – Identify Climate Stressors That May Impact Infrastructure Components

Sea level rise is the primary climate change related environmental factor considered in this study. Storm surge and high wind speeds may also be concerns for this facility. See Section 4.4.5 for a discussion of how a bridge can be analyzed for storm surge impacts coupled with sea level rise.

Step 4 – Decide on Climate Scenarios and Determine the Magnitude of Changes

The impacts of three potential global sea level rise values, based on projected climate changes, were considered for this case study. The values considered include:

- One foot (0.3 meters) of global sea level rise by 2050
- 2.5 feet (0.8 meters) of global sea level rise by 2100
- 6.6 feet (two meters) of global sea level rise by 2100

The one and 2.5 foot (0.3 and 0.8 meter) values represent two points in time in an intermediate sea level rise scenario that was selected in the *Climate Variability and Change in Mobile, Alabama* report[106] as the midrange of National Research Council estimates. The high projection of 6.6 feet (two meters) is derived from recent research indicating that the rates of sea level rise might actually be faster than initially thought based on a growing understanding of ice sheet melting dynamics; its use here illustrates an application of the precautionary principle whereby,

[102] Mean sea level is the average of the water elevations recorded at each hour of the day over the current National Tidal Datum Epoch. The tidal epoch is the specific 19-year period over which NOAA uses to obtain observations that are used to develop tidal statistics.
[103] The tidal epoch is the specific 19-year period over which NOAA uses to obtain observations that are used to develop tidal statistics.
[104] A datum is a reference plane used as a starting point from which to measure elevations.
[105] Source: Google Earth Street View
[106] USDOT, 2012

even though there is scientific uncertainty, plausible possibilities should be considered in order to prevent harm to the public. For more information on how these scenarios were developed, please refer to the *Climate Variability and Change in Mobile, Alabama* report.[107]

In addition to global sea level rise values, local vertical land movements may have a significant exacerbating or mitigating effect on local sea level change at any given transportation facility. Considering vertical land movements allows one to determine local relative sea level rise, the factor most relevant to facility design. Vertical land movement at the Cochrane-Africatown USA Bridge is 0.03 inches per year (0.7 millimeters per year) of uplift as obtained from U.S. Geologic Survey (USGS) data.[108] Thus, land movements at the site have a slight mitigating effect on global sea level rise. Relative sea level rise at the Cochrane-Africatown USA Bridge is therefore 0.9 feet (0.27 meters) for the intermediate scenario in 2050, 2.3 feet (0.7 meters) for the intermediate scenario in 2100, and 6.4 feet (1.95 meters) for the high scenario in 2100.

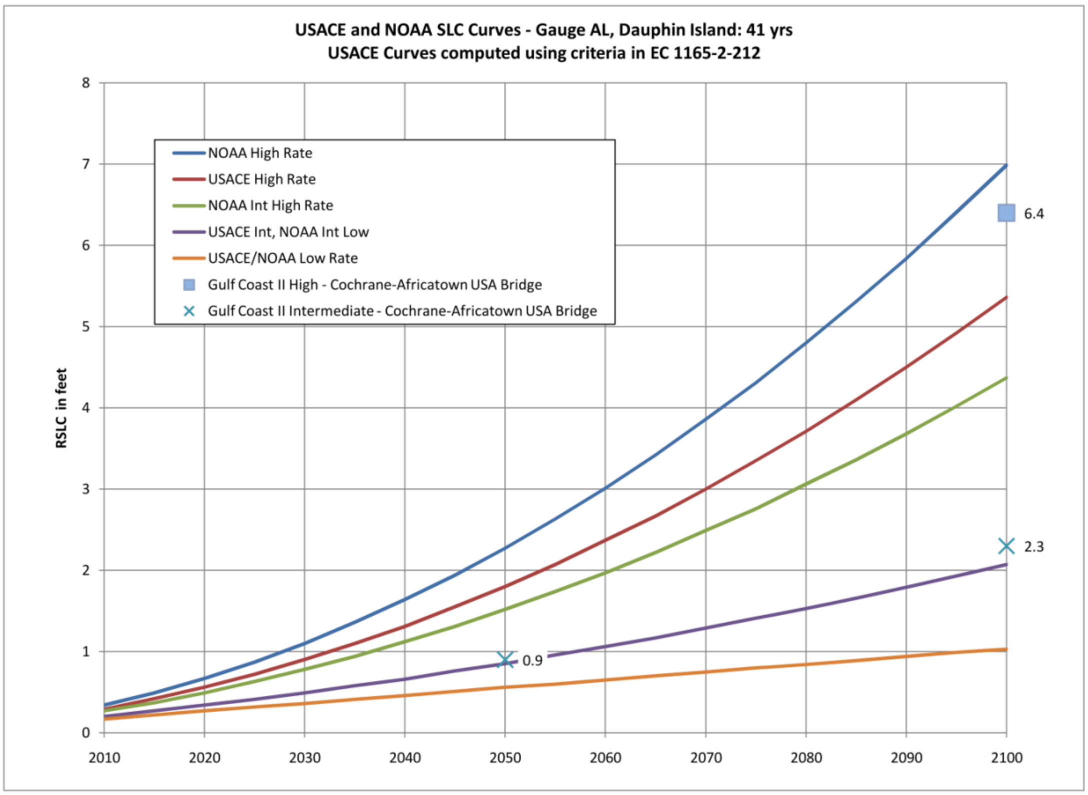
For locations outside the Mobile region, sea level rise scenarios can be generated using guidance developed by the U.S. Army Corps of Engineers (USACE).[109] The latest version of this guidance was released with an associated on-line calculation tool called the *Sea-Level Change Curve Calculator* that links to key tidal stations where a sufficient period of record exists to support projections of sea level rise with consideration of vertical land movement. To illustrate its use, sea level projections from the tool were obtained and compared with the aforementioned projections developed for the Gulf Coast Study Phase 2. The calculations are based on the Dauphin Island tide gauge and assume a starting date of 2010. The only information required to be provided to the *Sea-Level Change Curve Calculator* is selection of the closest NOAA gauge which is provided as an option from several gauges around the country. Once this is selected the land subsidence rate of 0.05 inches per year (1.22 mm/yr) at Dauphin Island is automatically provided. The user then selects the end date, 2100, for the project and the option to include the NOAA curves. The computations were based on the USACE Guidance.[110]

The results are presented in Figure 34 and include curves from (1) the USACE showing low, medium, and high estimates and (2) from NOAA showing low, low intermediate, high intermediate, and high estimates. The NOAA curves shown in Figure 34 are calculated based on criteria contained in a NOAA report[111] presenting global sea level rise scenarios. These global sea level rise scenarios were developed for use in the National Climate Assessment[112] draft report which indicates a one to four foot (0.3 to 1.2 meter) likely range for global sea level rise and a larger 0.7 to 6.6 foot (0.2 to two meter) range suggested for use in risk-based analyses (an approach consistent with that used in this report). Both the USACE and NOAA curves are based

[107] USDOT, 2012
[108] USDOT, 2012
[109] USACE, 2013
[110] USACE, 2011
[111] Parris, Bromirski, Burkett, Cayan, Culver, and Hall, 2012
[112] NOAA, 2013c

on the original parabolic equations presented in the National Research Council (NRC) report[113] on the engineering implications of responding to sea level rise. The equation includes a linear term that is used to simulate the current sea level trend and the nonlinear parabolic term which accounts for acceleration of the current sea level trend due to climate change projections. The differences between the USACE and NOAA curves stem from different sources and / or publication dates regarding global sea level rise projections which are key input parameters for the calculation. For further detail see the USACE guidance,[114] NOAA report,[115] and book by NRC.[116]

Figure 34: Comparison of Relative Sea Level Rise Projections for Dauphin Island, Alabama[117]



As shown in Figure 34, the sea level rise projections generated for this study at the Cochrane-Africatown USA Bridge lie within the envelope of the USACE and NOAA curves at Dauphin Island. In addition, the 2050 one foot (0.3 meter) and 2100 2.5 foot (0.8 meter) projections for sea level rise agree with the trend of the USACE intermediate and NOAA intermediate low projections. The highest sea level rise projection identified for this study, 6.6 feet (two meters), lies slightly below the high projection for NOAA and about 1.2 feet (0.4 meters) above the high

[113] NRC, 1987
[114] USACE, 2013
[115] Parris et al., 2012
[116] NRC, 1987
[117] USACE, 2013

projection from the USACE. Thus, the sea level rise projections for this study are consistent with both the USACE and NOAA guidance one might use to do adaptation studies in other coastal locations around the country. The three sea level rise values used in this study do not match exactly with the projections shown in Figure 34 because the most conservative and most extreme scenarios were not the ones selected for use in the Gulf Coast Study. Analysis of sea level rise impacts on the navigational clearance of bridge spans in other areas may choose to analyze the higher and lower bounds of sea level rise projections.

Step 5 – Assess Performance of the Existing Facility

The progression of sea level rise over the ensuing decades could present challenges for maintaining the required navigation allowances for the Cochrane-Africatown USA Bridge. Table 18 shows the projected clearances at the western and eastern side of the navigation envelope. Given the required minimum vertical clearance of 140 feet (42.7 meters), global sea level rise in combination with vertical land movement does not violate the navigation requirements for the main span clearance limits in the year 2050 when there is a 1 foot (0.3 meter) rise or in the year 2100 when there is a 2.5 foot (0.8 meter) rise. This is because the initial design provided an allowance above the 140 foot (42.7 meter) minimum requirement. However, global sea level rise will violate the permit requirement under the 6.6 foot (two meter) global sea level rise scenario for 2100. In this scenario, the vertical clearance near Bent 16 is only 137 feet (41.8 meters) and near Bent 17 is 137.3 feet (41.8 meters), well below the 140 foot (42.7 meter) minimum mandated in the U.S. Coast Guard permit.

Step 6 – Identify Adaptation Option(s)

No adaptations are required for the 2050 one foot (0.3 meter) and the 2100 2.5 foot (0.8 meter) global sea level rise scenarios. For the projected year 2100 6.6 foot (two meter) global sea level rise scenario, several adaptive design options to accommodate sea level rise for the Cochrane-Africatown USA Bridge were considered and are listed in Table 19 along with the pros and cons for each.

It should be noted that any potential sea level rise issues on the Cochrane-Africatown USA Bridge will occur after the conclusion of its design life, which, assuming the typical 75 year lifespan for a bridge, ends in 2066 (although it is common for bridges in well-maintained condition to remain in service well past their designated design lives). Given this, it might very well be the case that a full replacement of the existing structure with a design that accounts for anticipated sea level changes will be a more cost-effective solution than retrofitting the existing structure.

For bridges with navigable spans that might cause significant navigational impacts due to sea level rise during their design lives, future bridge rehabilitation efforts will need to consider significant modifications to reduce navigation clearance related impacts such as elevating the deck. Investigation of how substructures can be extended, modified, or completely rebuilt will need to be studied at the time of rehabilitation on a case-by-case basis.

Table 18: Clearances for Navigation at the Cochrane-Africatown USA Bridge under Projected Sea Level Rise Values

Global Sea Level Rise	Relative Sea Level Rise[118]	Clearance Between Bottom Slabs / Beams and Mean High Tide[119]	
		West Side of Envelope	East Side of Envelope
Current Conditions	None	143.4 feet (43.7 meters)	143.7 feet (43.8 meters)
2050 - One Foot (0.3 meters)	0.9 feet (0.3 meters)	142.5 feet (43.4 meters)	142.8 feet (43.5 meters)
2100 - 2.5 Feet (0.75 meters)	2.3 feet (0.7 meters)	141.1 feet (43.0 meters)	141.4 feet (43.1 meters)
2100 - 6.6 Feet (2.0 meters)	6.4 feet (2.0 meters)	137.0 feet* (41.8 meters)	137.3 feet* (41.8 meters)

*Does not meet minimum vertical clearance requirement of 140 feet (42.7 meters)

Step 7 – Assess Performance of the Adaptation Option(s)

The performance of each of the adaptation options discussed across the three sea level rise values evaluated are discussed below:

- **Option 1** maintains the existing bridge and therefore the 140 foot (42.7 meter) clearance requirement would not be met under the 2100 6.6 foot (two meter) scenario. Under this adaptation option, it would be accepted that vessel passage would be restricted to smaller vessels. This option would meet the requirements under the other sea level rise values tested.

- **Option 2** would meet the U.S. Coast Guard permit requirements under the year 2100 6.6 foot (two meter) global sea level rise scenario and all the other sea level rise values tested.

- **Option 3** would eliminate the bridge completely by replacing it with a tunnel, therefore the clearance requirements would be met under all sea level rise values investigated. However, this option is likely to be less desirable as it would eliminate the main route from hazardous materials to and from the port facility which was likely one of the primary purposes for construction of the Cochrane Africatown-USA Bridge.

- **Option 4** would meet the U.S. Coast Guard permit requirements under the year 2100 6.6 foot (two meter) global sea level rise scenario and all the other sea level rise values tested but, due to the vulnerability of moveable bridges to power loss and damage from storms, this option is not likely to be chosen.

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[118] These values account for global sea level rise along with an estimated land movement uplift of 0.1 feet (three centimeters) in 2050 and 0.2 feet (6.1 centimeters) in 2100.
[119] As reported in the Coast Guard permit diagram seen in Figure 31.

Table 19: Adaptation Options Considered for the Cochrane-Africatown USA Bridge

Possible Alternatives	Description	Pros	Cons
Restrict Marine Traffic (Option 1)	• The existing bridge would remain as is and marine traffic would be limited to only those vessels whose specific clearance requirements would be maintained	• No capital costs associated with a large project undertaking • No traffic outages	• Restrictions placed on larger vessel classes
Raise the Superstructure (Option 2)	• Rebuild the upper portion of the towers to accommodate the higher profile while preserving the stay cable angles so that the edge girders could be reused • Raise approach roadways to meet the raised center span. • Potentially reuse existing approach spans, main span foundations and towers below the roadway.	• Can elevate roadway to meet 6.6 feet (two meter) sea level rise.	• Large project undertaking with few comparable examples • Traffic outages during construction • Large capital and maintenance costs • Lengthy project development time
Replace the Bridge with a Tunnel (Option 3)	• Eliminate the existing bridge and replace with tunnel crossing	• Eliminates need for navigational clearances under all sea level rise scenarios	• Large capital and maintenance cost • Eliminates the only roadway for hazardous materials across the Mobile River as restrictions are in place for the I-10 and US 90 tunnels. • Long period of traffic disruption
Retrofit or Replacement of Fixed Span with Moveable Bridge Center Span (Option 4)	• Replace or retrofit center span of bridge with a moveable span	• Involves work only within center span • Allows larger vessels passage under bridge	• Vulnerabilities to loss of power or damage to mechanical systems • Interruption to highway traffic • FHWA policy is to provide fixed bridges wherever practicable

Step 8 – Conduct an Economic Analysis

An economic analysis was not included in this case study but is recommended for facility-level adaptation assessments. See Section 4.4.1 for an example of how an economic analysis was applied to a culvert exposed to changes in precipitation due to climate change.

Step 9 – Evaluate Additional Decision-Making Considerations

Other important factors that will likely influence whether the Cochrane-Africatown USA Bridge is adapted to accommodate sea level rise include:

- How existing or proposed port facilities utilizing the Mobile River to the north of the Cochrane-Africatown USA Bridge decide to adapt to sea level rise. If these facilities are compromised by sea level rise (or even if they are not), one broader-focused adaptation option might be to consider locating or relocating these facilities south of the bridge thereby reducing the navigation height requirements for the structure such that it only needs to accommodate barge traffic.
- How vehicle traffic volumes on the bridge evolve over time through shifts in population, land uses, or loss of service on other major roadways. Increased traffic volumes greater than the current service level design might provide added impetus to making changes to the bridge whereby sea level rise adaptations could be worked in. On the other hand, if sea level rise negatively affects adjoining land use to the point where they are abandoned, the need for the bridge within the larger transportation network may be diminished. In some cases, not likely in this one, a bridge may no longer be necessary if the land uses it serves are no longer viable due to an increase in sea levels.
- Potential local concerns from adjacent neighborhoods in response to plans for a higher bridge.

Step 10 – Select a Course of Action

In the case of this bridge, given the scenarios identified, no adaptive actions are recommended at this time. As sea level rise is a relatively gradual phenomenon (even considering its projected acceleration after mid-century) time will allow for continual evaluation of changes in actual sea levels as they relate to the curves shown in Figure 34. As 2100 approaches, it might be that the actual trend line is closer to the 2.5 foot (0.8 meter) scenario rather than the 6.6 foot (two meter) scenario and no adaptations will be required. In the case of the Cochrane-Africatown USA Bridge, it makes little sense to re-design the facility for the 6.6 foot (two meter) scenario until more information supports or conflicts this trend line. If the projections bear out as shown in Figure 34, this might become apparent by mid-century and a decision should be made then on how to proceed.

Step 11 – Plan and Conduct Ongoing Activities

Since whether sea level rise will follow the upper trend line that requires adaptation is highly uncertain at this time and sea level rise is gradual, the key recommendation for this analysis is to monitor trends in actual sea levels over time in the Mobile region and compare them to the projected sea level rise scenarios in Figure 34. This ongoing evaluation will inform conversations about the likely trends and may lead to identification of inflection points where trend analysis may indicate that the threat is more or less severe than projected. Local officials may wish to identify a "trigger threshold" for actual sea level change (informed by the sea level rise curves and the projected time required to plan, design, finance, and construct a chosen adaptation option). The trigger level would be less than the level of change required to violate the U.S. Coast Guard 140 foot (42.7 meter) requirement such that, when the trigger threshold is crossed, planning and financing activities for adapting the facility need to commence.

Conclusions

This case study has, using the *General Process for Transportation Facility Adaptation Assessments*, demonstrated how a bridge can be analyzed for sea level rise impacts on navigation. Navigation requirements for the Cochrane-Africatown USA Bridge were violated under only one of the climate scenarios tested; the scenario projecting 6.6 feet (two meters) of sea level rise by 2100. Several adaptation concepts were discussed for this scenario. Ultimately, this information must be shared with local stakeholders and discussed before any locally preferred decisions can be made on what adaptive actions would be appropriate for the community and when would be the best time to implement them.

The process shown is broadly applicable to bridges across the country where sea level rise has an influence. Bridges over navigable channels would need to be investigated for clearance reductions due to sea level rise and determine if any remedial action would need to be implemented. Ultimately, this effort is best handled at a planning level in a coordinated manner amongst all bridges along a shipping channel: it makes little sense to adapt one bridge to accommodate sea level rise if other bridges along the channel are not adapted as well and may impede access. It is recommended that such analyses be undertaken for navigable waterways across the country that are subject to sea level rise and have bridges that may impede maritime commerce.

4.4.3 Bridge Approach Embankment Exposure to Sea Level Rise – US 90/98 Tensaw-Spanish River Bridge (Western Approach)

Introduction

Sea level rise, irrespective of storm surge, is a potential threat not only to low-lying coastal roadways at risk of permanent inundation but also to more elevated roadways on embankments in the coastal zone where increased wave heights and energies impacting embankments can cause scouring and erosion. Continual inspection and maintenance is required to prevent the scouring and erosion that can result in significant loss of embankment material, loss of pavement sections, or ultimately, the complete breach of a roadway embankment.

> **Case Study Highlights**
>
> **Purpose:** Evaluate whether a tidally-influenced bridge approach embankment can withstand the increased heights of waves due to sea level rise. Overtopping of roadway and erosion impacts were considered.
>
> **Approach:** The height at which waves would impact the embankment was calculated to determine how high the riprap slope protection would need to be, and then evaluated against the estimated size of the current riprap slope protection.
>
> **Findings:** If adequately maintained, the current riprap slope protection would likely provide adequate protection from wave impacts under the 30 cm and 75 cm by 2050 sea level rise scenarios, although temporary inundation of the approach road could occur. Under the 200 cm by 2100 scenario, the existing riprap protection is not adequate to protect against wave impacts, and permanent overtopping of the roadway could occur.
>
> **Viable Adaptation Options:**
>
> - Ensure existing riprap protection is well maintained
> - Extend the current riprap slope protection
> - Raise the elevation of the riprap slope protection, approach, and bridge
>
> **Other Conclusions:** Problems associated with sea level rise would likely occur late in the design life of the approach. Therefore, it may be cost-effective to wait until a replacement is due before making major structural changes.

This case study assesses whether a tidally influenced approach roadway leading to the west abutment of the US 90/98 Tensaw-Spanish River Bridge can withstand changes in wave energy impacts as a result of various projected sea level rise scenarios. Specifically, the effect of increased wave heights from sea level rise on required embankment protection and roadway overtopping are studied. This case study does not analyze increased wave heights in combination with storm surge impacts. A complete analysis for embankment vulnerability would require analysis of both wave impacts and storm surge impacts. In addition, this study only analyzes a portion of the approach road to the embankment. Potential flooding as a result of sea level rise with or without storm surges should also be analyzed to determine if roadway inundation does occur and to what extent.

The assessment, limited as it was, determined that one of the three sea level rise scenarios tested would lead to permanent inundation of the approach roadway and would require adaptation actions such as raising the embankment elevation and enhancing the rip-rap protection measures (the existing roadway elevation and rip-rap measures were adequate for the other two sea level rise scenarios). However, as sea level rise is a gradual phenomenon and two of the three scenarios do not result in permanent inundation, it is recommended that no immediate action be taken towards this end and that the situation be monitored.

Although this case study focuses on an embankment approach to a bridge abutment with open water on both sides, the general analytical methods demonstrated here can also be applied to other coastal embankments, including causeways, coastal roadway embankments parallel to shorelines, or barrier island roads that are (or may become) subject to regular wave impacts due to increases in sea levels. Wave impact analysis at bridge embankments is a particularly complicated process, and it is recommended that it be performed by a qualified coastal engineer. The serviceable life of the facility should also be considered when analyzing impacts of sea level rise forecasts in the future. Comparing the expected remaining life of a bridge to the projected sea level elevation can help planners develop their approach to adaptation as discussed in Step 6, “Identify Adaptation Option(s)”, below.

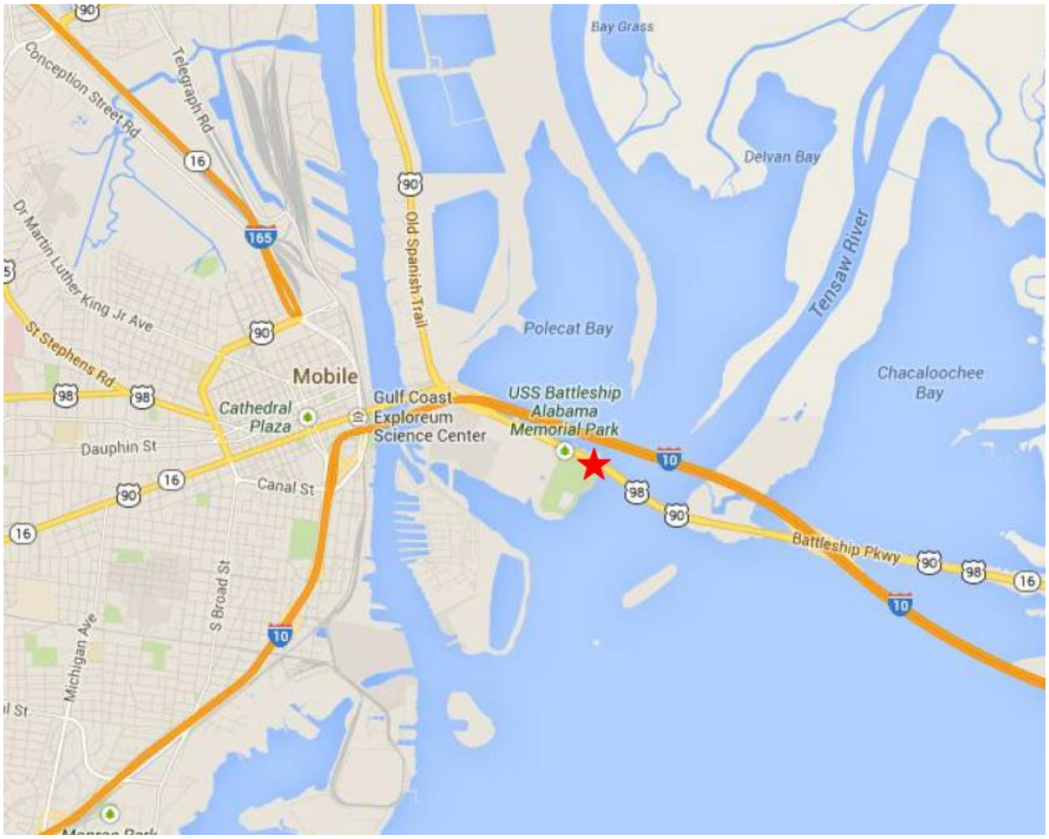
The sea level rise and wave analysis for the embankment was conducted using the 11 step *General Process for Transportation Facility Adaptation Assessments* and this serves as the organizing framework for this case study.

Application of the General Process for Transportation Facility Adaptation Assessments

Step 1 – Describe the Site Context

The case study approach roadway is located on Blakeley Island, just east of downtown Mobile, and acts as the western landing point for a bridge carrying US routes 90 and 98 (Battleship Parkway) over the mouth of the Tensaw and Spanish Rivers (see Figure 35). The bridge represents the beginning of the Battleship Parkway Causeway (the “Causeway”), a combination of causeways and bridges allowing US 90/98 to traverse the nearly eight mile (12.5 kilometer) wide tidally influenced Mobile Bay. The Causeway was the first route across the Bay and was followed later by the bridge carrying the Jubilee Parkway (I-10), commonly referred to as the “Bayway.” Today, the Causeway provides an important alternate route across Mobile Bay in the event of an incident that limits the use of I-10. It is also a key link serving the USS Alabama Battleship Memorial Park and commercial businesses on the islands along the Causeway. The subject approach roadway is located on the northern end of the Battleship Memorial Park, about one mile (1.6 kilometers) east of the interchange between US 90/98 and I-10 (Exit 27).

Figure 35: Location of the US 90/98 Tensaw-Spanish River Bridge within the Mobile Metropolitan Area[120]



Step 2 – Describe the Existing Facility

The US 90/98 Tensaw-Spanish River Bridge has separate eastbound and westbound spans: the westbound (northern) span was built in 1963 and the eastbound (southern) span was built in 1995. The bridge is 42.8 feet (13 meters) wide[121] and approximately 1,426 feet (435 meters) long from the western abutment on Blakeley Island to the eastern abutment. A total of 12 bents,[122] each one assigned a unique identifying number, support the bridge. The bents are numbered sequentially from Bent 1 at the west abutment to Bent 12 at the east abutment.

An aerial and topographic view of the western approach roadway and abutment can be seen in Figure 36 and an oblique aerial photo in Figure 37. A plan (overhead) view from ALDOT can be seen in Figure 38, while profile (side) views are shown in Figure 39 and Figure 40.

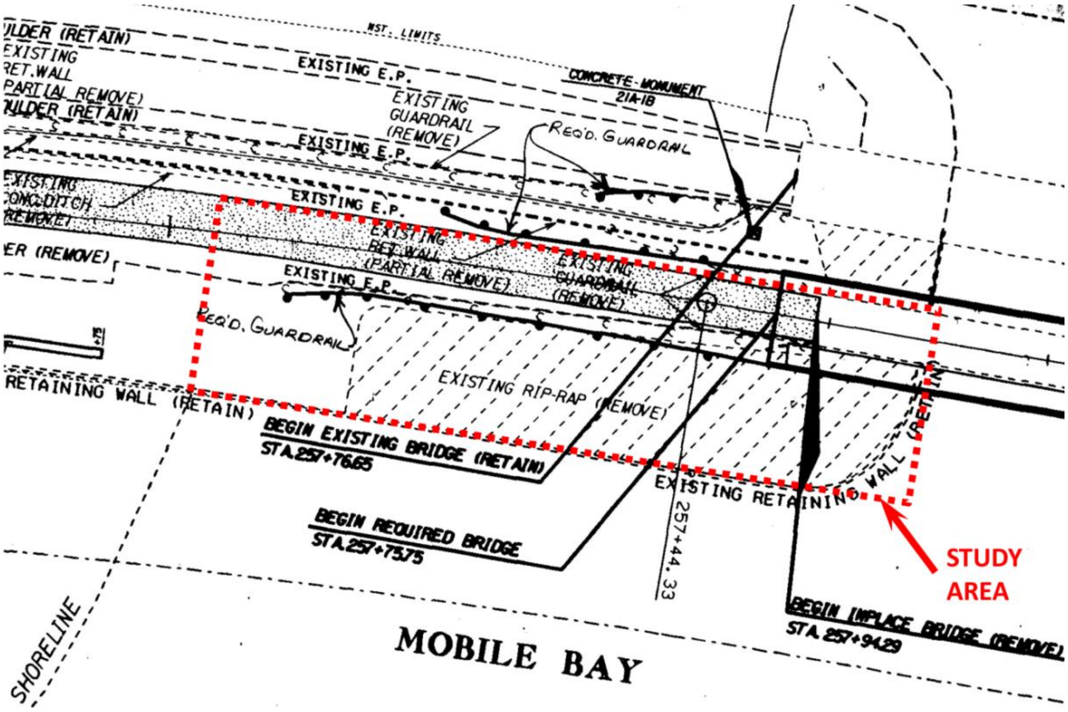
[120] Source of base map: Google Maps (as modified)
[121] Width as measured to the outside faces of the parapets. The parapets are the outside walls on either side of a bridge that are designed to prevent vehicles from careening off the structure.
[122] Bents, also known as piers, are the vertical columns supporting each bridge span along with the horizontal member, called a cap, which holds them together.

Figure 36: Aerial Image and Topography of the West Abutment of the US 90/98 Tensaw-Spanish River Bridge[123]

Figure 37: Oblique Aerial Image of the South Side of the US 90/98 Tensaw-Spanish River Bridge West Abutment[124]

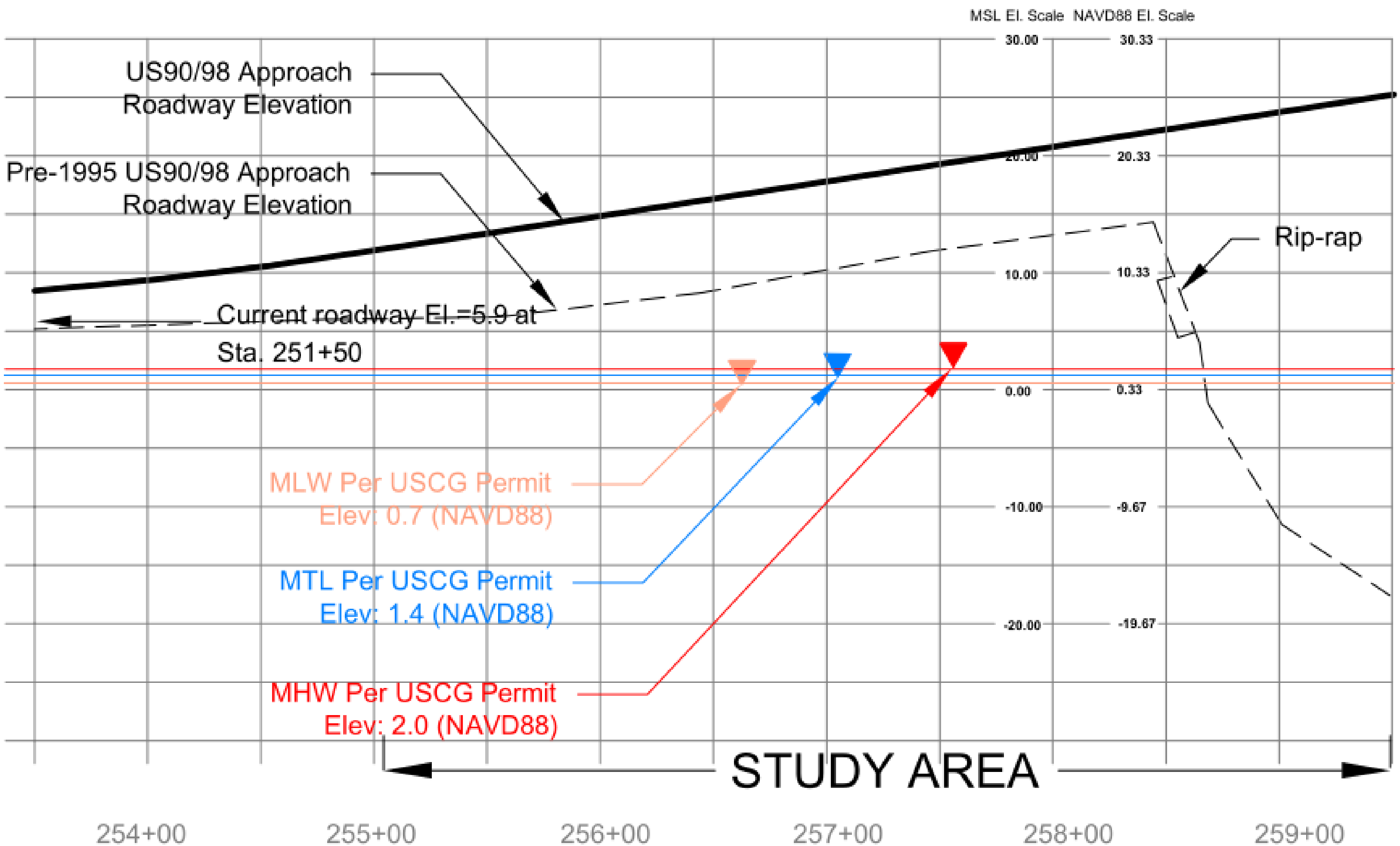
[123] Source of base map data: City of Mobile GIS. Note: All elevations shown are in relation to the North American Vertical Datum of 1988 (NAVD88).
[124] Source of image: Google Maps

Figure 38: Plan of the West Abutment of the US 90/98 Tensaw-Spanish River Bridge Showing the Area of Analysis[125]



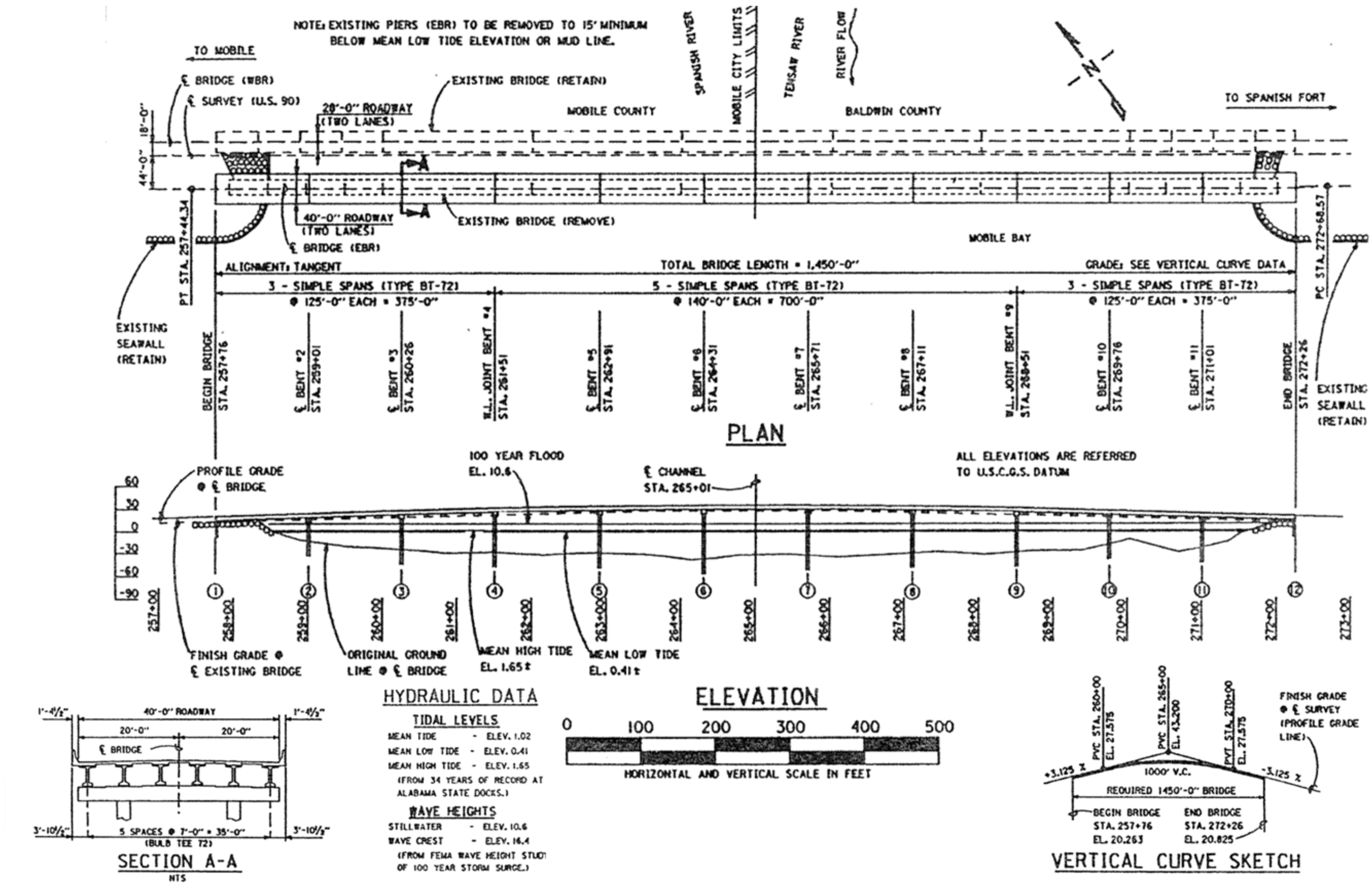
[125] Source: ALDOT, 1994 (as modified). Note: Figure obtained from the original 1995 construction drawings. All elevations shown refer to mean sea level. Mean sea level is the average of the water elevations recorded at each hour of the day over the current National Tidal Datum Epoch. The tidal epoch is the specific 19-year period over which NOAA uses to obtain observations that are used to develop tidal statistics.

Figure 39: Profile of the Western Approach Roadway to the US 90/98 Tensaw-Spanish River Bridge[126]



[126] Source: ALDOT, 1994 (as modified). Note: Figure obtained from the original 1995 construction drawings. All elevations shown refer to mean sea level. Mean sea level is the average of the water elevations recorded at each hour of the day over the current National Tidal Datum Epoch. The tidal epoch is the specific 19-year period over which NOAA uses to obtain observations that are used to develop tidal statistics.

Figure 40: Plan and Elevation of US 90/98 Tensaw-Spanish River Bridge[127]



[127] USCG, 1995. Note: All elevations shown refer to mean sea level. Mean sea level is the average of the water elevations recorded at each hour of the day over the current National Tidal Datum Epoch. The tidal epoch is the specific 19-year period over which NOAA uses to obtain observations that are used to develop tidal statistics.

The case study analysis will focus on wave impacts on the western approach roadway to the bridge, specifically the south side of the embankment as this is the location susceptible to wind-generated wave impacts. The west embankment of the Tensaw-Spanish River Bridge extends approximately 300 feet (91 meters) westward from the water's edge under the span. This studied segment of the bridge approach is highlighted in the plan view of the structure shown in Figure 38 and the elevation shown in Figure 39.

The approach roadway in the study area begins at an elevation of 5.9 feet[128] (1.8 meters), approximately 725 feet (221 meters) west of the abutment. The roadway increases in elevation as it reaches the bridge. The western end of the eastbound span is at an elevation of 20.9 feet (6.4 meters) and rises at a grade of 3.1% towards the center of the span (see Figure 39). The south side of the west abutment slopes down to a bulkhead with the top at an approximate elevation of 5.5 feet (1.7 meters). The maximum slope of the southern embankment was found to be 4 horizontal to 1 vertical. The south side of the west approach roadway and abutment currently is reinforced with riprap[129] revetment[130] with a D_{50}[131] of approximately 15 inches (38.1 centimeters) in size and a thickness of 3 feet (0.9 meters). The elevation of the seabed of Mobile Bay directly in front of the abutment is -0.7 feet (-0.2 meters).

Step 3 – Identify Climate Stressors That May Impact Infrastructure Components

Changes in wave impacts due to sea level rise are the primary climate change related environmental factors considered in this study. However, sea level rise and its saturation effect on soil stability above the current sea level elevation should be a consideration in the design of future facilities and the inspection of existing facilities. Storm surge based erosion and scour at the bridge abutment is another key environmental factor that should be considered in the design and protection of the approach roadway. Section 4.4.4 of this report provides a detailed study of this climate stressor for the same case study bridge abutment.

Step 4 – Decide on Climate Scenarios and Determine the Magnitude of Changes

Three sea level rise values were considered for this case study based on projected climate changes and land movements in Mobile: one foot (0.3 meters), 2.5 feet (0.8 meters), and 6.6 feet (two meters). The one foot (0.3 meter) scenario is projected to occur around 2050 while the other two scenarios capture the range of possible future changes by 2100 (uncertainty on sea levels increases further into the future). For more information on how these scenarios were developed, please refer to the *Climate Variability and Change in Mobile, Alabama* report.[132] See Step 4 in

[128] All elevations in the text of this document are in NAVD88.
[129] Riprap consists of loose stone placed in a manner to provide erosion protection or armoring over a soil area.
[130] Revetments are, "a layer or facing of rock, dumped or hand-placed to prevent erosion, scour, or sloughing of a structure or embankment" (FHWA, 1989).
[131] Dx denotes rock gradation percentages for which the distribution of individual stones will have diameters of X percentage of the sample batch smaller than the stated measurement value. For example, for D_{50} = 10 inches (254 millimeters), 50% of the rocks in the batch will have diameters smaller than 10 inches (254 millimeters) and 50% will have diameters greater than 10 inches (254 millimeters).
[132] USDOT, 2012

Section 4.4.2 for a discussion of how sea level rise scenarios can be generated for locations outside of the Mobile region.

The revetment riprap size and location required to protect an approach roadway against wave impacts are determined by the wave heights. These may be a combination of local wind-generated waves and ocean waves based on the amount of wave energy and associated wave height remaining after the wave transformation processes between the coast and the site's location within the bay. The largest waves that can impact the embankment may be limited by the available water depth directly in front of the embankment. This wave height, known as the depth-limited wave height, will likely increase as sea levels rise, resulting in deeper water that is capable of sustaining larger waves. During long term sea level rise, sedimentation could preclude the development of significantly deeper water depths at a given location, but for this analysis it is assumed that water depth increases with sea level rise.

As water depth increases there may also be a point at which the depth no longer limits wave height at the structure. This could be the case at a particular structure if the design wave height is less than the depth-limited wave height condition. Wave heights are primarily dependent on water depth, wind speed, and fetch.[133] In the open ocean or under surge conditions, water depth may increase to a point where the wind speed and fetch under a given design scenario are the limiting factors instead of the depth-limited wave conditions. In these conditions, design wave heights should either be calculated using guidance from the *Coastal Engineering Manual*[134] or determined through detailed modeling. For this analysis, it is assumed depth-limited wave height conditions prevail for the various sea level rise scenarios considered.

For this study, a comparison of the riprap slope protection required to protect against depth-limited waves was made for current conditions along with the three sea level rise values previously discussed. Depth-limited, wind-generated waves without storm surge influences were chosen for this analysis to provide a clear view on how sea level rise can affect the magnitude of wave impacts without coupling them with other factors such as storm surges. A depth-limited wave was chosen because they are the largest waves that can occur at a location for a given depth of water. For this study, two types of depth-limited wave heights were calculated: (1) breaking wave heights detailing the highest point a wave reaches prior to breaking and (2) the significant wave height[135] of incident waves at the toe of a structure. The impact of sea level rise on the depth-limited wave heights in front of the west approach roadway of the Tensaw-Spanish River Bridge can be found in Table 20.

[133] FHWA, 2004. Fetch refers to the area over water where the wind is unobstructed with fairly uniform speed and direction.
[134] USACE, 2012
[135] Significant wave height is the average height of the one-third largest waves for a specific set of wind, fetch and water depth conditions.

Table 20: Sea Level Rise Impacts on Depth-Limited Wave Heights along the Western Approach Roadway to the US 90/98 Tensaw-Spanish River Bridge

Sea Level Rise	MHW Elevation[136] Feet (Meters)	Water Depth in Front of Embankment Feet (Meters)	Depth-Limited Breaking Wave Height[137] Feet (Meters)	Depth-Limited Significant Wave Height[138] Feet (Meters)
None	2.0 (0.6)	2.7 (0.8)	2.1 (0.6)	1.6 (0.5)
One foot (0.3 Meter)	3.0 (0.9)	3.7 (1.1)	2.8 (0.9)	2.2 (0.7)
2.5 Foot (0.8 Meter)	4.5 (1.4)	5.2 (1.6)	4.0 (1.2)	3.1 (0.9)
6.6 Foot (2.0 Meter)	8.6 (2.6)	9.3 (2.8)	7.2 (2.2)	5.6 (1.7)

Step 5 – Assess Performance of the Existing Facility

The progression of sea level rise over the next several decades and its impact on sustaining taller waves could present challenges for maintaining the functionality of the roadway embankment. To design for riprap slope protection, the size of the stone used for protection was determined. Then, the height at which the crashing waves impact the embankment was calculated to determine how high the slope protection needs to be.

<u>Riprap Slope Protection Sizing</u>

For the purpose of this study, the size of the riprap slope protection required to armor against taller waves was analyzed. In addition, the height of wave run-up along the embankment was analyzed under each scenario. Specifically, for each of the three sea level rise values along with current conditions, the medium mass of rock (M_{50})[139] required to withstand wave impacts was determined using the Hudson equation[140] from the U.S. Army Corps of Engineers (USACE) *Coastal Engineering Manual*.[141] The Hudson equation is as follows:

$$M_{50} = \frac{\rho_S H^3}{K_D \left(\frac{\rho_S}{\rho_W} - 1\right)^3 cot\alpha}$$

[136] Mean High Water (MHW) is the average of all high tide elevations during the day over the current National Tidal Datum Epoch. The tidal epoch is the specific 19-year period over which NOAA uses to obtain observations that are used to develop tidal statistics. The MHW elevation was taken from the ALDOT bridge plans (ALDOT, 1994) and adjusted to the NAVD88 vertical datum.
[137] Breaking wave heights are calculated as 78% of the water depth directly in front of the study area (USACE, 2002).
[138] Significant wave heights are calculated as 60% of the water depth at the toe of the structure (USACE, 2002).
[139] The medium mass of rock refers to the average riprap stone size required to provide armoring for slope stability.
[140] Hudson, 1974
[141] USACE, 2002

Where,

H Characteristic wave height[142]
M_{50} Medium mass of rocks
ρ_S Mass density of rocks[143]
ρ_W Mass density of water[144]
α Slope angle[145]
K_D Stability coefficient (a value of two for this case study) [146]

Once the mass of rock was determined, the equivalent cube length of rock, which is the minimum size of rock required to provide armor slope stability, was calculated. The equivalent cube length of rock was determined using the following equation:[147]

$$D_{n50} = \left(\frac{M_{50}}{\rho_S}\right)^{1/3}$$

Where,

D_{n50} Equivalent cube length of median rock
M_{50} Medium mass of rocks
ρ_S Mass density of rocks[148]

Table 21 provides the standard riprap class sizes from ALDOT's *Standards and Specifications for Highway Construction*. Class sizes from this table are chosen based upon the medium mass and equivalent cube length of rock calculated. Table 22 provides a summary of the required mass, equivalent cube length, and selected riprap class for wave impacts for depth-limited breaking waves under current conditions and the three sea level rise scenarios at the embankment.

Under current conditions, the riprap size required to provide slope protection is Class 1, the smallest size provided in Table 21. The current riprap slope protection at the site is estimated to be Class 3. Under the three sea level rise scenarios, as the depth-limited breaking wave height increases, the class of riprap slope protection needed increases as well. See Table 21.

[142] The characteristic wave height is the depth-limited breaking wave height for the purposes of this study.
[143] Assumed to be 165 pounds per cubic foot (2,643 kilograms per cubic meter) for this case study
[144] 64 pounds per cubic foot (one tonne per cubic meter)
[145] 14° based on a 4/1 horizontal to vertical slope
[146] Based on rough, angular stone for breaking waves where characteristic wave height is equal to $H_{1/10}$ wave height (USACE, 1984)
[147] Hudson, 1974
[148] Assumed to be 165 pounds per cubic foot (2.6 tonnes per cubic meter) for this case study.

Table 21: ALDOT Standard Riprap Classes[149]

Riprap Class	Weight Range – D_{10} to D_{90} Pounds (kilograms)	D_{50} Size	
		Pounds (kilograms)	Inches (millimeters)
1	10 – 100 (4.5-45.4)	50 (22.7)	10 (254)
2	10 – 200 (4.5-90.7)	80 (36.3)	12 (305)
3	25 – 500 (11.3-227)	200 (90.7)	15 (381)
4	50 – 1,000 (22.7-454)	500 (227)	22 (559)
5	200 – 2,000 (227-907)	1,000 (454)	28 (711)

Table 22: Revetment Riprap Size Calculation Results to Withstand Wave Impacts at the Western Approach Roadway to the US 90/98 Tensaw-Spanish River Bridge

Projected Sea Level Rise	Depth-Limited Breaking Wave Height, H Feet (Meters)	Medium Mass of Rocks, M_{50} Pounds (Kilograms)	Equivalent Cube Length of Rocks, D_{50} Inches (Millimeters)	Riprap Class
None	2.1 (0.6)	46 (21.0)	8 (203)	1
One foot (0.3 Meter), Yr. 2050	2.8 (0.9)	121 (54.9)	11 (279)	2
2.5 Foot (0.8 Meter), Yr. 2100	4.0 (1.2)	340 (154.3)	15 (381)	3
6.6 Foot (2.0 Meter), Yr. 2100	7.2 (2.2)	1,971 (894.0)	27 (686)	5

Wave Run-Up Calculations for Slope Protection

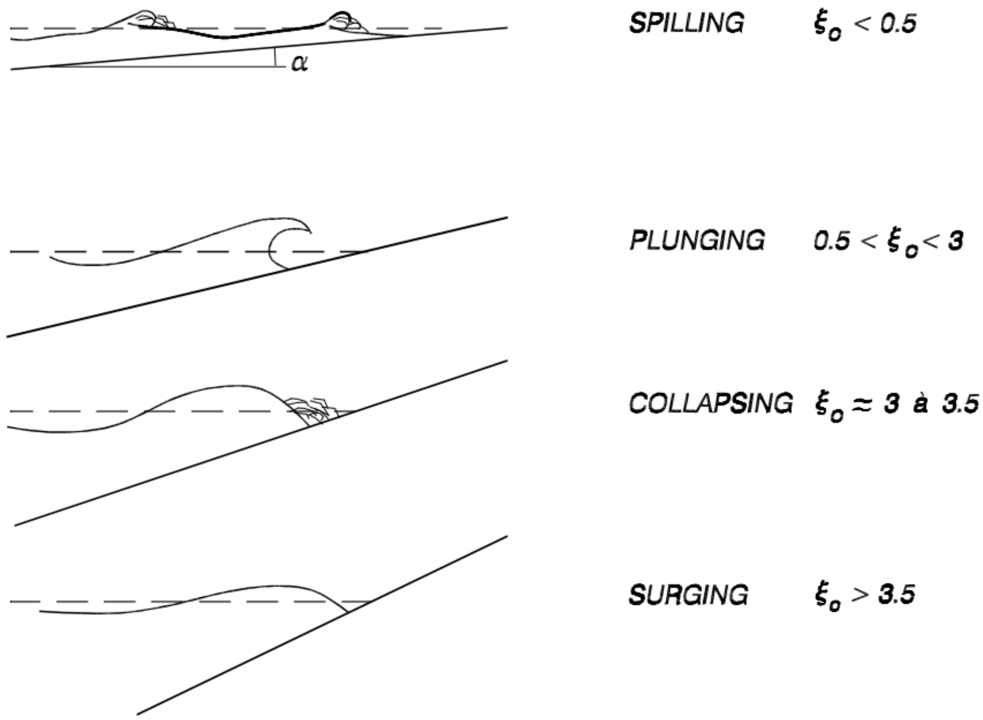
The next portion of the case study involves determining the height at which protection is needed to withstand wave impacts. Wind-generated waves and their associated peak wave periods[150] cause waves to break on sloping structures. The heights of the embankment that are susceptible to the crests and troughs of breaking waves are known as run-up (R_u) and run-down (R_d) heights, respectively. For the purposes of this case study, run-down and wave impacts to the toe

[149] ALDOT, 2012
[150] Peak wave periods refer to the time period between the most energetic waves in the total wave spectrum at a specific point.

of the embankment were not factored in because of the presence of an existing bulkhead. In cases where a bulkhead is not present, run-down and toe impacts would need to be calculated to determine the length down the embankment slope that protection would be needed.

Run-up and run-down are dependent on surf-similarity pattern or the Iribarren number[151] (ξ) which identifies the type of breaking wave. Breaking wave types identified through calculation of the Iribarren number are illustrated in Figure 41.

Figure 41: Types of Breaking Waves on an Impermeable Slope[152]



For the purposes of this study, irregular wave types were selected to represent a natural sea state with variability as opposed to a "monochromatic" (i.e., regular) wave pattern. As the Iribarren number increases, wave run-up height also increases. For example, "surging" waves, with an Iribarren number greater than 3.5, would generate higher run-up values at a particular embankment than the other wave types shown in Figure 41.

For irregular waves, the Iribarren number is calculated as shown in the following equation:[153]

$$\xi_{om} = \frac{\tan\alpha}{\sqrt{s_{om}}}$$

[151] Battjes, 1974
[152] USACE, 2002 (Table VI-5-1)
[153] USACE, 2002

Where,

ξ_{om} Iribarren number for mean wave period

α Slope angle[154]

s_{om} Mean wave steepness for mean wave period

To calculate the Iribarren number, the mean wave steepness was first calculated. The mean wave steepness is the ratio between the depth-limited significant wave height and the mean wave length and can be determined through the following formulae:

$$s_{om} = {}^{H_S}\!/_{L_{om}} = {}^{2\pi}\!/_{g} \times {}^{H_S}\!/_{T_m^2}$$

Where,

s_{om} Mean wave steepness

H_S Depth-limited significant wave height

L_{om} Mean wave length

g Acceleration due to gravity[155]

T_m Mean wave period ($= 0.87 \times T_p$)

Where,

T_p Wave period corresponding to the peak of the wave spectrum[156]

Calculation of the peak wave periods is described in detail in Part II-2-2 of the *Coastal Engineering* Manual.[157] The goal of this study is to evaluate the wave impact with various SLR scenarios. The wind-induced wave condition is considered the controlling scenario for design of the revetment as it produces a more erosive state than typical waves which would not be expected to be the controlling conditions for this specific case. For this study, the peak wave period was obtained from the Steady State Spectral Wave (STWAVE) model results for storm surge related wave conditions, since standalone modeling of extreme wind event wave conditions was not modeled for non-surge storm conditions as part of this study. The storm surge scenario used to obtain the peak wave period was the Hurricane Katrina Base Case Scenario as detailed in Section 4.4.4 . The storm surge peak wave period was selected for this study since waves at this location were depth-limited during the surge, which parallels the assumptions of depth limited wave for the non-surge extreme wind conditions. If wave modeling data is not available, the peak wave period would need to be calculated in accordance with the *Coastal Engineering Manual.*[158]

[154] 14° based on 4/1 horizontal to vertical slope
[155] 32.2 feet per second squared (9.8 meters per second squared)
[156] This period is the time between peak waves of the wave spectrum (derived from the STeady State spectral WAVE model [STWAVE] employed on this project).
[157] USACE, 2002
[158] USACE, 2002

Once the Iribarren number is calculated, determining the wave run-up height is the next step of the process. The wave run-up value at the two percent probability of exceedance level ($R_{u2\%}$) on an impermeable rock slope[159] was calculated using the following equation:[160]

$$\frac{R_{u2\%}}{H_S} = \begin{cases} A \times \xi_{om} & for\ 1.0 < \xi_{om} \leq 1.5 \\ B(\xi_{om})^C & for\quad \xi_{om} > 1.5 \end{cases}$$

Where,

$R_{u2\%}$ Wave run-up value at the two percent probability of exceedance level
H_S Depth-limited significant wave height
ξ_{om} Iribarren number for mean wave period
$A_{2\%} = 0.96$
$B_{2\%} = 1.17$
$C_{2\%} = 0.46$

The $A_{2\%}$, $B_{2\%}$, and $C_{2\%}$ values are coefficients that vary based upon the permeability of the embankment slope. For various slope types, the coefficients can be determined using Figure VI-5-11 and Table VI-5-5 in the *Coastal Engineering Manual*.[161]

The 2% wave run-up value represents the run-up height with a probability of occurrence being twice for every 100 waves. Presently, this is the most common run-up parameter used to determine the height of riprap slope protection. Table 23 provides the results of the run-up height and elevation calculations for determination of the vertical extent of wave impacts and revetment riprap protection placement. Figure 42 provides a representation of the difference between Surface Water Levels (SWL) and the run-up crest elevation. For the purposes of this study, SWL is assumed to be the Mean High Water (MHW)[162] level in each of the scenarios.

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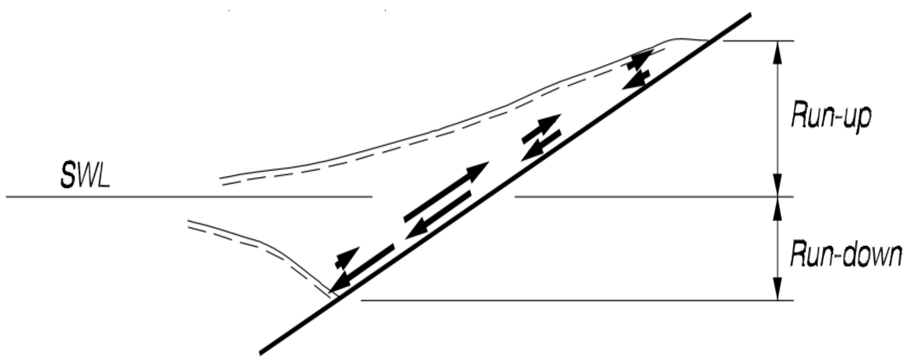
[159] Impermeable slopes represent embankments with impermeable (asphalt, concrete, etc.) surfaces and fine core materials that limit porosity. Permeable slopes would represent rubble-mound structures (USACE, 2002).
[160] USACE, 2002 (Equation VI-5-12). Also see Delft Hydraulics, 1989.
[161] USACE, 2002
[162] Mean High Water (MHW) is the average of all high tide elevations during the day over the current National Tidal Datum Epoch. The tidal epoch is the specific 19-year period over which NOAA uses to obtain observations that are used to develop tidal statistics.

Table 23: Revetment Riprap Run-up Elevation Calculation Results at the US 90/98 Tensaw-Spanish River Bridge Western Approach Roadway Embankment

Sea Level Rise	MHW Elevation Feet (Meters)	Run-up Height ($R_{U2\%}$) Feet (Meters)	Run-up ($R_{U2\%}$) Crest Elevation[163] Feet (Meters)
None	2.0 (0.6)	3.1 (0.9)	5.1 (1.6)
One foot (0.3 Meter)	3.0 (0.9)	3.9 (1.2)	6.9 (2.1)
2.5 Foot (0.8 Meter)	4.5 (1.4)	5.1 (1.6)	9.6 (2.9)
6.6 Foot (2.0 Meter)	8.6 (2.6)	8.1 (2.5)	16.7 (5.1)

Figure 42: Diagram of Surface Water Level versus Run-up[164]



Currently, with no sea level rise, wave run-up is halted only 0.4 feet (0.1 meters) above the bulkhead and does not reach the approach roadway elevation of 5.9 feet (1.8 meters) located approximately 725 feet (221 meters) west of the span. Under the projected 2050 one foot (0.3 meter) sea level rise scenario, the wave run-up reaches an elevation of 6.9 feet (2.1 meters), which does not overtop the embankment but will most likely lead to temporary inundation of the approach road. As waves subside, the temporary inundation will in turn subside. Under the projected 2100 2.5 feet (0.8 meter) sea level rise scenario, the wave run-up reaches an elevation of 9.6 feet (2.9 meters). Again, under this scenario, temporary inundation of the approach roadway will occur but will subside. Under the highest sea level rise 2100 projected scenario of 6.6 feet (two meters), wave run-up comes within 2.5 feet (0.8 meters) vertically of the beginning of the bridge span. In this scenario, overtopping does occur along the approach roadway beginning at a location 80 feet (24.4 meters) west of the embankment. In this scenario, the MHW

[163] Elevation where 2% of wave run-up heights reach.
[164] Note: For the purposes of this study, Surface Water Level (SWL) in the diagram represents MHW (USACE, 2002).

elevation will create a permanent inundation of the approach roadway. Figure 43 and Figure 44 illustrate the MHW level and calculated run-up heights and their impact upon the embankment and approach roadway in plan and profile views, respectively.

As previously noted, the current riprap protection in place on the project is estimated to be Class 3 riprap (D_{50} = 18 inches [45.7 centimeters]) with a thickness of 3 feet (0.9 meters) and has been place on the embankment slope from the bulkhead to elevation 10.3 feet (3.1 meters). Table 24 provides a summary of the required size of the riprap slope protection and height of wave run-up at the case study embankment. The existing riprap slope protection meet the design standards under the 2050 one foot (0.3 meter) and the 2100 2.5 feet (0.75 meter) projected sea level rise scenarios. In the 2100 6.6 feet (3.0 meters) sea level rise scenario, the existing riprap slope protection is neither large enough or at a high enough elevation to protect against wave impacts.

Figure 43: Plan of the US 90/98 Tensaw-Spanish River Bridge Approach Roadway and Western Embankment with MHW and 2% Run-up Water Levels

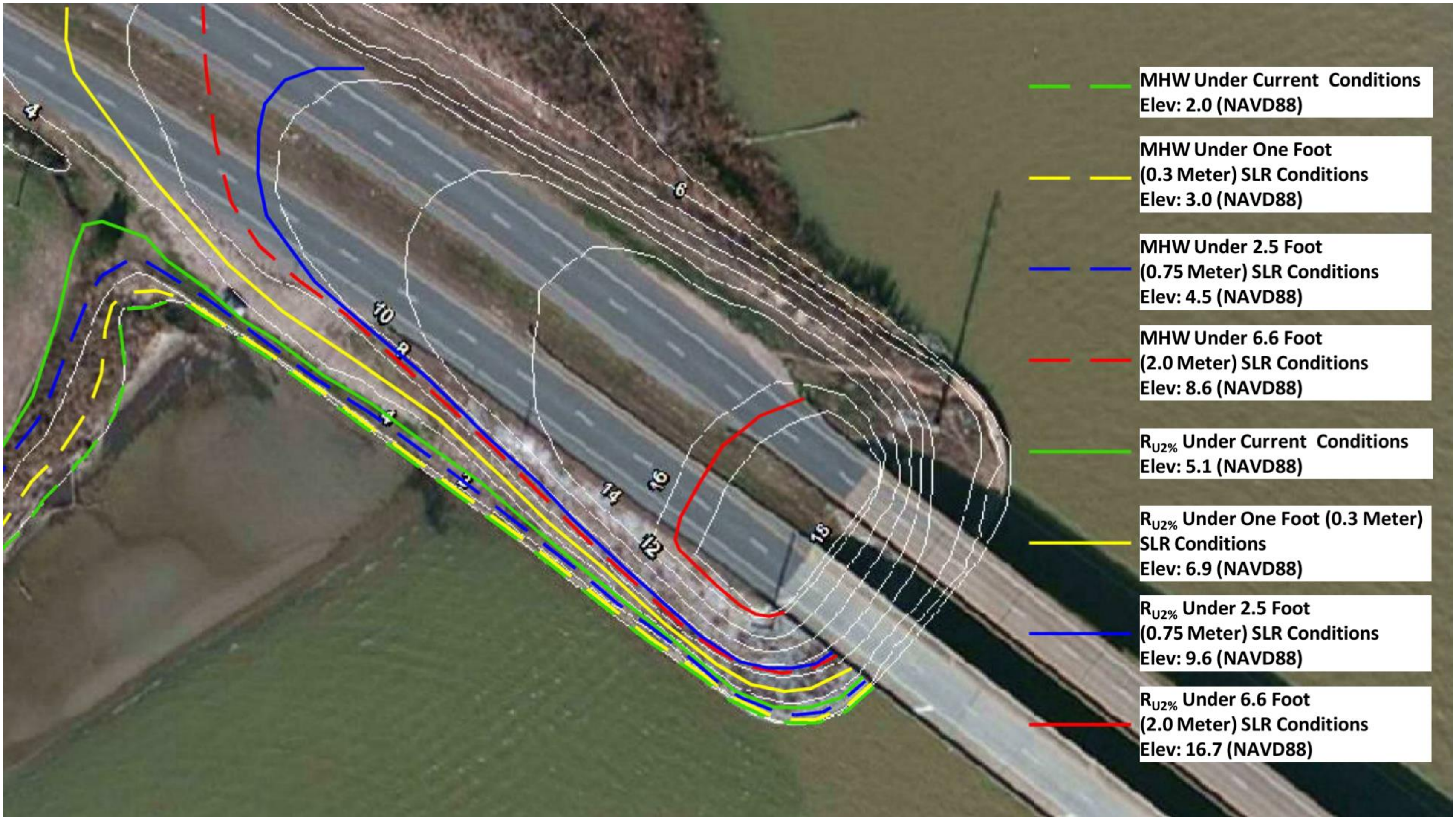
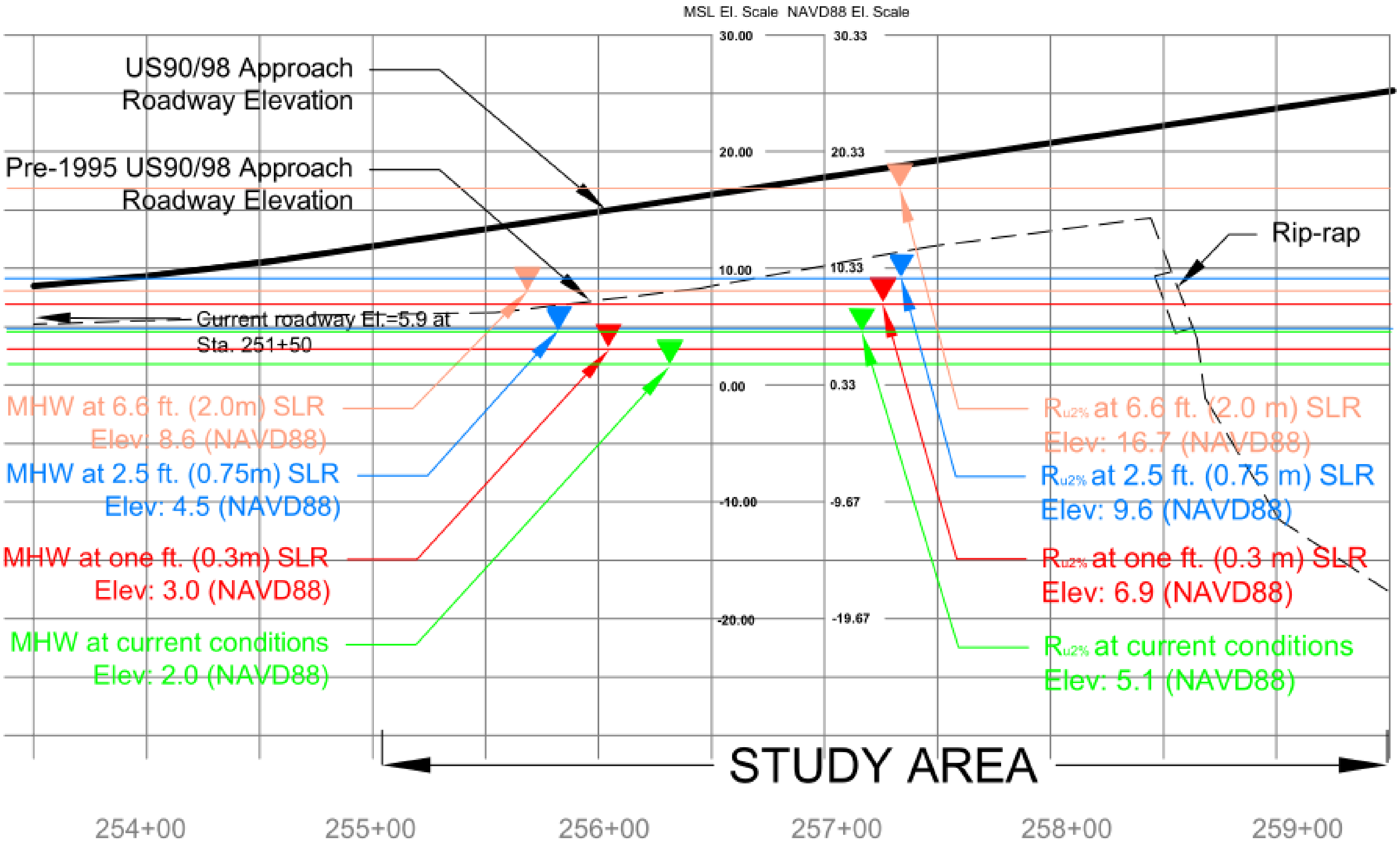


Figure 44: Profile of the US 90/98 Tensaw-Spanish River Bridge Approach Roadway and Western Embankment with MHW and 2% Run-up Water Levels[165]



[165] Source: ALDOT, 1994 (as modified). Note: All elevations shown refer to mean sea level. Mean sea level is the average of the water elevations recorded at each hour of the day over the current National Tidal Datum Epoch. The tidal epoch is the specific 19-year period over which NOAA uses to obtain observations that are used to develop tidal statistics.

Table 24: Summary of Revetment Riprap Size and Location Calculation Results at Western Approach Roadway to the US 90/98 Tensaw-Spanish River Bridge

Sea Level Rise	Medium Mass of Rocks, M_{50} Pounds (Kilograms)	Equivalent Cube Length of Rocks, D_{50} Inches (Meters)	Run-up ($R_{U2\%}$) Crest Elevation Feet (Meters)	Roadway Inundation Level
None	46 (21.0)	8 (0.2)	5.1 (1.5)	None
One foot (0.3 Meter)	121 (54.9)	11 (0.3)	6.9 (2.1)	Temporary from wave run-up
2.5 Foot (0.8 Meter)	340 (154.3)	15 (0.4)	9.6 (2.9)	Temporary from wave run-up
6.6 Foot (2.0 Meter)	1,971 (894.0)	27 (0.7)	16.7 (5.1)	Permanent from mean high water

As seen in Table 24, as sea levels rise and water depths in front of the embankment increase, the height of waves that can be sustained without breaking prior to impacting the embankment increase, consequently increasing the weight and dimensions of the riprap needed to protect against them. In addition, as the wave size increases, the height at which the run-up impacts the embankment and approach roadway increases as well.

It should be noted that although this case study provides an examination of the consequences that sea level rise will have on wind-generated wave heights and wave impacts to a bridge approach embankment, several other factors should be considered when analyzing potential threats to stability and functionality of a bridge approach in a tidal area. As previously mentioned, Section 4.4.4 provides a case study on the impacts of storm surge to bridge abutments, specifically, how flow velocities due to storm surges at a bridge abutment can cause scour impacts, and what adaptations should be made to respond to these impacts.

In addition, although this study looked only at the gradual impacts of a general sea level rise in association with climate change, the consequences of increased wave action due to storm surges should also be considered when analyzing wave impacts on embankments. It is very likely that wave heights that do not overtop the embankment in this case study may indeed overtop the roadway when considering storm surge impacts. In this case, there is a much higher risk of permanent loss of functionality of the roadway.

Step 6 – Identify Adaptation Option(s)

Under current conditions and the 2050 one foot (0.3 meter) and the 2100 2.5 feet (0.8 meter) sea level rise scenarios considered in this case study, the current riprap slope protection will provide adequate protection from wind-generated wave impacts if properly maintained. Under the 6.6

feet (two meter) seal level rise scenario, adaptations to protect the embankment slope and prevent water from accumulating on the western approach roadway would be required to maintain functionality of the existing bridge. To withstand wave run-up and impact under this scenario, placement of riprap slope protection at a size and elevation as shown in Table 24 would be required.

Under the 6.6 feet (two meter) seal level rise scenario, raising the height of the roadway will limit or eliminate the risk and impact of flooding along the bridge's western approach. Under the three sea level rise scenarios considered, the bridge itself was not overtopped but the approach roadway located west of the abutment was susceptible to flooding. Under the projected 2100 6.6 feet (two meter) sea level rise scenario, a portion of the embankment itself experiences overtopping. The following subsections provide specific adaptation options to meet the scenarios considered.

<u>Option 1 – Provide Maintenance to Ensure Proper Function of Existing Riprap Slope Protection</u>

Under this option, no substantive changes would be made to the existing riprap slope protection. The only action under this option would be to ensure that the coverage currently designed for the area of Class 3 riprap of a 3 foot (0.9 meter) thickness up to elevation 10.3 feet (3.1 meters) is properly maintained.

<u>Option 2 – Raising the Elevation of the Riprap Slope Protection, Approach Road, and Bridge</u>

The projected year 2100 6.6 foot (two meter) sea level rise scenario would require raising the roadway and seawall to an elevation of 16.7 feet (5.1 meters) so that it would not be susceptible to regular tidal elevations or overtopping with wave run-up heights. This option would involve armoring the embankment with Class 5, 28 inch (711 millimeter) minimum riprap slope protection to an elevation at or above 16.7 feet (5.1 meters) along the entire southern face of the west embankment. This armoring layer would consist of a 58 inch (1,420 millimeter) thick layer of stone with a D_{50} of 28 inches (711 millimeters), a 14 inch (356 millimeter) layer of 6 inch (152 millimeter) stone, and an impermeable geotextile fabric.

<u>Option 3 – Extending the Embankment Slopes</u>

Another option to address impacts from the projected year 2100 6.6 foot (two meter) sea level rise scenario would be to extend the slope of the embankment to decrease the amount of wave height and run-up exposure. To prevent roadway overtopping of waves, the angle of the embankment slope would need to be lessened to 2.9 degrees or 20 horizontal to 1 vertical. This would halt run-up at the same elevation of the upper limits of the existing riprap slope protection, 10.3 feet (3.1 meters). The embankment would extend approximately 225 feet (69 meters) further away from the roadway into the Bay. This option would involve armoring the embankment with Class 4, 22 inch (559 millimeter) minimum riprap slope protection to an elevation at or above 10.3 feet (3.1 meters) along the entire southern face of the west embankment. This armoring layer would consist of a 44 inch (1,120 millimeter) thick layer of stone with a D_{50} of 22 inches (559 millimeters), an 11 inch (279 millimeter) layer of 5 inch (127 millimeter) stone, and an impermeable geotextile fabric.

It should be noted that any potential sea level rise issues on the embankment of the Tensaw-Spanish River Bridge will occur late in the design life of the bridge crossing. Given this, it might very well be the case that a full replacement of the existing bridge structure and approach roadways with a design that accounts for anticipated sea level changes will be a more cost-effective solution than retrofitting the existing structure.

Although not the focus of this case study, storm surge combined with sea level rise would need to be a consideration in any adaptive design. In the projected year 2100 6.6 feet (two meter) sea level rise scenario where wind-generated waves overtop the embankment and roadway completely, substantial protection would be needed to decrease the risk of damage to the roadway. In the case of the western approach to the Tensaw-Spanish River Bridge, significant surge combined with wind-generated wave impacts could potentially overtop the road. In this case, water can seep into the subgrade below the road surface and cause wave induced pressure penetration, which would dramatically increase the risk that the pavement is washed away and subgrade infrastructure (subbase material) is affected. In this case, more significant countermeasures would be required to mitigate this risk such as the installation of steel sheet piling. Other problems that can occur with roadway overtopping include wave induced slamming pressures caused by wave impacts to the underside of the bridge, where the abutment meets the bridge soffit (underside). Additional structural measures and protection could be required to ensure the integrity of the bridge structure in this location.

Step 7 – Assess Performance of the Adaptation Option(s)

Table 25 summarizes how well each of the proposed adaptation options performs under each of the storm surge scenarios. If these adaptation options actually were being considered for design, a full analysis quantifying the performance of each option under each scenario would need to be conducted and the results used in the economic analysis in Step 8.

Step 8 – Conduct an Economic Analysis

An economic analysis was not included in this case study but is recommended for facility-level adaptation assessments. See Section 4.4.1 for an example of how an economic analysis was applied to a culvert exposed to changes in precipitation due to climate change.

Table 25: US 90/98 Tensaw-Spanish River Bridge West Approach Embankment Adaption Performance Summary

Sea Level Rise	Impact Considered	Option 1	Option 2	Option 3
Current Conditions	Provides Embankment Protection	Yes	Yes	Yes
	Provides Overtopping Protection	Yes	Yes	Yes
	Type Of Inundation	None	None	None
Projected Year 2050 One foot (0.3 Meter) Sea Level Rise	Provides Embankment Protection	Yes	Yes	Yes
	Provides Overtopping Protection	Yes	Yes	Yes
	Type Of Inundation	Temporary	None	None
Projected Year 2100 2.5 Foot (0.8 Meter) Sea Level Rise	Provides Embankment Protection	Yes	Yes	Yes
	Provides Overtopping Protection	Yes	Yes	Yes
	Type Of Inundation	Temporary	None	None
Projected Year 2100 6.6 Foot (2.0 Meter) Sea Level Rise	Provides Embankment Protection	No	Yes	Yes
	Provides Overtopping Protection	No	Yes	Yes
	Type Of Inundation	Permanent	None	None

Step 9 – Evaluate Additional Decision-Making Considerations

Other important factors that might influence whether and how the approach roadway embankment is adapted to accommodate sea level rise include how land uses on Blakeley Island and the Causeway are impacted by sea level rise. If impacts are great enough to eliminate the need for vehicular access to these areas, then executing adaptations on both the I-10 roadway and the US 90/98 roadway may be redundant and unnecessary. In this case, it's possible that only one facility would be chosen and utilized in the future (likely I-10). The changing land uses may also have a significant impact on traffic volumes and the need for the bridge within the larger transportation network. In addition, there may be potential concerns with having a higher bridge adjacent to the USS Alabama Battleship Memorial Park, two ships in which are National Historic Landmarks. The entirety of the park is also on the Alabama Register of Landmarks and Heritage.

Step 10 – Select a Course of Action

As the facility currently meets design standards for riprap slope protection under current conditions as well as the 2050 one foot (0.3 meter) and the 2100 2.5 feet (0.75 meter) sea level rise scenarios, the recommended course of action is only to provide maintenance to ensure adequate protection along the embankment. It is advisable that the existing protection be inspected to ensure it meets the standard for a Class 3 riprap slope protection on an embankment to withstand the effects of wind-generated wave impacts under the project year 2100 2.5 foot (0.8 meter) sea level rise scenario. The 2100 2.5 foot (0.8 meter) sea level rise scenario is recommended because this is the most extreme scenario where permanent inundation of the approach roadway does not occur under water level variations due to tides (an impact that would require additional actions). The existing Class 3 riprap slope protection along the length of the embankment and up to elevation 10.3 feet (3.1 meters) should provide adequate protection on the embankment if installed according to guidance from the *Coastal Engineer Manual* and HEC-25. Further study should be done along the entire approach roadway to determine the impacts of sea level rise on the approach roadway further from the bridge than was considered in this study. In addition, storm surges under which the embankment and approach roadway may be exposed should be considered as well.

Regarding the elevation of the road itself, as sea level rise is a relatively gradual phenomenon (even considering its projected acceleration after mid-century), monitoring over time will allow for evaluation of actual sea level rise trends. If trends point to the likely occurrence of a more extreme sea level rise scenario that could cause temporary or permanent inundation of the approach roadway, appropriate actions should be taken to begin the process of raising the embankment and approach roadway or altering the embankment slope to protect against tidal and wave run-up influences.

Step 11 – Plan and Conduct Ongoing Activities

The recommended ongoing activity is to monitor actual trends in observed sea level in the Mobile region and compare them to the projected sea level rise scenarios with the goal of ascertaining what scenario is being realized. A “trigger level” of sea level change might be established (based on the sea level rise curves and the projected time required to plan, design, finance, and construct a chosen adaptation option) at some point prior to the development of conditions that could cause wave run-up that overtops the roadway along the embankment. When the trigger level is crossed, planning and financing activities for adapting the facility can then commence.

Conclusions

This case study has, using the *General Process for Transportation Facility Adaptation Assessments*, demonstrated how sea level rise effects on wind-generated wave heights and wave impacts can be analyzed for an approach roadway embankment. Protection and risk reduction measures for the US 90/98 Tensaw-Spanish River Bridge west approach embankment would be necessary for each of the climate scenarios tested. An adaptation concept was developed for each scenario whereby the roadway, approach roadway, and abutment would need to be raised to a height that would significantly reduce the risk of overtopping from wave run-up. Information must be shared with local stakeholders and discussed before any locally preferred decisions can be made on what adaptive actions (if any) would be appropriate for raising the road / bridge and when would be the best time to implement them.

The process shown is broadly applicable to roadway embankments across the country where sea level rise and wind-generated waves have an influence. Embankments with susceptibility to wind-generated waves would need to be investigated for potential instability and wave overtopping due to sea level rise to determine if any adaptive action would need to be implemented. Ultimately, this effort is best handled at a planning level in a coordinated manner amongst all embankments along a roadway to ensure that functionality is maintained at all susceptible points. It is recommended that such analyses be undertaken for bridge embankments across the country that are subject to sea level rise and wind-generated wave impacts.

4.4.4 Bridge Abutment Exposure to Storm Surge – US 90/98 Tensaw-Spanish River Bridge (Western Abutment)

Introduction

More powerful storm surges resulting from higher sea levels and stronger hurricanes represent potentially serious threats to coastal bridges. One of these threats is the weakening and potential failure of bridge abutments, critical components of a bridge structure and the primary means of retaining the soil supporting the approach roadway's embankment. Bridge scour is the erosion of the soil surrounding bridge abutments caused by fast flowing water (see Figure 45 for an example of severe abutment scour). Loss of the supporting soil can cause structural instability, shifting of key support structures, and the actual collapse of a bridge. This case study analyzes the potential for scouring of the west abutment of the US 90/98 bridge over the mouths of the Tensaw and Spanish Rivers and the adequacy of the existing rock riprap scour protection measure. Other sections of this report provide case studies of climate impacts to different bridge components such as sea level rise effects on approach embankments (see Section 4.4.3) and storm surge effects on piers and decks (see Section 4.4.5).

Case Study Highlights

Purpose: Evaluate whether a bridge abutment is vulnerable to potential storm surges

Approach: Using the NCHRP 24-20 Abutment Scour Approach, the estimated scour depth and elevations for three storm surge scenarios were determined. Then, the scour protection that would be provided by the protective structures in place (riprap, bulkhead, and willow mattress pads) was evaluated.

Findings: Although on its own the bridge abutment might be vulnerable to the surges, sufficient protective features are in place that will this particular abutment is likely not vulnerable to the surges studied

Viable Adaptation Options (for other, vulnerable sites):

- Widen or lengthen the bridge
- Armor the bridge opening
- Control drainage to avoid erosion in the abutment area
- Control water flow to minimize erosion

Other Conclusions: Protective features like riprap and willow mats play an important role in the ability of an asset to withstand surge—even if the asset itself appears to be on dry ground. It is vital that inspectors look at the whole picture when inspecting assets.

Formulas for estimating scour are very conservative, leading agencies to protect foundations rather than design the foundations to resist scour.

Figure 45: Example of Severe Scour at a Bridge Abutment[166]

The bridge abutment analysis was conducted using the 11 step *General Process for Transportation Facility Adaptation Assessments*, which serves as the organizing framework for this case study. The assessment determined that none of the storm surge scenarios studied presented a threat to the integrity of the abutment and, as a result, no adaptation options are recommended at this time.

Application of the General Process for Transportation Facility Adaptation Assessments

Step 1 – Describe the Site Context

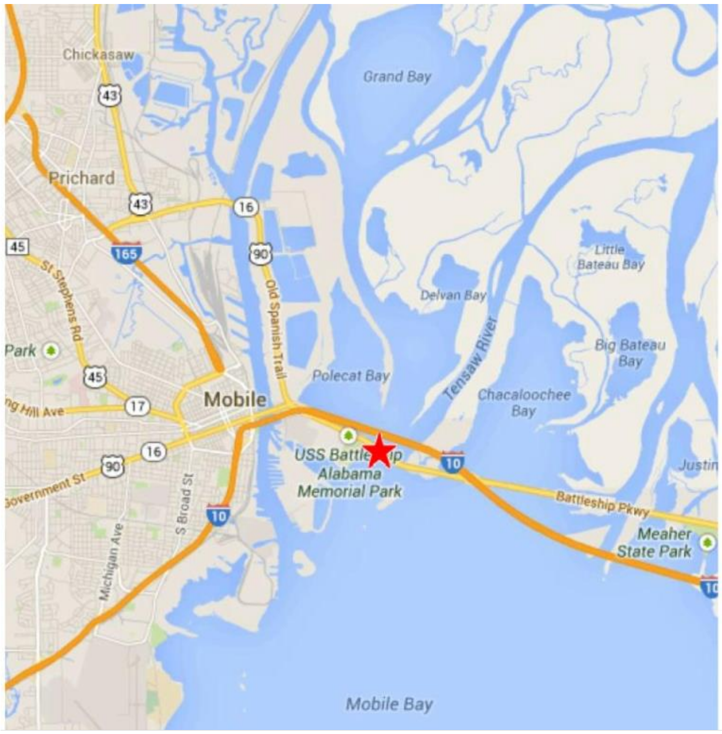
The bridge abutment case study was developed for the west abutment of the US 90/98 (Battleship Parkway) bridge over the mouths of the Tensaw and Spanish Rivers (The Tensaw-Spanish River Bridge). This is the same facility studied for sea level rise impacts to the approach embankments in Section 4.4.3 of this report. The eastbound and westbound bridge abutments are situated 12 feet (3.7 meters) apart (edge-to-edge) and have the same orientation, elevations, and distance from the mean water line. As such, the abutments are considered as a single unit for this study and the resulting scour depths and protections discussed herein can be considered applicable to either the westbound or eastbound abutment.

As noted in Section 4.4.3 , the bridge provides an alternative crossing of Mobile Bay in case the I-10 (Jubilee Parkway) bridge (the “Bayway”) is closed. US 90/98 also serves as the access road for nearby areas including several commercial businesses and the USS Alabama Battleship

[166] Arneson, Zevenbergen, Lagasse, and Clopper, 2012.

Memorial Park, which is located just southwest of the study site. See Figure 46 for a map showing the location of the case study bridge within the Mobile region.

Figure 46: Location of the US 90/98 Tensaw-Spanish River Bridge within the Mobile Metropolitan Area[167]



Step 2 – Describe the Existing Facility

As shown in the aerial photograph in Figure 47, the Tensaw-Spanish River Bridge consists of two parallel bridge crossings: a northern span for westbound traffic and a southern span for eastbound traffic. The northern span was designed in 1963 and has steel girders,[168] whereas the southern span was added in 1995 and utilized pre-stressed[169] concrete girders. Both spans have 10 in-water piers with spacing varied between 125 feet to 140 feet (38.1 to 42.7 meters) apart. Figure 48 provides plan and profile views of the pier configurations and plan details of the southern (eastbound) span.

[167] Source of basemap: Google Maps (as modified)
[168] Girders are the main horizontal supporting members of the bridge.
[169] Pre-stressed construction involves the process of applying tension to the longitudinal steel rebar reinforcement inside a concrete unit during the manufacturing process.

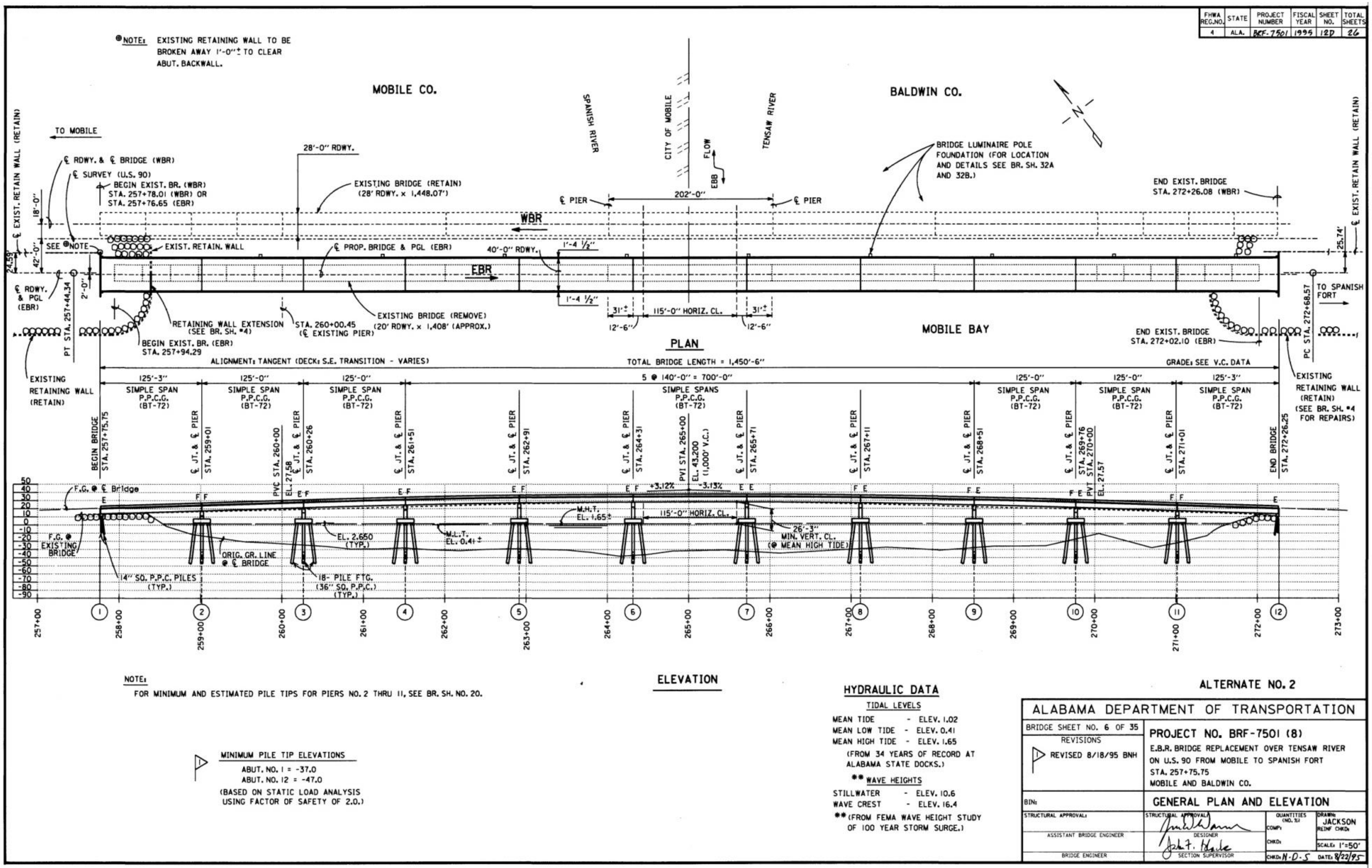
Figure 47: Aerial Image of the US 90/98 Tensaw-Spanish River Bridge[170]

The bridge abutments are a pile-supported concrete design as detailed in Figure 49 and Figure 50. The abutment is supported by 14 inch (35.6 centimeter) square pre-stressed concrete piles as shown on Figure 49. The pile tip bottoms were designed to be driven down to an elevation of -17.7 feet (-5.4 meters).[171]

The abutments are armored against scour by three different design features: a willow mattress,[172] a bulkhead, and stone riprap.[173] The three features work in unison with the goal of providing complete armoring to the bridge abutment and supporting soils. A section view showing the riprap, the timber bulkhead, and the willow mattress and their relation to the west bridge abutments is presented in Figure 51.

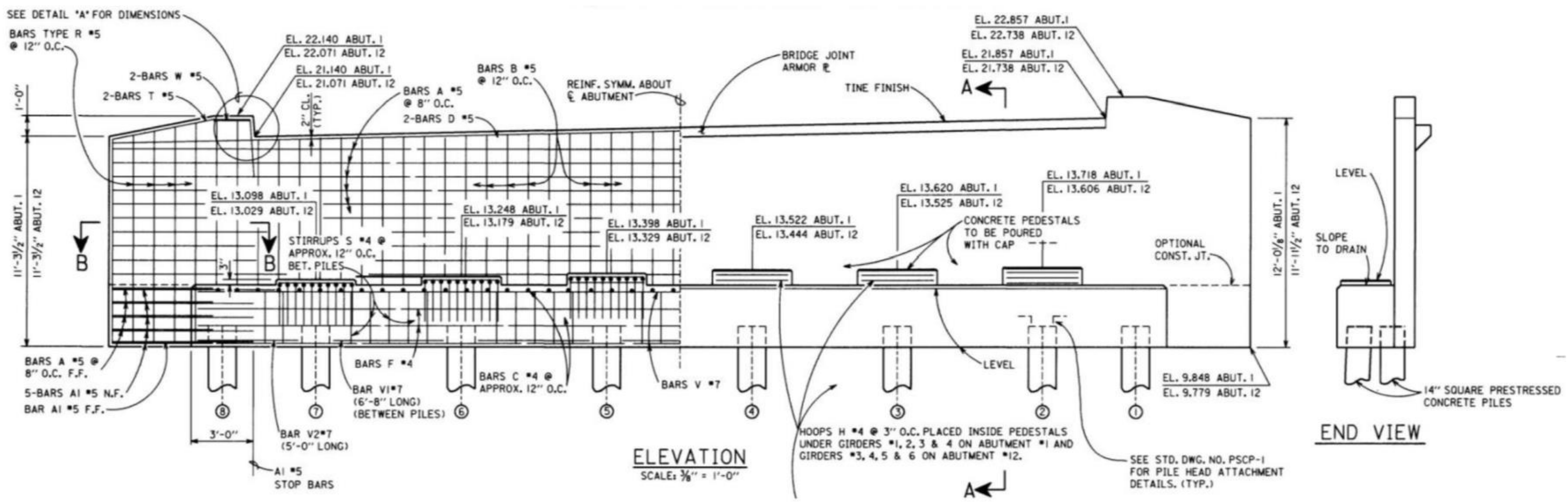
[170] Source of aerial image: Google Maps (as modified)
[171] All elevations in the text of this report are with respect to the North American Vertical Datum of 1988 (NAVD88)
[172] A willow mattress is an interwoven series of willow branch cuttings, joined to form a contiguous semi-rigid mattress.
[173] Riprap consists of loose stone placed in a manner to provide erosion protection or armoring over a soil area.

Figure 48: Plan and Elevation of the US 90/98 Tensaw-Spanish River Bridge Southern (Eastbound) Span[174]



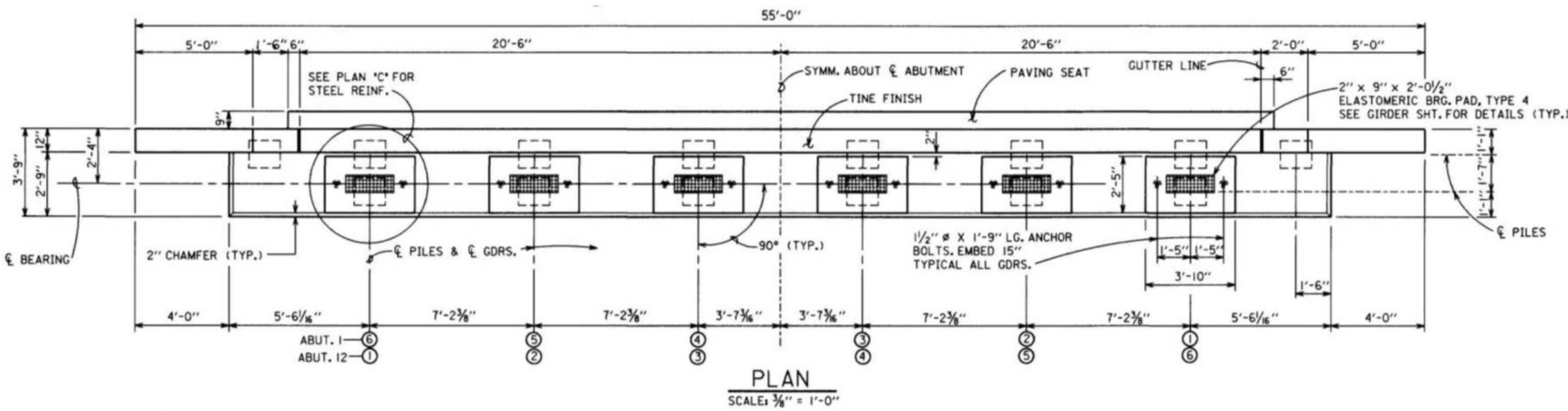
174 ALDOT, 1994

Figure 49: Western Abutment of the US 90/98 Tensaw-Spanish River Bridge Front Elevation Detail[175]



175 ALDOT, 1994

Figure 50: Western Abutment of the US 90/98 Tensaw-Spanish River Bridge Plan View Detail[176]



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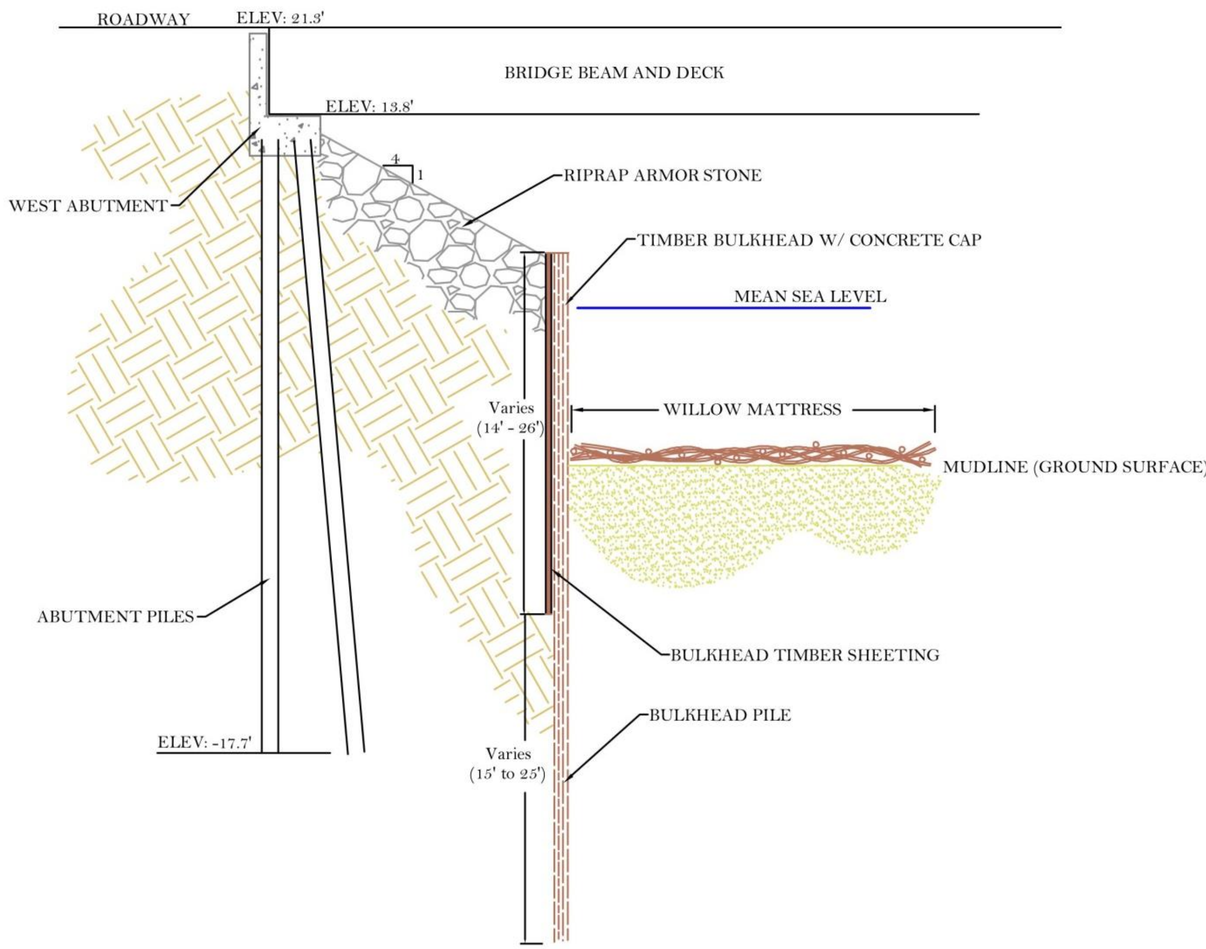
176 ALDOT, 1994

Figure 51: Typical Section View of the West Abutments of the US 90/98 Tensaw-Spanish River Bridge Showing the Scour Countermeasure Protections.[177]



Willow mattress mats were included in the 1963 plans to stabilize the river bottom around the abutment. The 1963 plan showing the locations of these protections is shown in Figure 52. Figure 52 has been modified to show the approximate alignment of the eastbound bridge span, which was not included in the original drawing set. An example of a willow mattress mat being installed is shown in Figure 53. The mattress pads are sunk into place and weighted down using large rock or sections of demolished pavement. Plans for the willow mattress mat show a 100 foot wide by 210 foot long (30.5 meter wide by 64 meter long) mattress that is roughly centered along the alignment of the southern bridge span. In addition, a 50 by 150 foot (15.2 by 45.7 meter) willow mattress also wraps around the north side of the abutment. Construction plans for the southern bridge note that a cutting of the mattress was necessary for the new bridge construction in 1994.

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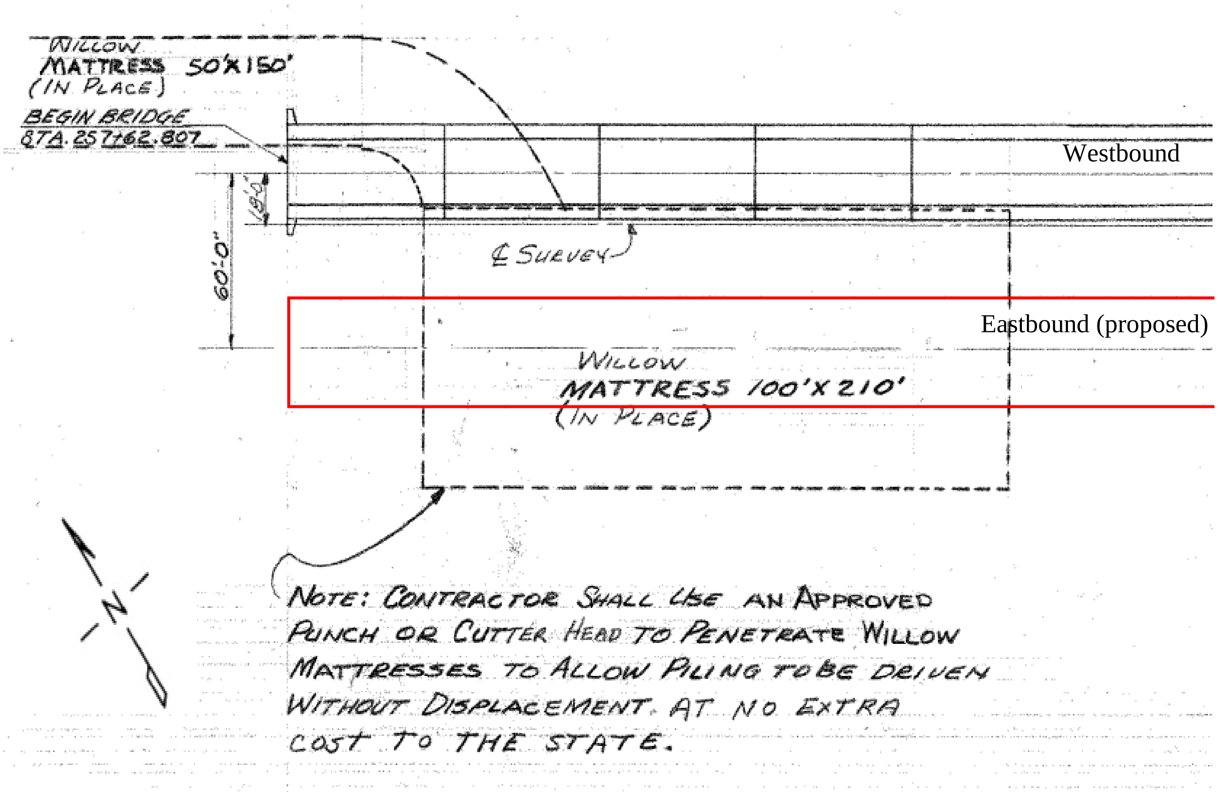
177 Sketch is not to scale and elevations as shown are approximate.

Figure 52: Plan for the Location of the Willow Mattress Pads at the US 90/98 Tensaw-Spanish River Bridge[178]



[178] Source: Highway Department of Alabama, 1963 (as modified). Note: The plan shows only the northern (westbound) span because, as noted previously, the southern (eastbound) span was not part of the original bridge project. Lines have been added showing the approximate location of the eastbound span.

Figure 53: Representative Willow Mattress Pad (Assembly Prior to Submergence)[179]

In Figure 52 note that no information is presented regarding protection along the southern side of the abutment. It is unclear if there exists no protection in this area or if the 50 by 150 foot (15.2 by 45.7 meter) willow mattress marked along the north side of the abutment extends down into this area and was just not included in this drawing. The current study only considers the typical protection section (as shown in Figure 51). Future detailed evaluations of this bridge abutment are recommended to include further research into the coverage and condition of the willow mattresses. In the event that either the southern side of the abutments is not protected or if any portion of the mattress has significantly degraded, the conclusions of the study may be impacted due to the development of scour at a weak point in the system not currently under consideration.

Along with the willow mattresses, a concrete and timber bulkhead protects the southern side of the western bridge abutments and approach roadway. The bulkhead, in this case, is a timber construction coastal retaining wall which extends vertically from a specified depth below the sea floor up to an elevation of approximately 5.5 feet (1.7 meters). Bulkheads in a coastal setting are also commonly referred to as seawalls and are employed to resist coastline erosion, or in this case, storm induced scouring of the roadway approach and bridge abutment. The bulkhead at the US 90/98 Tensaw-Spanish River Bridge consists of timber planks 20 to 26 feet (6.1 to 7.9 meters) long supported by 30 to 40 foot (9.1 to 12.2 meter) vertical timber piles spaced nine feet (2.7 meters) apart. The sheeting and piles are labeled on Figure 51 and an example can be seen in the typical timber bulkhead shown in Figure 54 below. Although 50 years into its design life, aerial imagery indicates the US 90/98 Tensaw-Spanish River Bridge Bulkhead appears to be in

[179] Legasse, Clopper, Schall, and Zevenbergen , 2001

generally sound condition. Although not shown on the available construction plans, a continuous concrete cap was added to the top of the timber structure.

Figure 54: Example of Typical Timber Bulkhead Protection at a Highway Bridge Abutment[180]

Complementing the bulkhead, rock riprap protection armors the slopes above it. This rock riprap protection is the same protection that is evaluated in section 4.4.3 of this report with respect to sea level rise. Review of site photos indicates that the riprap is approximately 15 to 18 inches (38.1 to 45.7 centimeters) in size. The riprap is generally situated on a four horizontal to one vertical slope and has a design depth of three feet.[181] From inspection of available imagery, the sizing is estimated as a Class 3 riprap according to the ALDOT classification system (see Table 26). The riprap coverage displays a significant amount of soil interspersed amongst the rocks. The visible soil interspersed among the rocks could have occurred due to either wind / water borne deposition of sediments or due to dislodging / movement of the riprap. Some bare earth was also observed in historical photos under the bridge adjacent to the abutments, which may indicate some local erosion occurring along the roadway embankments. Future studies of the bridge protection are recommended to include a detailed inspection of the riprap, including determination of the coverage limits and quality of the rock placements, to ensure that no degradation and formation of failure points is developing. The current study assumes that the rock protection has maintained the as-built condition and that the visible soil is due to deposition and not degradation of the rock protection.

[180] Photo of the Mullica River Bridge, New Jersey (taken in 2006).
[181] ALDOT, 1994

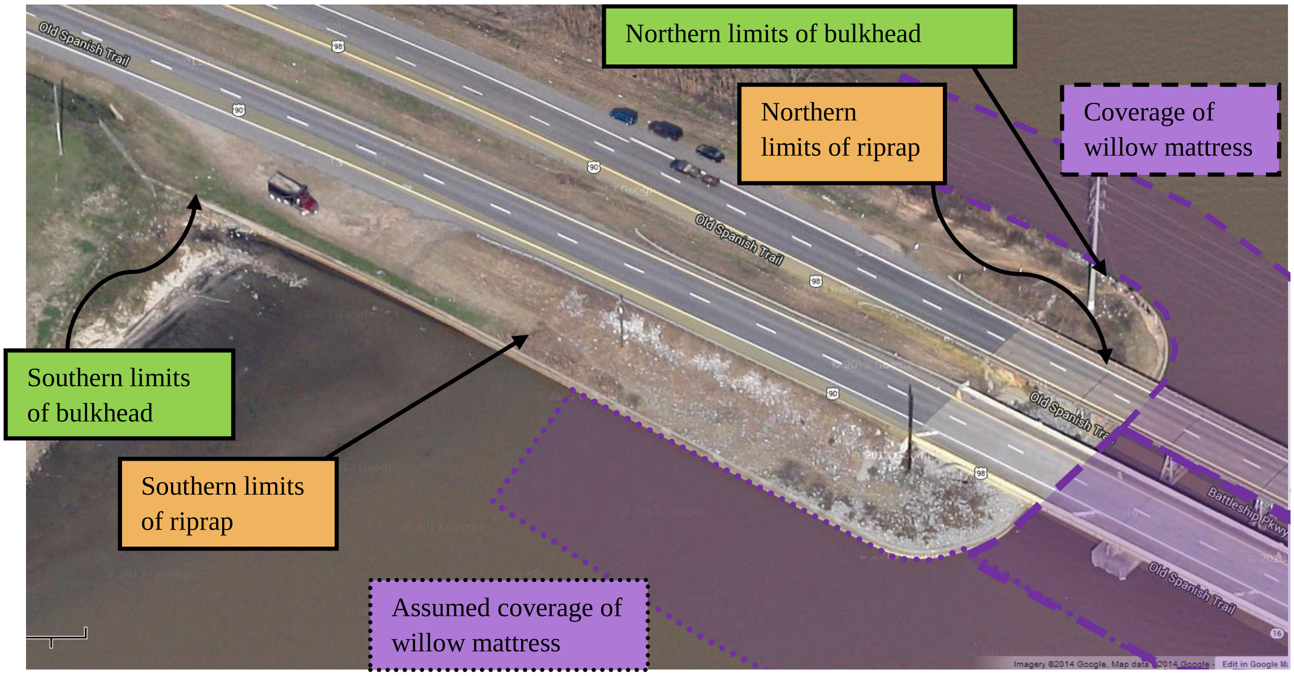
Table 26: ALDOT Standard Riprap Classes[182]

Riprap Class	Weight Range – D_{10}[183] to D_{90} Pounds (Kilograms)	D_{50} Size	
		Pounds (kgs)	Inches (mm)
1	10 – 100 (4.5 – 45)	50 (22.7)	10 (254)
2	10 – 200 (4.5 – 90.0)	80 (36.3)	12 (305)
3	25 – 500 (11.3 – 226.8)	200 (90.7)	15 (381)
4	50 – 1,000 (22.7 – 453.6)	500 (226.8)	22 (559)
5	200 – 2,000 (90.7 – 907.2)	1,000 (453.6)	28 (711)

Figure 55 shows the limits of each of the three scour countermeasure protections overlain on an aerial image of the bridge abutment. In Figure 55, the approximate layout of the willow mattress is shown, including the assumption that the willow mattress was placed to cover the southern side of the approach roadway / abutment.

[182] ALDOT, 2012
[183] Dx denotes rock gradation percentages for which the distribution of individual stones will have diameters of X percentage of the sample batch smaller than the stated measurement value. For example, for D_{50} = 10 inches (254 millimeters), 50% of the rocks in the batch will have diameters smaller than 10 inches (254 millimeters) and 50% will have diameters greater than 10 inches (254 millimeters).

Figure 55: Aerial view of the Tensaw-Spanish River Bridge West Abutment Showing the Limits of Scour Protections[184]



The condition of the various protective design features on the abutment are a key determinant in how effectively they might perform during a storm. In the development of this study, ALDOT underwater inspection records[185] from various timeframes were investigated to determine the conditions of the abutment protection measures, however, in general the inspectors classified the abutments as being "dry" and no additional investigations were performed in these areas. Future studies should investigate the condition of the various protection measures through field inspections (this work was beyond the scope of this case study).

Step 3 – Identify Climate Stressors That May Impact Infrastructure Components

Coastal storm surge on top of sea level rise is the primary climate change-related environmental factor of concern to abutment design. Other climate change considerations that are not included in this study but which may be relevant in more inland riverine settings include precipitation changes and associated changes in riverine flooding.

Step 4 – Decide on Climate Scenarios and Determine the Magnitude of Changes

Design standards for coastal infrastructure such as the Tensaw-Spanish River Bridge typically specify acceptable return periods (e.g., the 100-year storm) for which water is not allowed to overtop facilities or reach other pre-specified thresholds. This section will therefore begin with a detailed discussion of storm surge return periods in the context of a changing climate and some

184 Source of aerial image: Google Maps (as modified)
185 ALDOT, (various dates)

of the difficulties associated with their computation. Then, given the difficulties in accurately computing future storm surge return periods with climate change, an alternative approach to considering surge in the context of a changing climate is presented and used in this case study.

In order to account for the effects of climate change on a particular design event like the 100-year base flood elevation there is a wide range of long term changes to the region that need to be considered. For the nearshore areas of Mobile County, adjacent to the coast and bay, the FEMA 100-year base flood elevation is based on the elevation of the wave crest and thus accounts for both the hurricane induced storm surges and associated wave conditions. Over the following decades it is possible climate change may affect several aspects of the regional topography, bathymetry, and land surface cover or use. For example, sea level rise may alter sedimentation patterns within the bay; may drown existing marshes unless sedimentation rates are able to keep up with rising sea level; and land use may change along the shoreline resulting in modified land cover as well as re-locations of levees, roads and other assets which may affect local flooding conditions. Regional changes in land topography and bathymetry as well as land cover and use have the potential to alter the flood levels (surge plus wave crest) for a given future hurricane event. The long range prediction of these changes is highly uncertain. In addition, land use change may not occur until a catastrophic event forces change (for example, if limited resources or other reasons preclude re-establishment of a structure at its previous location).

How can one adjust 100-year coastal floodplains to accurately account for climate change? The update process for present-day flood mapping provides a starting point for understanding what would be involved. At the time of publication, several regions along the U.S. East and Gulf Coasts are undergoing significant analyses and updates to their FEMA flood insurance studies. An example is the work being performed in the New York and New Jersey area.[186] These flood insurance studies typically include analyses of hundreds if not thousands of hurricane scenarios including historic hurricanes as well as synthetic variations of these historic events.[187]

The most accurate methods of accounting for sea level rise impacts on surge return would be to use an approach similar to the one used in the FEMA updates but, in addition, analyze the set of hurricanes combined with various relative sea level rise scenarios. This would allow incorporation of sea level rise in the results so the resulting recurrence interval events (e.g., the 50-year, 100-year and 500 year) include sea level rise as well as its effect on hurricane surge and waves. In addition, other effects such as increasing storm intensity could be analyzed by accounting for this effect on the storms and using these enhanced storms in the hurricane surge and wave modeling. In areas where drainage of inland floodwaters is impeded by elevated water levels due to surge, the impact of heavier precipitation due to climate change could also be analyzed. However these are significant efforts and will not likely be undertaken as part of

[186] FEMA, 2013a
[187] Synthetic variations of storms are created by altering many variables such as central pressure, forward speed of the storm, landfall location and storm radius.

isolated transportation related planning, repair, or capital projects. Each of these regional flood insurance study updates is a multi-million dollar program that includes:

- Development of a topographic and bathymetric terrain model
- Consideration of the local and regional hurricane history
- Assessment of historical storms for primary parameters that define a particular storm (e.g., storm path, pressure deficit, radius to maximum wind, forward speed of storm, and storm track)
- Choice of a subset of these storms for analysis. The subset is carefully selected to yield a set of hurricanes that capture the general behavior of historic storms.
- Development of synthetic versions of the selected storms to create a wide array of possible storm scenarios by altering the primary storm parameters mentioned above. An example of this is shown in the currently effective Flood Insurance Study (FIS) for Mobile County.[188]
- Assessment of the probability of occurrence for each storm
- Running of each of the synthetic storms (as well as the actual event for the selected storms) with a hurricane modeling system such as the ADvanced CIRCulation model (ADCIRC) and STeady State spectral WAVE model (STWAVE) (the combination used in this study) or, alternatively, ADCIRC coupled with the Simulating WAves Nearshore model (SWAN),[189] another commonly used wave transformation model.
- Noting the model outputs of interest (e.g., wind speeds, water levels, currents, waves)
- Analyzing the hurricane input parameters, probabilities, and model outputs to yield the resulting probability of occurrence (i.e., average annual return interval) as a function of the output parameter of interest (e.g., storm surge, wave height)
- Considering additional flooding aspects beyond the tropical storms mentioned above (e.g., extra-tropical storms, tsunamis)

Given the time and expense associated with an analysis of this type, alternative approaches must be considered to provide surge levels for individual projects that aim to consider sea level rise and surge. The approach taken to develop storm surge scenarios for this study provides one example. Instead of a full scale statistical analysis to develop return period storms, a more limited number of model runs were executed to explore the range of possible impacts in the context of historic storm events that local stakeholders were familiar with. The three storm surge scenarios that were developed for this analysis include:

- Hurricane Katrina Base Case Scenario
- Hurricane Katrina Shifted Scenario
- Hurricane Katrina Shifted + Intensified + Sea Level Rise (SLR) Scenario

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188 FEMA, 2010d
189 Dietrich, Tanaka, Westerink, Dawson, Luettich, and Zijelma, Holthuijsen, Smith, Westerink, and Westerink, 2012

Hurricane Katrina was selected as the base case due to its impacts on the study area and its recent occurrence in 2005 resulting in significant observations and modern data collection of meteorological and oceanographic effects such as pressure deficit, wind, storm track, water level, waves, etc. In addition, this event is the most recent hurricane involving significant storm surge, wind, and waves for residents and transportation stakeholders in the Mobile area. As such it represents an actual event allowing residents and stakeholders a point of reference relative to the additional surge scenarios developed for this study involving an altered storm path and climate change.

For the second surge scenario, the path of Katrina was shifted for a direct hit on Mobile using current sea level and climate conditions. This shifted storm scenario recognizes the accepted practice in hurricane simulation and FEMA Flood Insurance Studies that historic storm landfall locations are assumed to be statistically as equally likely to have made landfall anywhere within the local coastal area.

The third scenario is the shifted Katrina that has been intensified with a reduction in atmospheric pressure resulting in higher wind speeds. This scenario also incorporates an intermediate long term increase in global sea level of 2.5 feet (0.8 meters).[190] This scenario is used in this adaptation assessment to identify impacts associated with possible future climate changes during the latter half of the 21st century.

Simulations of storm-induced water levels (i.e., storm surge) and associated currents were performed for each scenario using the two dimensional depth average version of the ADvanced CIRCulation model, ADCIRC. While the ADCIRC model is capable of applying a variety of internal and external forcings (including tidal forces and harmonics,[191] inflow boundary conditions,[192] density stratification,[193] and wave radiation stresses[194]), only the meteorological forcing[195] input was used to drive the storm-induced flows and water levels for this study. In some circumstances, effects such as elevated stream or river inflow due to previous storms or a particularly wet season may exacerbate flood levels when interacting with storm surge. Numerical models such as ADCIRC can account for this type of situation provided the streamflows are included as part of the hydrodynamic model input.

[190] See Section 4.4.3 for a discussion of the sea level rise scenarios used in this report

[191] A harmonic is a tide wave with a given frequency and amplitude

[192] Inflow boundary conditions are specified when flow conditions are known at the model boundary (e.g., in an estuary, an inflow boundary condition may be specified where a significant stream or river enters the model)

[193] Density stratification may be due to temperature or salinity variation in space. A salinity wedge is an example of a density stratification induced effect where the more saline (heavier) water will tend to flow below the less saline (lighter) water resulting in a density driven current. The salinity difference between the two sources of water is mixed across this interface.

[194] Wave radiation stresses occur where there is variability in the wave conditions from one location to another. For example, in the surf zone the rapid variation of wave height leads to wave radiation stress variation across the surf zone with induced wave setup (a superelevation of water level in the vicinity of the shoreline above the prevailing still water level) and wave induced currents.

[195] Meteorological forcing used in the ADCIRC modeling consisted of a time history of the following data: the latitude and longitude of the hurricane eye, the maximum observed wind speed, the minimum sea level pressure, and the radius from the center of the storm to a specified wind intensity. For further detail the reader is referred to USDOT, 2012.

The wave characteristics accompanying each of the storm surge scenarios were simulated using the STeady State spectral WAVE (STWAVE) model. STWAVE is a flexible, robust model for nearshore wind-wave growth and propagation. It is a steady-state,[196] finite difference,[197] spectral model[198] based on the wave action balance equation. STWAVE simulates depth-induced wave refraction[199] and shoaling,[200] current-induced refraction[201] and shoaling, depth- and steepness-induced wave breaking, diffraction,[202] wave growth based on wind input, and wave-wave interaction and white capping that redistribute and dissipate energy in a growing wave field. Recent upgrades to the model include wave-current interaction and steepness-induced wave breaking. More details on the development of each of the surge scenarios and wave modeling can be found in the *Climate Variability and Change in Mobile, Alabama* report.[203]

With respect to this case study, the model generated outputs for storm surge elevations, flow velocities, and water depths that were collected at five points of analysis (see Figure 56). The points were chosen to represent the peak near abutment conditions (points A and B), average mid-span conditions (point C), the representative approach flood tide (point E), and the representative approach ebb tide (D) conditions. These points were chosen using sound engineering judgment to represent the bridge hydraulics for use in the abutment scour prediction. The selection of the points is intended to be consistent with the hydraulic conditions utilized in the development of the abutment scour equations. While each of these reported points did not ultimately factor into the final computations, they were chosen as important considerations in the understanding of flow conditions around the bridge and the conditions that produced the maximum scour.

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[196] Steady-state implies there is no variation with time. The STWAVE model is run with a single representation of the wind field and offshore wave condition at a single point in time. With this input data STWAVE calculates wave conditions within the modeled area under the given steady-state condition.

[197] Finite-difference indicates the model grid is a series of rectangles and these are typically uniform across the model area. This is in contrast to model grid that can accommodate variation of the model elements in space (such as the finite-element ADCIRC model where, through the use of triangular cells, the size can be altered for model performance improvement such as large cells in the deep ocean and smaller cells nearshore or where higher resolution is needed).

[198] A spectral model indicates the book-keeping for nearshore wave transformation effects is done with a wave spectrum which is a representation of a random wind wave field as a series of sine waves of varying amplitude, direction, and frequency. For additional detail the reader is referred to USACE, 2002.

[199] Wave refraction can alter a wave field due to wave speed modifications that are not uniform in space. Due to variable bathymetry and associated water depths, the wave speed decreases with deceasing depth. Thus, around a headland, wave refraction will focus the wave resulting in increased wave heights. In curved embayments, the wave form will spread out resulting in decreased wave heights.

[200] Wave shoaling results from decreasing wave propagation speed and wave length as a wave advances into shallower water. This effect tends to steepen the wave due to shorter wave lengths and higher wave heights (USACE, 2002).

[201] Another important wave refraction effect is currents that vary in space. A wave entering a region with an opposing current will steepen and may undergo other changes such as altered propagation direction or reduced wave length (USACE, 2002). At the mouths of rivers (known as bars) on the coastline, opposing currents during ebb tide will steepen the oncoming ocean waves, due to smaller wave lengths and higher wave heights, resulting in more hazardous navigation conditions.

[202] Wave diffraction results in wave energy being spread laterally (i.e. perpendicular to the wave propagation direction). This effect is commonly seen behind breakwaters where the advancing wave crest is blocked by an obstruction creating a shadow area behind the structure that is sheltered from the oncoming waves. The wave diffraction process results in wave energy spreading laterally from the undisturbed wave crests into the sheltered area. The result is the classic semi-circular wave pattern of decreasing wave height behind the breakwater that can sometimes be seen in aerial photos (USACE, 2002).

[203] USDOT, 2012 (Section 7)

Figure 56: Aerial Image of the US 90/98 Tensaw-Spanish River Bridge Showing the Five Points of ADCIRC-STWAVE Analysis[204]

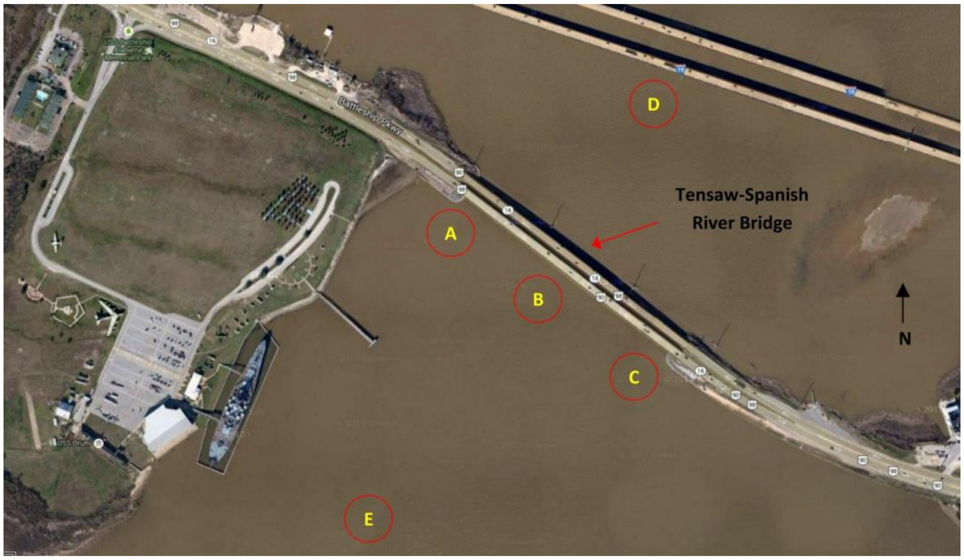
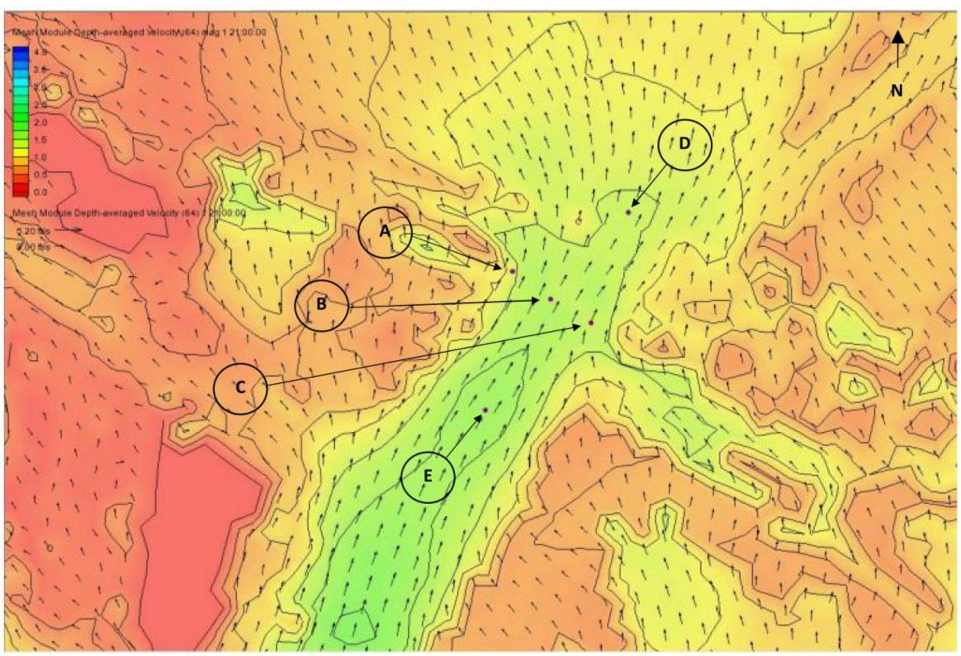


Figure 57 depicts the surge flow vectors from the model during a storm surge condition in relation to the five data points in Figure 56. The total depth of water at a point of analysis is arrived at by totaling the water elevation and the river mudline[205] elevations. The vectors shown in the image have been interpolated from the ADCIRC model and are not representative of the computation points in the model.

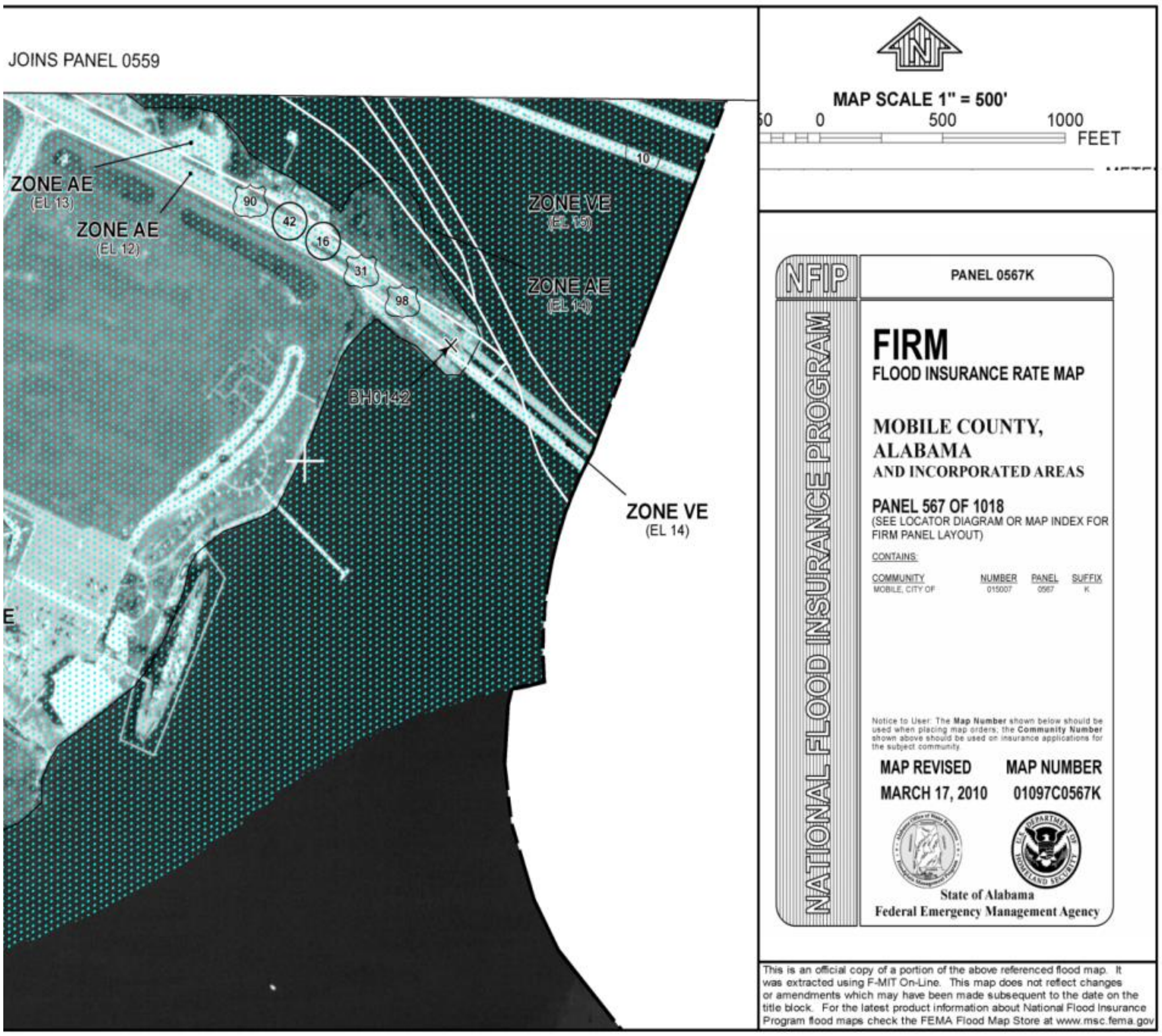
Figure 57: Flow Velocity Vectors and Evaluation Data Points at US 90/98 Tensaw-Spanish River Bridge



[204] Source of aerial image: Google Maps (as modified).
[205] Mudline is a reference to the ground elevation at the bottom of the channel at a given point.

Although the return periods of each of the surge scenarios now or in the future was not calculated, a comparison was provided to the current FEMA 100-year storm. As shown in Figure 58, the current FEMA 100-year storm overtops the entire abutment. The FEMA 100-year flood elevation is higher than the Hurricane Katrina Base Case Scenario, but less than the Hurricane Katrina Shifted Scenario as summarized in Table 27.

Figure 58: FEMA Flood Insurance Rate Map for the West Abutment of the US 90/98 Tensaw-Spanish River Bridge[206]



[206] FEMA, 2010b. Note: The elevations shown are in NAVD88.

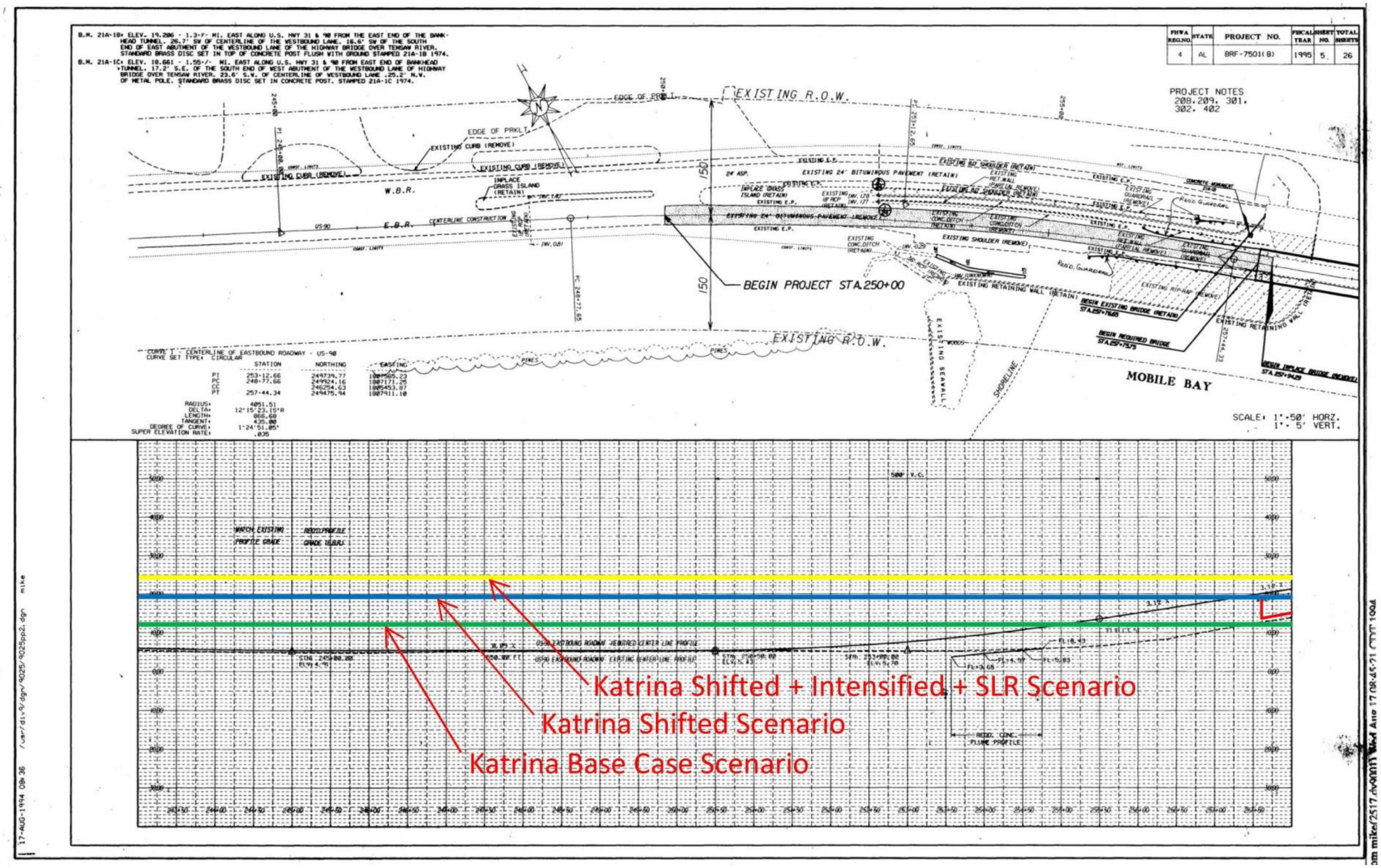
Table 27: Stillwater Elevations and Wave Heights at the Western Abutment of the US 90/98 Tensaw-Spanish River Bridge

Surge Scenario	Bridge Abutment Bottom of Girder / Roadway Top Elevation Feet (Meters) NAVD88	Stillwater[207] Elevation Feet (Meters) NAVD88	Wave Height Feet (Meters)
Hurricane Katrina Base Case Scenario	13.8/21.3 (4.2/6.5)	12.4 (3.8)	5.2 (1.6)
Hurricane Katrina Shifted Scenario		19.7 (6.0)	8.2 (2.5)
Hurricane Katrina Shifted + Intensified + Sea Level Rise (SLR) Scenario		24.6 (7.5)	4.1 (1.2)
FEMA Base Flood Elevation (100-year flood)[208]		14.0 (4.3)	

Figure 59 and Figure 60 show the surge stillwater elevations with respect to the approach to the western abutment and the bridge bottom chord elevation, respectively. The flooding profiles presented show that each storm surge condition overtops the timber bulkhead (elevation 5.5 feet [1.7 meters]), and will impact the abutment and the riprap along with the bulkhead and the willow mattress. Note that the stillwater elevations for two of the modeled storm surge scenarios and the FEMA base flood elevation overtop the western approach roadway. Additionally, the stillwater elevations of the Hurricane Katrina Shifted Scenario and the Hurricane Katrina Shifted + Intensified + SLR Scenario are higher than the lower chord of the bridge deck section in the area of the abutment. This overtopping condition will serve to lower the predicted abutment scour at the Tensaw-Spanish River Bridge but could result in damage to the roadway and loss of service during the surge and the immediate aftermath (due to clean-up). While issues related to the overtopping of the approach roadway and surge impacts on the bridge deck are not the primary focus of this study, they are larger issues of concern that are recommended to be investigated during the detailed evaluation of this or any bridge structure.

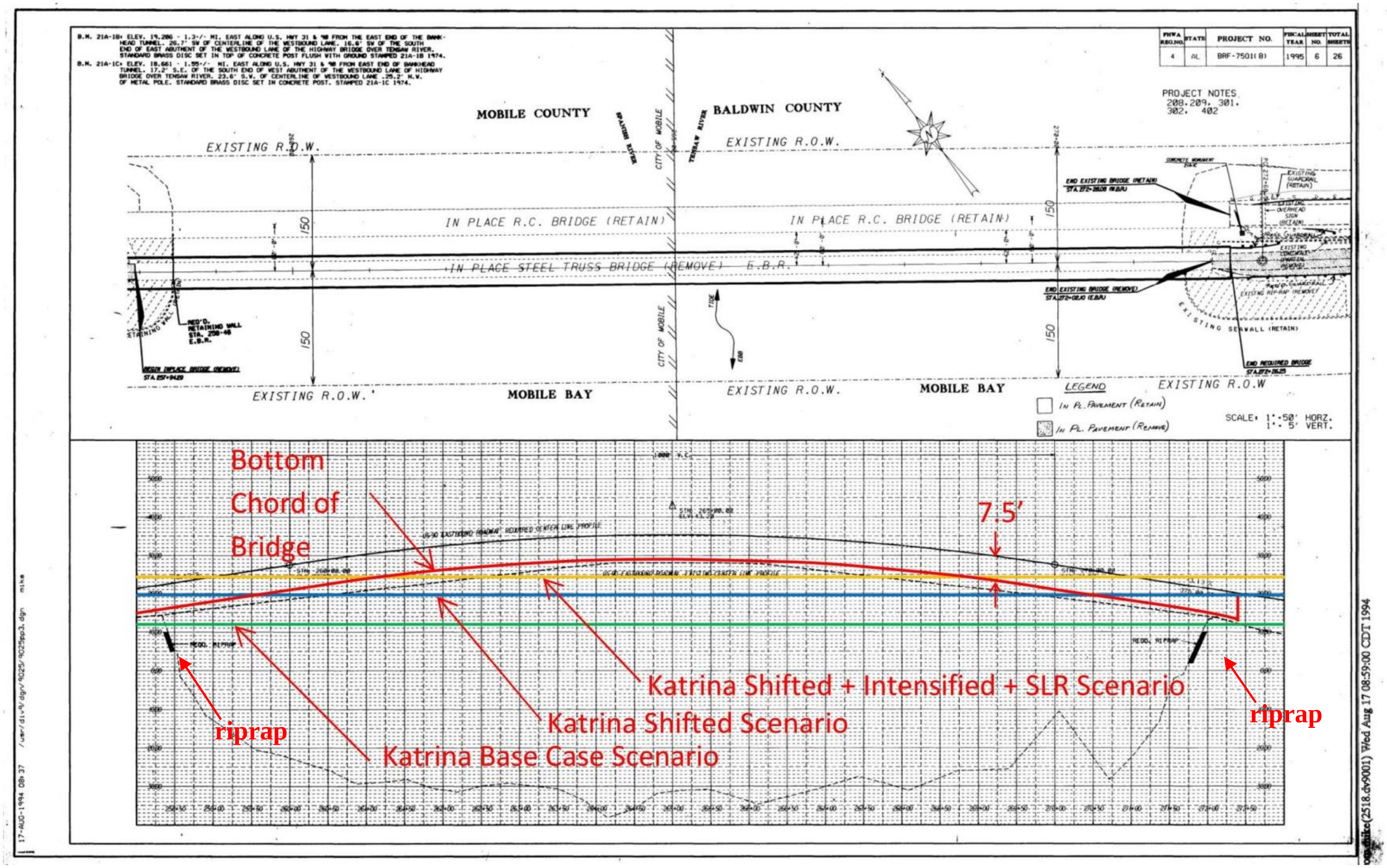
[207] Stillwater refers to the maximum elevation of a coastal storm surge without the addition of waves.
[208] FEMA, 2010b

Figure 59: Plan and Profile of the Approach Roadway to the Western Abutment of the US 90/98 Tensaw-Spanish River Bridge with Water Levels[209]



[209] Source: ALDOT, 1994 (as modified)

Figure 60: Plan and Profile of the US 90/98 Tensaw-Spanish River Bridge with Water Levels[210]

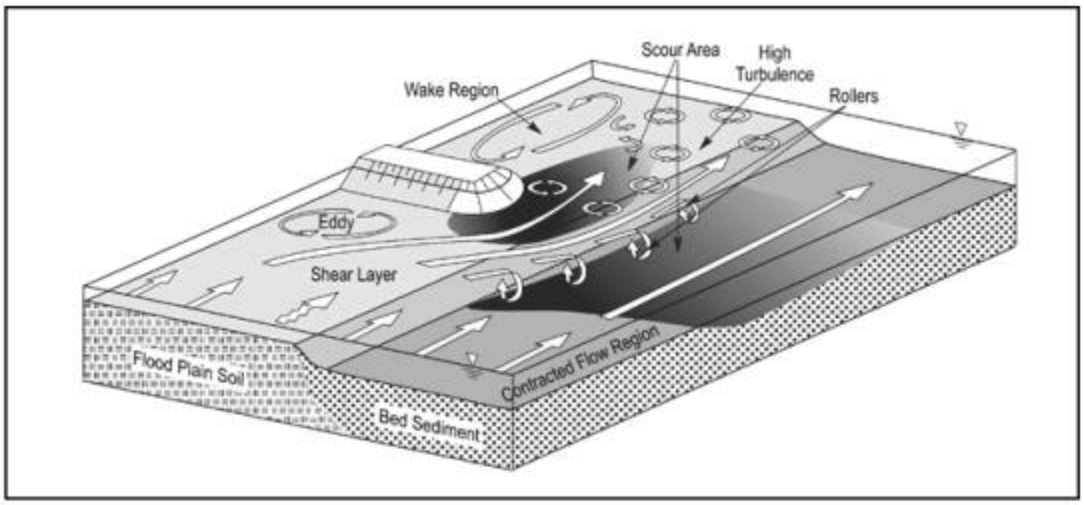


[210]Source: ALDOT, 1994 (as modified)

Step 5 – Assess Performance of the Existing Facility

Erosion of the soils at bridge abutments is generally caused by three hydraulic forces: contraction scour, horseshoe vortex formation, and wave vortices. Contraction scour occurs at abutments as the approaching flow is contracted from a wider flow area (in this case Mobile Bay) into the smaller bridge opening. As the flow is contracted, velocities are increased and the ability to erode soil can be greatly increased. Horseshoe and wake vortices are turbulent hydraulic formations that occur as flow impinges on a solid structure. Figure 61 shows the typical formation and shape of each of these vortices.

Figure 61: Schematic Representation of Abutment Scour[211]



Abutment scour is generally calculated as a combination of velocity, flow rate, flow depth, and geometric factors related to the abutment. In general, peak scour depths for any structural element normally occur when the flow velocity is highest. Under coastal storm surge conditions this peak velocity does not coincide with the peak water surface elevation. Due to the bi-directional nature of coastal surges (flood and ebb surge), the peak velocity occurs at two points, first during the flood surge and later during the ebb surge. The peak velocity for each of these conditions occurs when the rate of water surface change is at its greatest. In this case study, the peak flood and ebb surge velocities were compared at the approach locations (points E and D on Figure 56, respectively) and the velocities for the flood surge condition were found to be controlling. Thus, the abutment scour and protection computations were performed for the flood surge peak velocity.

A bridge abutment scour study usually requires a great deal of data, often collected on the latest foundation and material conditions at the abutments. Much of the required data was not available for this case study. Therefore, several assumptions were made concerning some basic inputs into

[211] NCHRP, 2011

the analysis. These assumptions were made specifically to be conservative in the estimation of impacts. Important assumptions for this analysis were:

- Soil borings or other geotechnical sampling and measurement data were not readily available at the abutment location. Given the location and setting of the bridge crossing, the soils were assumed to be medium sand with an average diameter of 0.01 inches (0.3 millimeters). For an actual study, the soils conditions should be ascertained from on-site boring information.
- A detailed bathymetric survey was not performed for this investigation, thus the channel bed data from the ADCIRC model was assumed to provide an accurate representation of the channel bed in front of the abutment. The ADCIRC model was built upon historic bathymetric and topographic data available from USGS and NOAA. While this data provides a reasonable basis for the current study, the accuracy of the data is expected to be limited by potential long-term changes to the channel bottom (sedimentation) or other influencing factors. Project specific bathymetric survey of the channel bottom would be recommended for a detailed study of the bridge abutment.
- Return period type storm events (e.g., the 100-year and 500-year storms) were not modeled as part of the coastal storm surge simulations for this project, thus the three chosen hurricane simulations were chosen as analogs for the design storms for the crossing.
- The abutments of the Tensaw-Spanish River Bridge have a complex protection scheme with the pile-supported abutment protected by willow mattresses, a bulkhead, and stone riprap. Following standard engineering practice, scour at the abutment was computed without the influence of the protections and the sufficiency of each protection was then investigated individually. Combining the results for each component's sufficiency – that is, for the abutment, willow mattresses, bulkhead and stone riprap – led to an overall conclusion on the stability of the abutment.

The three common abutment scour prediction equations presented in FHWA's *Evaluating Scour at Bridges; Fifth Edition* (HEC-18)[212] were assessed for their appropriateness to the case study site. The equations included:

- The Froehlich Abutment Scour Equation
- The HIRE Abutment Scour Equation
- The National Cooperative Highway Research Program (NCHRP) 24-20 Abutment Scour Approach[213]

The evaluation concluded that the HIRE equation development conditions did not meet the hydraulic conditions at the west abutment as the ratio of the flow depth to approach roadway length fell short of the recommended value of 25. The Froehlich equation was also investigated;

[212] Arneson et al., 2012
[213] NCHRP, 2010

however, as is often observed in practice, the equation produced overly conservative scour predictions. Thus, the NCHRP 24-20 approach was used for the detailed evaluation of the west abutment. The NCHRP approach was developed considering, "a range of abutment types, locations, flow conditions, and sediment transport conditions."[214] The approach considers contraction scour as a component of the total scour predictions, whereas the HIRE and Froehlich methods require a separate evaluation of contraction scour. The NCHRP method utilizes flow depth, unit discharges,[215] and a scour amplification factor[216] to compute total abutment scour. Chapter 8.6.3 of the HEC-18 publication includes a detailed discussion of the development and application of the NCHRP 24-20 approach. Table 28 presents the results of the NCHRP approach as applied to the west abutment of the Tensaw-Spanish River Bridge.

Table 28: Predicted Abutment Scour Depths and Elevations at the West Abutment of the US 90/98 Tensaw-Spanish River Bridge

Surge Scenario	Predicted Scour Depth Feet (Meters)	Predicted Scour Elevation Feet - NAVD[217] (Meters)	West Abutment Bottom Pile Elevations Feet-NAVD (Meters)
Hurricane Katrina Base Case Scenario	33.2 (10.1)	-43.2 (-13.2)	
Hurricane Katrina Shifted Scenario	38.4 (11.7)	-48.4 (-14.8)	-17.7 (-5.4)
Hurricane Katrina Shifted + Intensified + Sea Level Rise (SLR) Scenario	35.0 (10.7)	-45.0 (-13.7)	

As shown in Table 28, all three of the coastal storm surge scenarios result in scour depths that exceed the constructed depth of the abutment foundation. Thus, the analysis shows that the west abutment was not designed in consideration of full abutment scouring conditions, and in lieu of other protection factors, the abutment could be reasonably expected to fail. However, this design condition is not unique to this bridge design or unexpected. Given the widely held view that abutment scour equations produce overly conservative results, many state agencies have chosen to armor or otherwise protect abutments from scour rather than design the foundations for the full scour depth. In the case of the Tensaw-Spanish River Bridge, the previously noted riprap, willow mattress pad, and timber bulkhead have been utilized to protect the abutment from scouring. A comprehensive evaluation of the sufficiency of the abutment will be defined as the sum of the sufficiency of the protections, as the scour evaluation has shown that the abutment

[214] Arneson et al., 2012 (page 8.8)
[215] Unit discharges is defined as the average flow rate over a one foot (0.3 meter) unit width. Unit discharge has units of square feet per second as opposed to discharge which has units of cubic feet per second.
[216] Scour amplification factors are determined by charts specific to the NCHRP methods and are found as figures 8.9 through 8.12 in Arneson et al., 2012
[217] Predicted scour elevations estimated the bed elevation at the edge of channel being-10 feet (-3.1 meters)

itself will likely fail in their absence. In actual practice, the soils conditions should be accurately determined and, typically, a cost analysis performed to compare the costs of additional pile length to the cost of installing a number of scour protection measures. The following sections include evaluations of the scour and hydraulic sufficiency for each protection measure as related to the forces that constitute abutment scouring.

The upper most protection along the abutment is the rock riprap. The riprap runs from the top of the bulkhead to the top of the abutment. The design of the protecting riprap was evaluated using guidance set forth in the FHWA HEC-23 publication.[218] The size of riprap required to protect against scour in the abutment area is largely driven by the velocities of the flows modeled adjacent to the abutment (Point A in Figure 56). The Isbash relationship was utilized to compute the required size of riprap for protection of the abutment. The relationship has the following form:[219]

$$\frac{D_{50}}{y} = \frac{K}{(S_s - 1)}\left[\frac{V^2}{gy}\right]$$

Where;

y	Flow depth (feet)
V	Flow velocity (feet per second)
D_{50}	Median diameter of the rock riprap (feet)
K	Abutment shape coefficient (0.9, used for spill-through abutments)
g	Gravitational acceleration (32.2 feet per second squared [9.8 meters per second squared])
Ss	Specific gravity[220] of stone (2.7)

As discussed in Step 4, the classification of the existing riprap protecting the abutment was estimated to be Class 3 with a D_{50} of 15 inches (0.4 meters). Table 29 details the velocities, riprap size required, and the riprap size observed in place. The Isbash equation shows that the current riprap is sufficient to resist abutment scour. However, it should be noted that the riprap size from the Isbash equation does not consider wave impacts and the effects of general sea level rise on the embankment. These factors are analyzed in Section 4.4.3 and should be considered as part of the combined treatment of the abutment and approach embankment. The larger of the two rock sizes determined from the different approaches would govern and should be considered for use in armoring the abutment area. The results of the wave scour study are presented in Table 29 and show that the wave scour sized riprap with a 28 inch (71.1 centimeter) D_{50} (Class 5 riprap) would be recommended for placement along the entire abutment. While the computations have

[218] Legasse, Clopper, Pagan-Ortiz, Zevenbergen, Arneson, Schall, and Girard, 2009
[219] Legasse et al, 2009
[220] Specific gravity relates the density of one substance to that of another substance

shown that the current riprap sizing is sufficient to protect against storm surge induced currents, the riprap is not sufficiently sized to combat wave impact forces. As such, the riprap could represent a weak link in the scour protection for the abutment, in the event that the sea level rise and wave conditions documented in Section 4.4.3 occur. Readers are referred to Section 4.4.3 for a discussion of the sizing of the riprap for sea level rise conditions and associated adaptation options. The potential under-sizing of the riprap provides one area of potential weakness in the overall abutment protection scheme; however, since the system works together with multiple components, the bulkhead and willow mattress also factor into the overall vulnerability of the system.

Table 29: Required Riprap Size to Resist Abutment Scour at the Western Abutment of the US 90/98 Tensaw-Spanish River Bridge[221]

Storm Surge Scenario	Velocity at West Abutment (Point A) Feet/Sec (Meters/Sec)	D_{50} Computed Size Feet (Meters) – Abutment Scour	D_{50} Computed Size Feet (Meters) – Wave Scour	Estimated D50 Size of In-place Riprap Feet (Meters)
Hurricane Katrina Base Case Scenario	3.9 (1.2)	0.3 (0.09)	2.3 (0.7)	1.3 (0.4)
Hurricane Katrina Shifted Scenario	5.9 (1.8)	0.6 (0.2)		
Hurricane Katrina Shifted + Intensified + Sea Level Rise (SLR) Scenario	6.2 (1.9)	0.6 (0.2)		

The bulkhead protecting the southern side of the abutment is the next protection factor that was considered. As noted above, the bulkhead is a predominantly timber construction, but has been capped with concrete. As a solid structural element, the bulkhead is anticipated to perform following the same rules as the abutment. In this case, the bulkhead would either need to be designed to the full abutment scour depth, or would require stabilized protection as is the case for the abutment. While detailed pile tip information is not readily available for the bulkhead, the record drawings do show a length of 30 to 40 feet (9.1 to 12.2 meters) for the timber piles. The bulkhead has a top elevation of approximately 5.5 feet (1.7 meters) which would correlate to pile tip elevations between -25 and -35 feet (-7.6 to -10.7 meters), depths insufficient to protect the bulkhead against the peak scour conditions. An additional consideration for the viability of the timber bulkheads is the depth of the solid wood planking which retains the roadway fill behind the bulkhead. The wood planking is estimated to be present from the top of bulkhead to elevations -9.5-feet to –20.5-feet. While scour greater than these stated elevations may be

[221] Legasse et al, 2009

required to fail the timber piles which support the bulkhead, scouring to a depth below the wood planks will similarly result in a failure as fine fill materials used for the approach roadways will be readily eroded causing slumping, soil loss, and potential failure of the overlying roadway. However, as with the abutment, the ultimate stability of the bulkhead will be determined by also considering the protection afforded by the last component of the system, the willow mattress pad.

Willow mattress pads, also known as fascine[222] sinker pads, are a long-standing practice for the protection of bridges or other waterway structures. The structure is much more common in Europe, where it has a record of good performance. The willow mattress pad for the case study abutment was installed in 1963 prior to the construction of the southern (eastbound) bridge. Despite being constructed from natural woody materials, under anaerobic[223] and permanently submerged conditions these mats are very durable[224] and could still effectively function to prevent erosion. It was estimated that the permissible velocity[225] of the willow mattress pads was equal to or greater than 12 feet per second (3.7 meters per second),[226] the permissible velocity of a newly constructed brush layering revetment. The peak storm velocities for each of the analyzed conditions do not exceed the estimated permissible velocity for the willow mattress, thus the mattress is considered to be stable for all of the considered storm events.

Results of the evaluation of the individual protections show overall stability for the system. This conclusion is based upon a holistic review of the protection scheme which showed:

- The willow mattress pad is stable against the design condition flow velocities
- The bulkhead is protected along its base by the willow mattress pads and thus is also stable
- The riprap protection is appropriately sized for storm velocities
- The base of the riprap is protected by the bulkhead and the top of the riprap extends up to the concrete abutment, thus the riprap is also stable.

In consideration of the complex nature of the abutment protection scheme, each of the individual components was reviewed for stability under the various storm conditions. The review concluded that while the abutment itself is not designed to be stable under storm scour conditions, the protection components of riprap, bulkhead, and willow mattress have all been shown to be stable. Thus, the combined considerations for the abutment and the protection scheme shows that the system is stable and capable of performing for the current design conditions and each of the projected storm events. Ideally, the bridge abutment foundations would be designed for protection of the bridge and the approach roadway against the full depth scour event, providing a more sustainable protection than use of various scour countermeasures. However, the design and

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[222] Fascine refers to the cylindrical bundle of sticks making up the pad.
[223] Anaerobic conditions refer to environments free of oxygen.
[224] Biedenharn, Elliott, and Watson, 1997
[225] Permissible velocity is the maximum velocity at which a given material meets stability criteria. Above this velocity a material may move or deteriorate.
[226] Fishenich, 2001

construction of bridges with the use of scour countermeasures over deep foundations is a common practice that is generally employed for bridge abutments.

That said, while the materials for protection of the abutment have been shown to be adequately sized, the materials used are subject to degradation over time. Proper maintenance and upkeep of the system is necessary to ensure the long-term success of the protection. As previously noted, review of the bridge underwater inspection records have shown that the condition of the bulkhead, willow mattresses and riprap armoring have not been as closely monitored as other aspects of the bridge. With the conclusion of this study and the observation that the stability of each of the three scour countermeasures is integral to the overall stability of the abutment, future inspections should include detailed inspections of each of these components.

Step 6 – Identify Adaptation Option(s)

The results of the abutment analysis concluded that the structure and provided protection measures were sufficient to meet current and projected storm conditions for all surge scenarios analyzed. Thus, adaptation for this particular system may not be necessary based upon the climate scenarios considered in this study. However, for other potential future climate scenarios that portend more extreme surges or for other abutments where adaptation measures might be needed, potential adaptive design options for controlling abutment scour include:

- Reconstruction of the protective bulkhead to a depth that is stable under projected scour conditions. Bulkhead would be constructed using a more sustainable material such as Fiber Reinforced Polymer or Vinyl sheeting to a deeper pile penetration. Riprap overlying the slopes above the bulkhead would be replaced with appropriately sized riprap coverage. Lastly, with bulkhead driven to the appropriate depth, the willow mattresses would not be necessary and protections further below the water could be removed from the bridge.
- Controlling the approach and departure flow to realign water passage though the waterway. This could be done by providing a stable and gradual transition to and from the bridge opening by using guidebanks, spur dikes, bendway weirs, or vanes.[227]
- Armoring of the bridge opening with riprap, concrete revetment,[228] or bulkhead / retaining walls
- Modifications to the bridge including widening, lengthening and / or shifting it
- Control of drainage from the embankment and roadway to avoid erosion starting in the abutment area

Step 7 – Assess Performance of the Adaptation Option(s)

No adaptive measures have been proposed for the study site. For abutment sites where adaptation options would be required, the performance of each adaptation option relative to each climate

[227] Guide banks, bendway weirs, spur dikes, and vanes are all rock riprap formations that are strategically placed to direct flows into the center of a bridge opening.
[228] Concrete revetments would include large or interconnected concrete blocks placed as embankment armoring.

scenario should be evaluated. This will provide important information for use in the economic analysis.

Step 8 – Conduct an Economic Analysis

An economic analysis is not required for this case study since no adaptation options were deemed necessary. For other abutment studies where adaptive alternatives have been proposed, see Section 4.4.1 for an example of how an economic analysis could be applied.

Step 9 – Evaluate Additional Decision-Making Considerations

The case study for abutment scour does not recommend any adaptive design measures for the Tensaw-Spanish River Bridge west abutment. However, for other abutment studies where adaptations are required, factors that will influence whether abutment protective measures are installed include:

- **Redundancy**: Abutment failures can take a long time to repair, resulting in road closure or reduced capacity. Not having redundancy in the network in case of failure should weigh on the decision of whether to pursue adaptation measures.
- **Constructability:** Retrofits to bridge foundations are complex and difficult construction projects, and could result in the temporary closure of a structure. Constructability and traffic studies to ensure that a proposed project is feasible could be a key consideration for any type of bridge adaptation project. Scour countermeasure work can present constructability issues due to limited clearance over many low lying bridges or due to limited or difficult access to the embankment slopes of shoreline.
- **Durability and maintenance**: The durability of scour countermeasures, especially in light of expected surge, is a key criterion in the design of such measures. In addition, the level of maintenance associated with the countermeasures should be considered.
- **Environmental issues**: Given their innate proximity to waterways and position along shorelines and streambanks, the use of scour countermeasures or construction activities could have negative impacts on the environment. In the evaluation of adaptations the impacts of both construction access and of changes in the shoreline / streambank composition (i.e., covering a sandy shoreline with large riprap armor stone or disturbing aquatic vegetation beds for placement of a scour countermeasure) should be carefully considered. Adaptation options should protect and minimize the impact on habitat and natural resources.
- **Aesthetics / recreational use** – This can be a key issue for public and stakeholder acceptance of adaptation countermeasures. In some locations, bridge crossings / abutments are located in highly visible areas, such as beaches or nature areas. In these locations, the use of a context sensitive treatment that does not limit the usage of the shoreline / riverbank or create an eyesore, should be considered. These considerations may include avoiding the use of armor stone due to its potential to both provide a hazard to pedestrians and its unsightly nature. Adaptations can consider the use of bioengineering treatments, subterranean countermeasures, or construction of stable foundations in lieu of countermeasures.

Step 10 – Select a Course of Action

The recommended course of action is to undertake no adaptations to prevent abutment scour at this time. This conclusion should be reevaluated if updated climate projections are developed that portend more severe surge conditions at the facility.

Step 11 – Plan and Conduct Ongoing Activities

Adaptation has not been recommended as a course of action for this study site, based on the combination of climate scenarios considered in this study. However, the materials used in the protection scheme for the abutment (riprap, bulkhead, and willow mattress pads) will all require regular maintenance inspections and periodic maintenance upkeep. Even with adequately sized riprap and willow mattress protection, these types of scour countermeasures are subject to degradation due to rocks shifting, settlement, or damage to the mattress or riprap from boat or debris impacts. The wearing of these revetments can occur due to several small storm events, but can be anticipated to accelerate during larger events. In keeping with Federal Highway Administration (FHWA) directives, the protections at the abutment should undergo periodic inspections (including inspections after significant storm events) that assess the condition of each protection element and recommend any needed repairs to the structures.

Conclusions

Based on the *General Process for Transportation Facility Adaptation Assessments*, this case study has demonstrated how a bridge abutment can be analyzed for various storm surge scenarios, including one factoring in sea level rise due to climate change. The analysis showed that the foundation design of the west abutment to the US 90/98 Tensaw-Spanish River Bridge is vulnerable to scour from all of the surge scenarios tested but that the existing protection measures in place today will provide adequate protection of the facility under each scenario. Thus, no adaptation option is recommended for the facility at this time to address the issue of abutment scour.

This case study demonstrated that the methodology for estimating abutment scour is very conservative, especially for typical coastal conditions. By standard practice, scour is usually protected against by adequately sized and installed riprap armoring. Abutment scour analysis procedures should be developed to allow for more accurate prediction and characterization of abutment scour. Updates to the equations would be appropriate and should include more open and detailed discussion within the design community related to both the development uncertainty inherent to the equations and the uncertainty related to scour prediction for any structure under changing climate conditions. With updates to prediction techniques, structural design guidelines should be updated to require the design of abutments to be stable without the need for outside protection schemes, such as riprap or a bulkhead.

This case study also included evaluation of underwater inspection records for the bridge which included the observation that current underwater inspection practices may not be fully considering the protection schemes at bridge abutments. Based on these observations, the practices performed during a given inspection should be revisited to ensure the long-term success of abutment scour countermeasures. Inspectors should be informed that even if the structural portion of an abutment is situated on "dry" ground, other components such as bulkhead, riprap, or other stability measures may play a key role in the overall scour resistance of the abutment and should likewise be monitored.

4.4.5 Bridge Segment Exposure to Storm Surge – The US 90/98 Ramp to I-10 Eastbound at Exit 30

Introduction

Bridges serve as important linkages between communities. With climate change, higher storm surges resulting from rising sea levels and potentially stronger coastal storms enhance the threat of devastating impacts to coastal bridges that can sever vital connections for long periods of time. One such vital connection is the bridge carrying I-10 (the Jubilee Parkway) across Mobile Bay (more commonly known as the "Bayway"); a 7.5 mile (12.1 kilometer) long structure connecting Mobile to the eastern shore of the Bay. This case study assesses projected storm surge impacts on a ramp connecting to that bridge: the elevated ramp leading from US 90/98 (the "Causeway") to I-10 eastbound at Exit 30.

The storm surge analysis for this bridge was conducted using the 11 step *General Process for Transportation Facility Adaptation Assessments*: this serves as the organizing framework for the remainder of the case study. The assessment determined that the portion of the ramp studied was currently highly vulnerable to damage from storm surges and will be even more so in the future. The recommended course of action is to de-commission all or a portion of the interchange served by the ramp after the next storm event that causes major damage. In the near-term, a detailed study exploring the implications of this action should be conducted.

Case Study Highlights

Purpose: To evaluate whether storm surge could cause a bridge to fail via any of three failure modes: (1) a wave uplifting and washing away the superstructure, (2) failure of the substructure due to the lateral forces of the wave, and (3) failure of the substructure due to excessive scour.

Approach: Failure Mode 1 was evaluated by using equations from AASHTO's *Guide Specifications for Bridges Vulnerable to Coastal Storms* to determine the forces on the superstructure under the selected storm surge scenarios, and comparing those forces to the force capacity of the bolts. Failure Modes 2 and 3 were evaluated by analyzing lateral and axial pile loadings with the procedures in AASHTO's *Guide Specifications for Bridges Vulnerable to Coastal Storms* and *LRFD Bridge Design Specifications.*

Findings: The bridge is likely not vulnerable to Failure Mode 1, but could be vulnerable to Failure Modes 2 and 3.

Viable Adaptation Options:

- Design bridge to breakaway to minimize overall damage
- Strengthen bolt connections
- Install open grid decks
- Design shallower girder sections
- Use open rail parapets
- Shorten the bridge, replacing lower segments with protected embankment

Other Conclusions: The worst case storm scenario does not necessarily translate to the worst effects on the facility. Also, retreat might be a viable adaptation option, but further study is needed to determine the costs of benefits of doing so.

Application of the General Process for Transportation Facility Adaptation Assessments

Step 1 – Describe the Site Context

The case study bridge is located on the east side of the Mobile metropolitan area at Exit 30 on I-10 (see Figure 62). The interchange at Exit 30 is uniquely situated in the middle of Mobile Bay where the bridge carrying the Bayway intersects with the Causeway. The case study bridge connects US 90/98 to I-10 eastbound and is located on the southwest side of the interchange (see Figure 63). Land uses served by Exit 30 include the USS Alabama Battleship Memorial Park and a number of local businesses. Exit 30 also acts as an important connection between the Causeway and the Bayway: in the event that an incident disrupts traffic on one of the roads, motorists can use Exit 30 to access an alternate route across the Bay.

Step 2 – Describe the Existing Facility

The case study bridge was built in 1974. The bridge is 27.8 feet (8.5 meters) wide[229] and approximately 1,205 feet (367.4 meters) long from its base at the Causeway to its intersection with the viaduct carrying I-10 eastbound. A total of 29 bents,[230] each one assigned a unique identifying number, support the bridge from Bent 1 at the beginning of ramp to Bent 29 at the merge with the I-10 viaduct.

[229] Width as measured to the outside faces of the parapets. The parapets are the outside walls on either side of a bridge that are designed to prevent vehicles from careening off the structure.
[230] Bents, also known as piers, are the vertical columns supporting each bridge span along with the horizontal member, called a cap, which holds them together.

Figure 62: Location of Exit 30 within the Mobile Metropolitan Area[231]

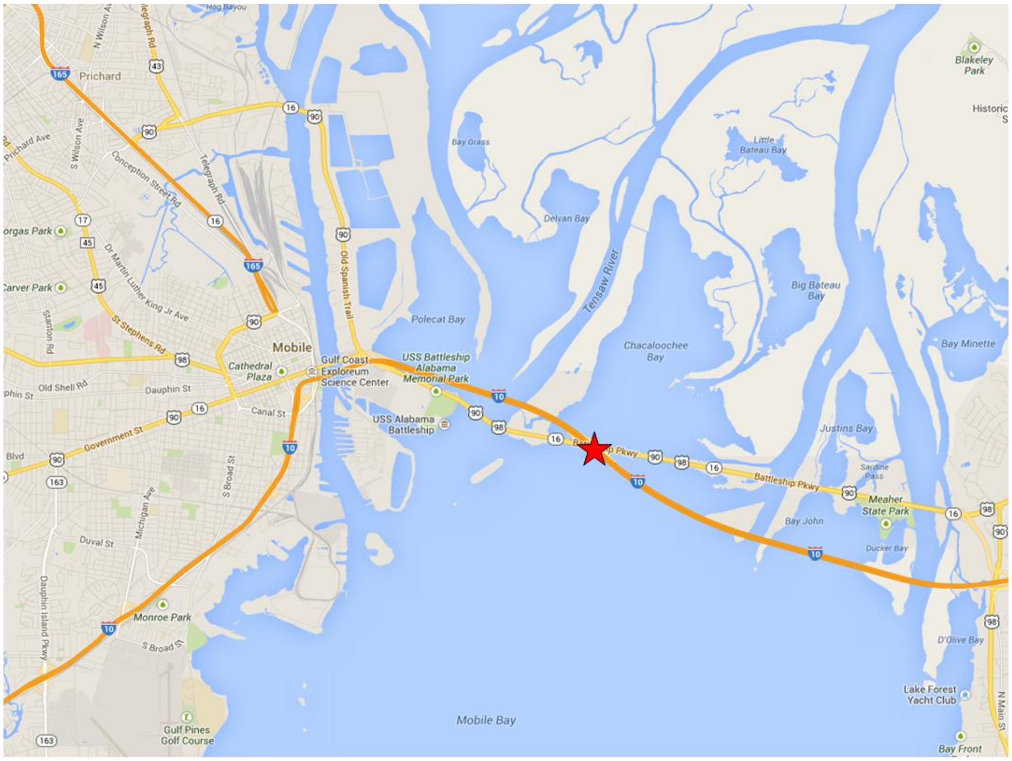
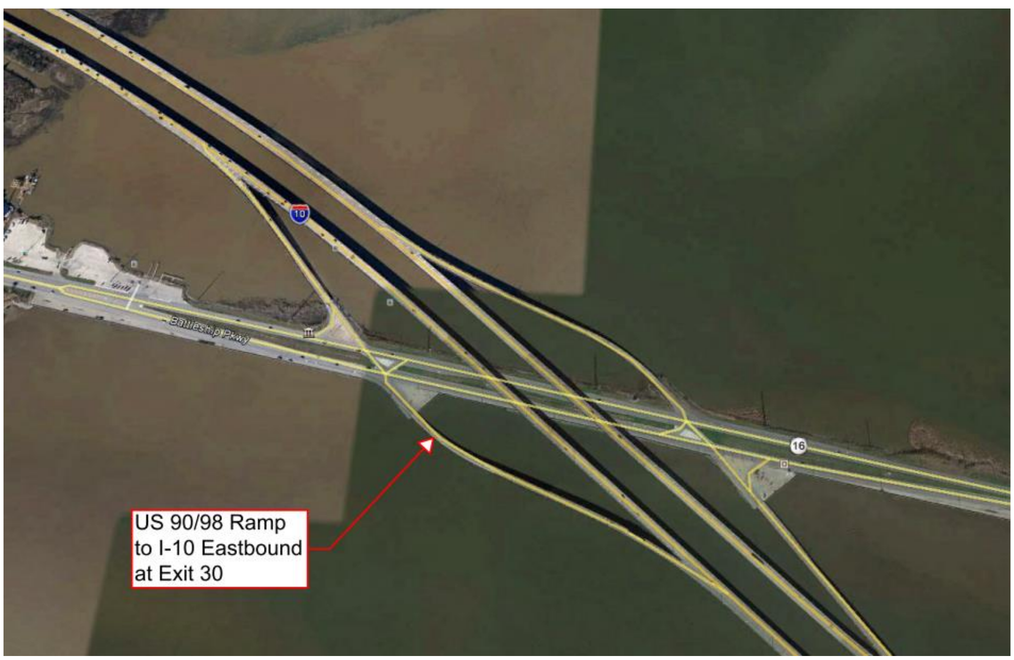


Figure 63: Location of the Ramp to I-10 Eastbound within the Exit 30 Interchange[232]



[231] Source of base map: Google Maps (as modified)
[232] Source of base map: Google Maps (as modified).

At the suggestion of ALDOT, the case study analysis focuses on the portion of the bridge between Bents 9 and 14 due to damages incurred on this segment from recent storm surge events associated with Hurricanes Katrina and Georges. This portion of the bridge is highlighted in the plan view of the structure shown in Figure 64 and the elevation shown in Figure 65.

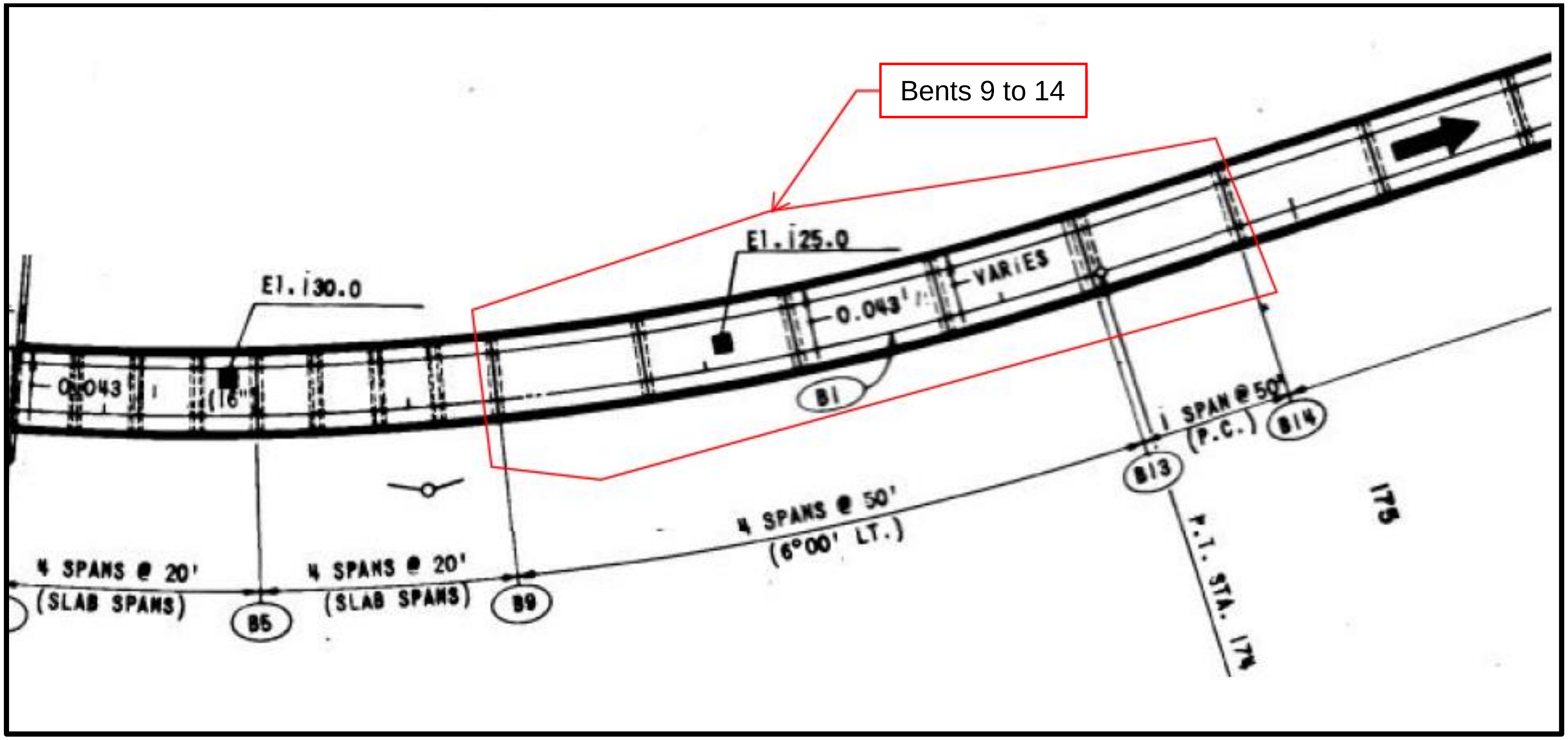
The five spans between Bents 9 and 14 are each 50 feet (15.2 meters) long making the portion of the ramp in this study 250 feet (76.2 meters) long. The superstructure[233] consists of a seven inch (178 millimeter) deck and four concrete beams spaced 7.3 feet (2.2 meters) apart (see Figure 66). The superstructure between Bents 1 and 9 is comprised of concrete slabs with no beams.

Under current sea levels, the (lower) Causeway end of the upward sloping bridge comes quite close to the Mean Higher High Water (MHHW)[234] elevation.[235] At the lowest bent, Bent 1, the clearance from the bottom of the slab to MHHW is approximately 1.2 feet (0.4 meter) and at the highest bent, Bent 29, the clearance from the bottom of the lowest beam to MHHW is 22.1 feet (6.7 meters). Within the segment being studied in this analysis, the lowest bent, Bent 9, has 2.3 feet (0.7 meter) clearance from MHHW whereas Bent 14, the highest bent, has 9.6 feet (2.9 meters) clearance from MHHW.

Most of the bents in the study segment consist of three 24 inch (60.1 centimeter) square concrete piles[236] topped with a concrete pile cap[237] (see Figure 67). The exception is Bent 13, which is a fixed anchor bent that does not allow the ends of the girders[238] to move, and contains six concrete piles (see Figure 68). The bents at the other sections of the ramp, Bents 1 to 8 (which support concrete slabs as the superstructure) and Bents 15 to 29 (which support concrete girders as the superstructure), are similar in construction to the section of ramp in this case study. The concrete beams are anchored to the top of the concrete pile caps with steel connection angles[239] and a series of bolts (see Figure 69). Bridge superstructures are typically connected to a bent (or pier) cap with a bearing and / or anchor bolts. The anchor bolts provides vertical uplift resistance and lateral force[240] resistance. In this case study bridge, the superstructure is connected to the bent cap with anchor bolts and horizontal through bolts (which penetrate through the entire width of the bottom of the concrete girders).

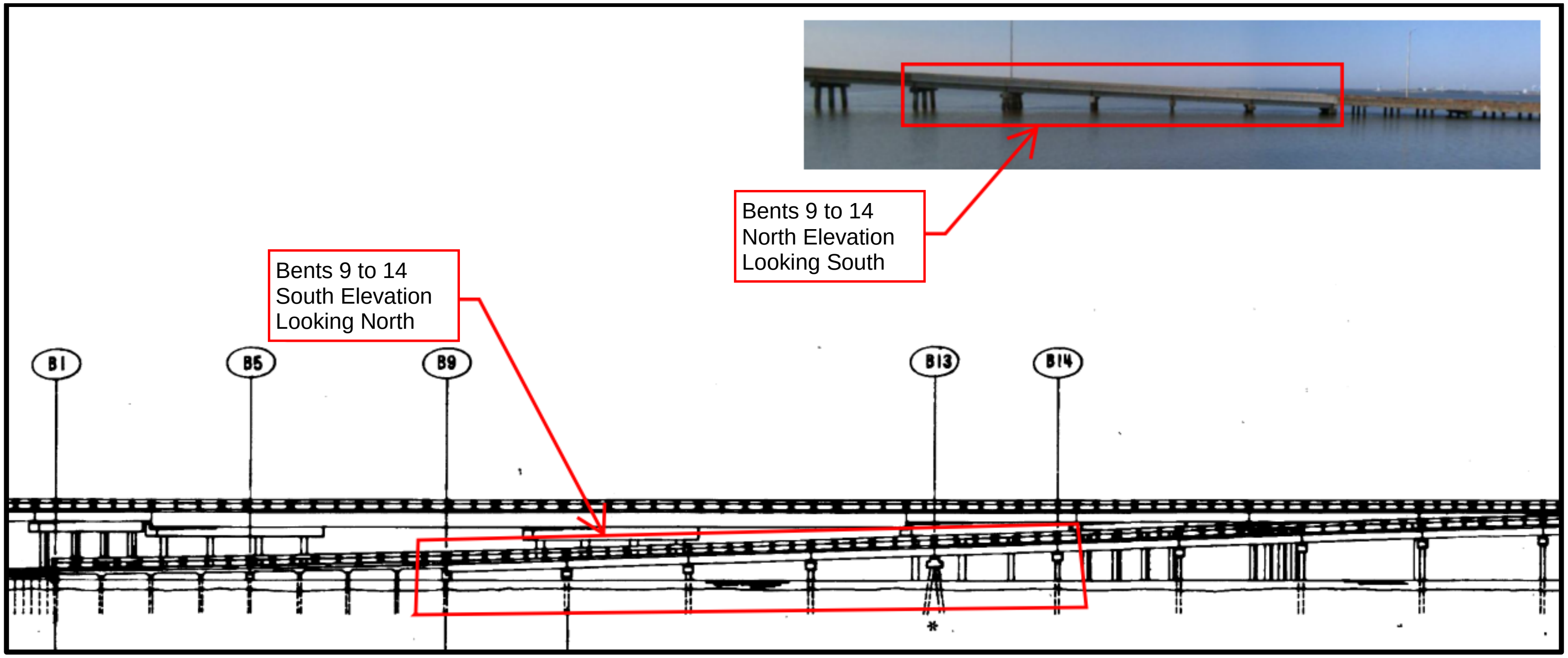
[233] The superstructure is the top part of the bridge and consists of the horizontal support girders, deck, and parapet walls preventing vehicles from falling off the structure.
[234] Mean higher high water is the average elevation of the highest daily high tide over the current National Tidal Datum Epoch. The tidal epoch is the specific 19-year period over which NOAA uses to obtain observations that are used to develop tidal statistics.
[235] All elevations within the text are with respect to the North American Vertical Datum of 1988 (NAVD88).
[236] Piles are the vertical support structures extending from the bridge deck to the seabed below.
[237] The pile cap is the horizontal member that ties together the vertical piles.
[238] Girders are the main horizontal supporting members of the bridge; there are four concrete girders for this case study bridge.
[239] Steel connection angles are used to connect the concrete girders to the concrete pile cap.
[240] Lateral forces are the horizontal component of a force affecting a structure.

Figure 64: Plan of the Bridge to I-10 Eastbound at Exit 30 Showing the Section of Analysis (Bents 9 to 14)[241]



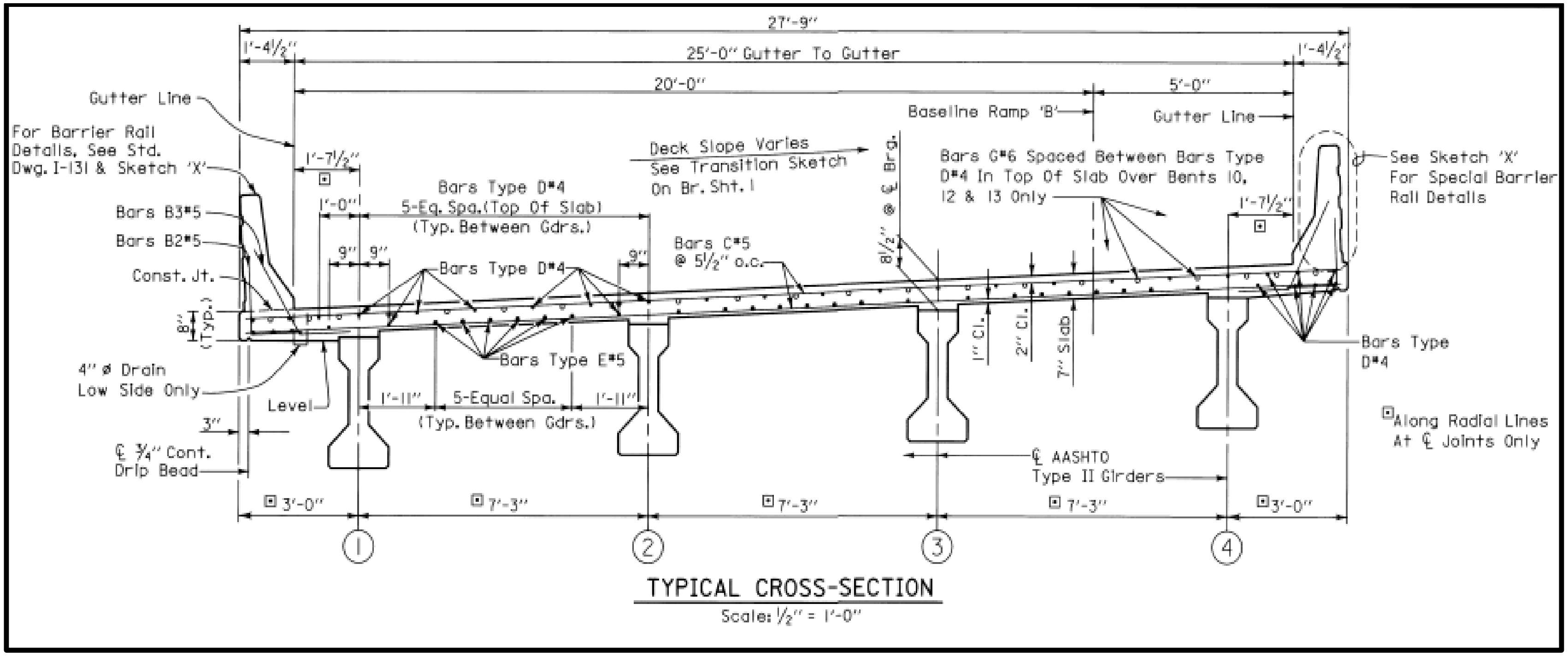
241 Source: ALDOT, 1974 (as modified)

Figure 65: Elevation of the Bridge to I-10 Eastbound at Exit 30 Showing the Section of Analysis (Bents 9 to 14)[242]



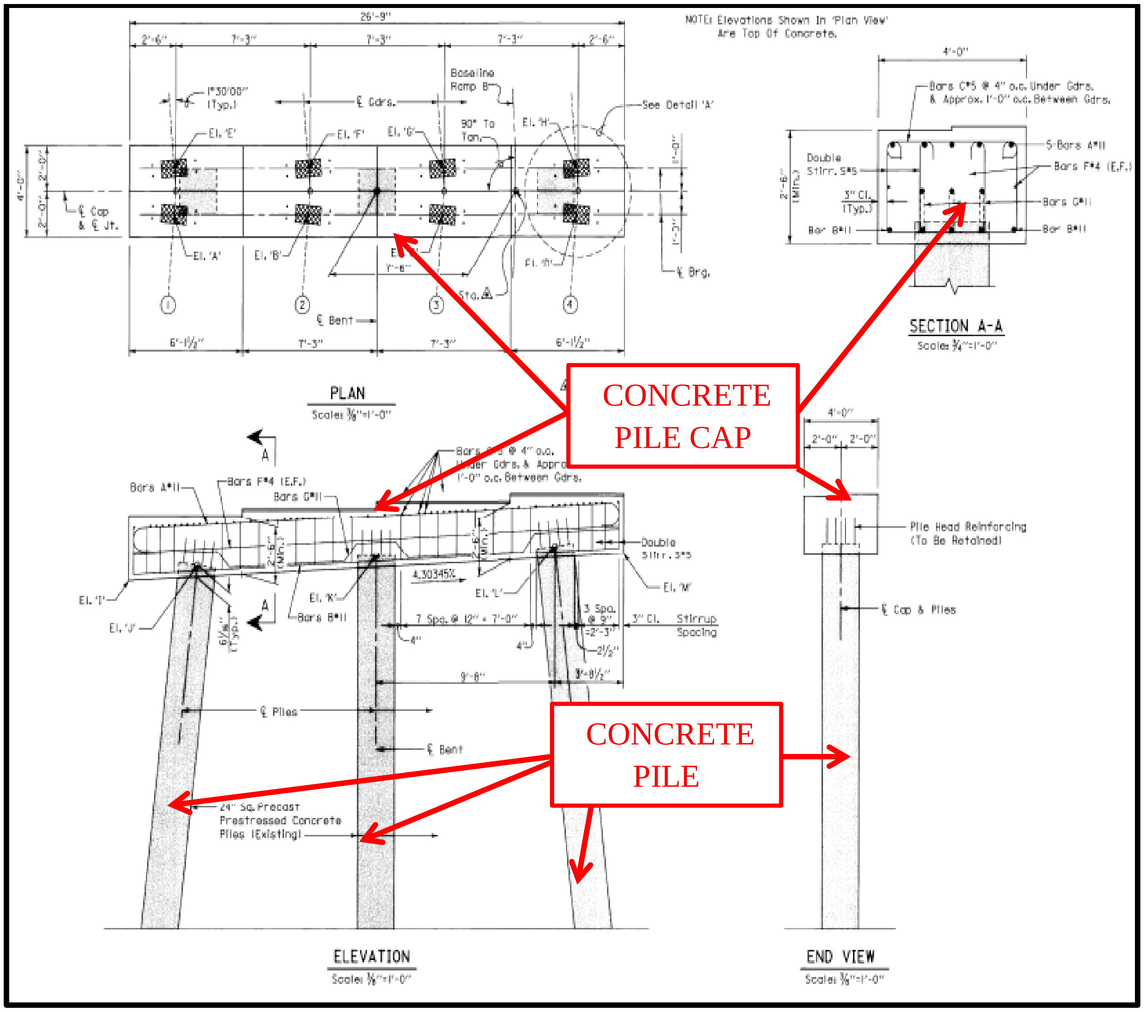
[242] Source: Elevation diagram from ALDOT, 1974 (as modified). Photo from Bing Maps (as modified).

Figure 66: Typical Section of the Superstructure between Bents 9 and 14 on the Bridge to I-10 Eastbound at Exit 30[243]



243 ALDOT, 2006

Figure 67: Typical Bent Details (Bents 10, 11, and 12) on the Bridge to I-10 Eastbound at Exit 30[244]



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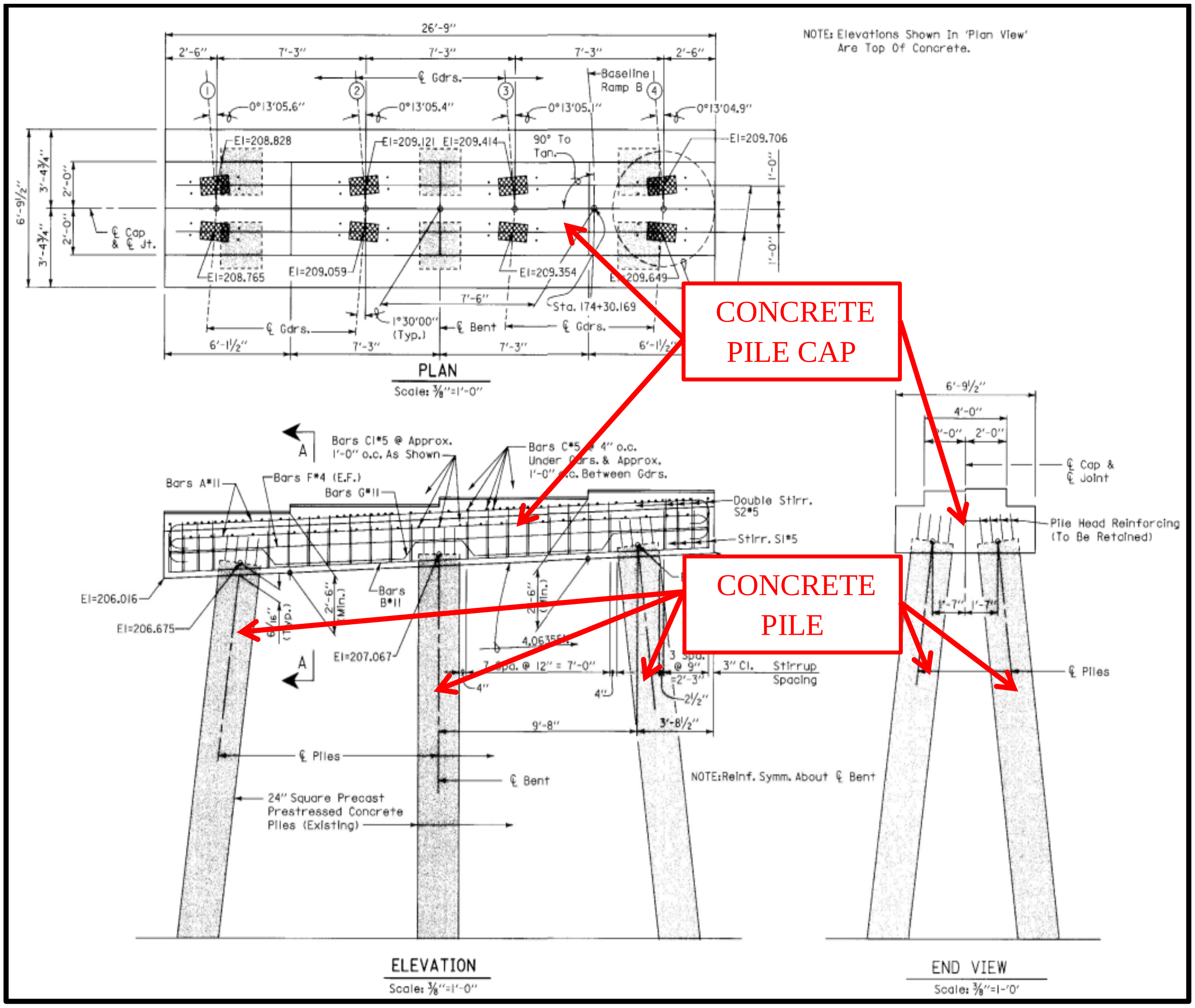
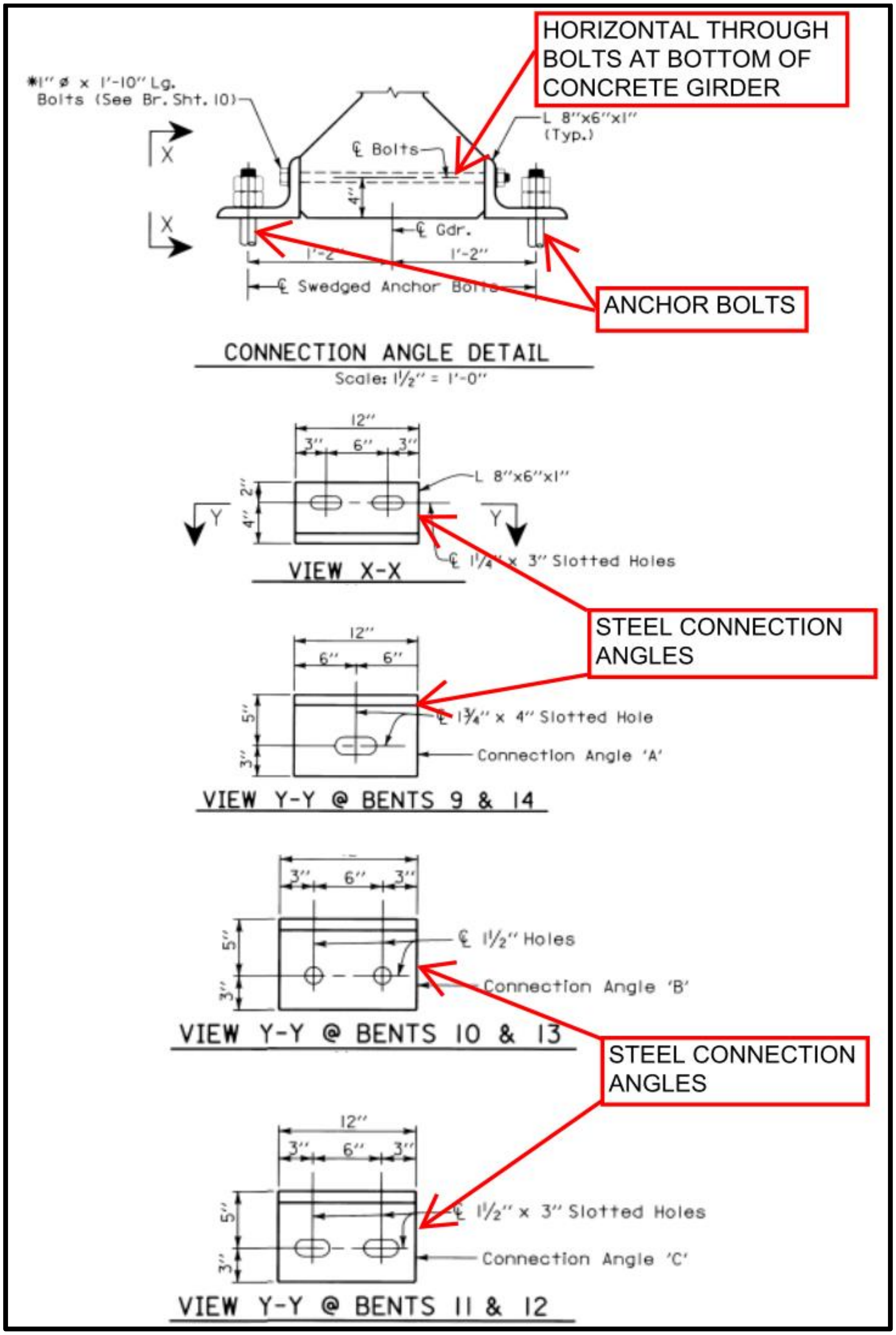
[244] Source: ALDOT, 2006 (as modified)

Figure 68: Bent 13 Details on the Bridge to I-10 Eastbound at Exit 30[245]



[245] Source: ALDOT, 2006 (as modified)

Figure 69: Steel Connection Angle and Bolt Details between Bents 9 and 14 on the Bridge to I-10 Eastbound at Exit 30[246]



246 Source: ALDOT, 2006 (as modified)

Since all the bents between Bents 9 and 14 are relatively similar in structure, this analysis focuses on the surge effects at only two bents (rather than at all six bents). An analysis of all the bents on this ramp would be recommended if it was decided to implement construction efforts to resist storm surge effects properly. Bent 11 was chosen to represent a typical bent for the section with three piles supporting 50 foot (15.2 meter) spans on either side of the bent. The second bent, Bent 13 was selected because it is unique and has six piles, rather than the typical three piles, and has a wider pile cap than the typical bent. Key elevations for Bents 11 and 13 are provided in Table 30.

Table 30: Key Elevations at Bents 11 and 13 on the Bridge to I-10 Eastbound at Exit 30

	Mean Sea Level Elevation[247] Feet (Meters)	MHHW Elevation Feet (Meters)	Bottom of Lowest Beam Elevation[248] Feet (Meters)	Highest Top of Deck Elevation[249] Feet (Meters)	Depth of Water Feet (Meters)	Mud Elevation[250] Feet (Meters)	Bottom of Pile Elevation Feet (Meters)
Bent 11	0.3 (0.1)	1.1 (0.3)	6.3 (1.9)	11.0 (3.3)	2.0 (0.6)	-1.7 (-0.5)	-72.7 (-22.2)
Bent 13	0.3 (0.1)	1.1 (0.3)	9.2 (2.8)	13.8 (4.2)	2.0 (0.6)	-1.7 (-0.5)	-73.7 (-22.5)

Step 3 – Identify Climate Stressors That May Impact Infrastructure Components

Storm surge in combination with sea level rise is a major concern for the facility. Any low lying facility such as this that is already relatively close to MHHW could be greatly impacted by storm surge and sea level rise. Historical storms have illustrated the threat: the storm surge caused by Hurricane Katrina dislodged the deck and girders of the ramp six feet (1.8 meters) to the north.[251]

Step 4 – Decide on Climate Scenarios and Determine the Magnitude of Changes

Three storm surge scenarios were developed for the assessment of storm surge-related impacts and infrastructure vulnerability. These consist of the following:

- **Hurricane Katrina Base Case Scenario**: This scenario represents the surge conditions that actually occurred in Mobile with Hurricane Katrina making landfall at the Louisiana-Mississippi border.

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[247] Mean sea level is the average of the water elevations recorded at each hour of the day over the current National Tidal Datum Epoch. The tidal epoch is the specific 19-year period over which NOAA uses to obtain observations that are used to develop tidal statistics.
[248] Due to the deck cross slope, the lowest beam elevation occurs at Beam 1, the beam closest to the inside face of the north parapet.
[249] Due to the deck cross slope, the highest top of deck elevation occurs at the top concrete deck surface at the base of the inside face of the south parapet.
[250] Mud elevation is the elevation of the top of the soil which is below the water surface.
[251] Cuomo, Shimosako, and Takahashi, 2009

- **Hurricane Katrina Shifted Scenario**: This scenario estimates the surge levels that would occur if Hurricane Katrina's path was shifted east to make landfall in Mobile.
- **Hurricane Katrina Shifted + Intensified + Sea Level Rise (SLR) Scenario**: This scenario estimates the surge levels that would occur if Hurricane Katrina was shifted, intensified with stronger winds due to climate change, and came on top of 2.5 feet (0.8 meters) of sea level rise.

A more detailed description of each scenario and how it was developed can be found in Section 4.4.4 of this document (under Step 4) and in the *Climate Variability and Change in Mobile, Alabama* report. [252] Table 31 shows the key storm surge characteristics at Bent 11 under each scenario. Table 32 shows this same information for Bent 13.

Table 31: Storm Surge Characteristics at Bent 11 on the Bridge to I-10 Eastbound at Exit 30 by Scenario

Storm Surge Scenario	Storm Surge Model Results[253]			Wave Model Results[254]		
	Water Surface Elevation Feet (Meters)	Sustained Wind Speed MPH (KPH)	Depth Averaged Current[255] Knots (KPH)	Wave Height[256] Feet (Meters)	Peak Wave Period[257] Seconds	Wave Direction Compass Degrees[258]
Hurricane Katrina Base Case	12.8 (3.9)	74 (119)	2.6 (4.8)	6.2 (1.9)	7.7	7
Hurricane Katrina Shifted	20.0 (6.10)	104 (167.3)	4.3 (8.0)	8.9 (2.7)	8.3	7
Hurricane Katrina Shifted + Intensified + SLR	24.9 (7.59)	110 (177.0)	4.4 (8.1)	4.4 (1.3)	8.3	7
FEMA[259] Base Flood Elevation (100-yr Flood Level)	17.0 (5.2)	-	-	-	-	-

[252] USDOT, 2012
[253] Simulations of storm-induced water levels (i.e., storm surge) and associated currents were performed using the two dimensional depth average version of the ADvanced CIRCulation model, ADCIRC.
[254] The wave characteristics accompanying each of the storm surge scenarios were simulated using the STeady State spectral WAVE (STWAVE) model.
[255] The two dimensional (depth averaged) version of the ADCIRC model calculates currents that represent the average current over the total depth at any given location. Thus, effects such as wind driven current variation with depth or smaller currents near the seabed are not included in the results.
[256] Zeroth moment wave height, H_{mo}, is equal to 4.0*square root (m_o) where m_o is the zeroth moment of the wave spectrum. For more detail refer to NOAA, 1996. In deep water the zeroth moment wave height is equal to the "significant wave height", Hs, which is the average of the highest one third of waves in a random wave field. In shallow water the significant wave height may be up to ten percent higher than H_{mo} (USACE, 2002).
[257] Peak wave period, Tp, is the period corresponding to the frequency band with the maximum value of spectral density in the non-directional wave spectrum as described in NOAA, 1996.
[258] Waves are propagating toward the indicated direction. Thus, zero degrees imply waves are propagating northward, whereas 90 degrees imply waves are propagating eastward.
[259] FEMA, 2007b

Table 32: Storm Surge Characteristics at Bent 13 on the Bridge to I-10 Eastbound at Exit 30 by Scenario

Storm Surge Scenario	Storm Surge Model Results[260]			Wave Model Results[261]		
	Water Surface Elevation Feet (Meters)	Sustained Wind Speed mph (kph)	Depth Averaged Current[262] Knots (kph)	Wave Height[263] Feet (Meters)	Peak Wave Period[264] Seconds	Wave Direction Compass Degrees[265]
Hurricane Katrina Base Case	12.8 (3.9)	74 (119)	2.3 (4.3)	6.2 (1.9)	7.7	7
Hurricane Katrina Shifted	20.0 (6.1)	104 (167)	3.9 (7.2)	8.9 (2.7)	8.3	7
Hurricane Katrina Shifted + Intensified + SLR	24.6 (7.5)	110 (177)	4.2 (7.8)	4.4 (1.3)	8.3	7
FEMA[266] Base Flood Elevation (100-yr Flood)	17.0 (5.2)					

Figure 70 shows the FEMA regulatory floodplain for the bridge crossing locations. The FEMA mapping shows a storm surge elevation of 17 feet (5.2 meters) at the bridge location. The crossing is in FEMA Zone VE, which is defined as an area of inundation with additional hazards due to storm-induced velocity wave action.

Step 5 –Assess Performance of the Existing Facility

To assess performance of the case study bridge under storm surge forces, three general failure modes were examined:

- **Failure Mode One** – The superstructure fails by wave uplifting and it washes away. It is assumed the deck slab and girders remain intact and that failure would occur at the superstructure-bearing components.[267]

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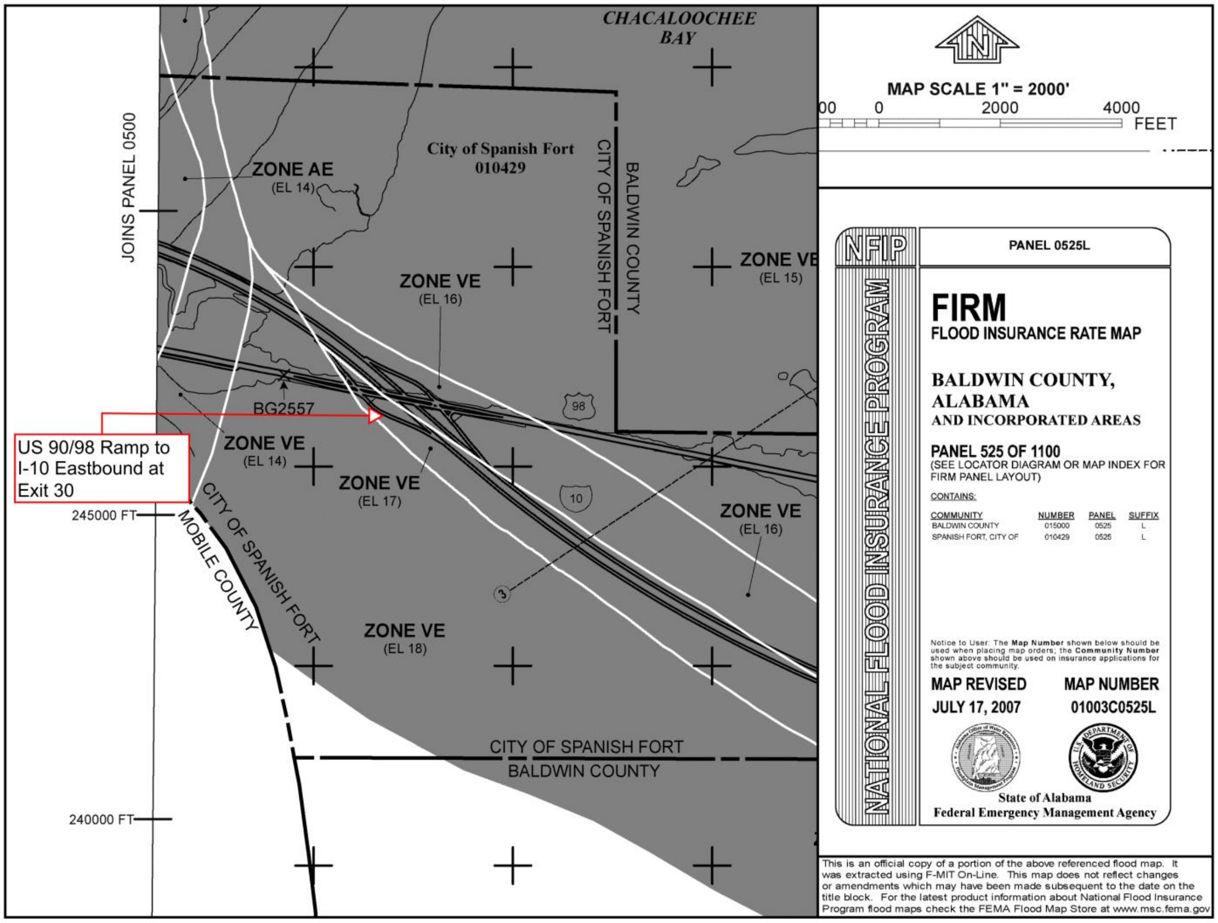
[260] Simulations of storm-induced water levels (i.e. storm surge) and associated currents were performed using the two dimensional depth average version of the ADvanced CIRCulation model, ADCIRC.
[261] The wave characteristics accompanying each of the storm surge scenarios were simulated using the STeady State spectral WAVE (STWAVE) model.
[262] The two dimensional (depth averaged) version of the ADCIRC model calculates currents that represent the average current over the total depth at any given location. Thus, effects such as wind driven current variation with depth or smaller currents near the seabed are not resolved in the results.
[263] Zeroth moment wave height, H_{mo}, is equal to 4.0*square root (m_o) where m_o is the zeroth moment of the wave spectrum. For more detail refer to NOAA, 1996. In deep water the zeroth moment wave height is equal to the "significant wave height", Hs, which is the average of the highest one third of waves in a random wave field. In shallow water the significant wave height may be up to ten percent higher than H_{mo} (USACE, 2002).
[264] Peak wave period, Tp, is the period corresponding to the frequency band with the maximum value of spectral density in the non-directional wave spectrum as described in NOAA, 1996.
[265] Waves are propagating toward the indicated direction. Thus, zero degrees imply waves are propagating northward, whereas 90 degrees imply waves are propagating eastward.
[266] FEMA, 2007b
[267] Superstructure bearing components are typically bearing elements that connect the bottom of the girders or beams to the pier cap.

Figure 70: FEMA Flood Insurance Rate (FIRMette) Map for the Bridge to I-10 Eastbound at Exit 30[268]



- **Failure Mode Two** – The substructure[269] fails due to lateral forces applied from the waves or gets uprooted by the upward vertical forces acting on the superstructure. The majority of the lateral and vertical wave loads act on the superstructure and are transmitted to the substructure. Substructure conditions with and without scour were calculated and analyzed. In the analysis of this failure mode, it is determined if the substructure will remain intact with the superstructure attached.
- **Failure Mode Three** – The substructure fails due to excessive scour. Excessive scour at any bridge foundation leads to bridge instability and eventual failure. There are also different types of scour (which are discussed below) that can occur at a bridge foundation. Each storm surge scenario is investigated with and without scour. The scour is assumed to be caused by the particular storm surge scenario being investigated.

Under all three storm surge scenarios described in Step 4, the superstructure and substructure at both Bents 11 and 13 are inundated. Thus, it is assumed that the case study section of the bridge

[268] FEMA, 2007b. Note: The elevations shown are in NAVD88.
[269] The substructure in this case study bridge consists of concrete piles and the pile cap connecting the concrete piles.

will not be used by the public and trucks and cars will not be imposing live load on the structure during the storm surge event. It should also be noted that under the Hurricane Katrina Shifted + Intensified + SLR scenario the sea level rise of 2.5 feet (0.8 meters) would itself place parts of the concrete deck at the beginning of the ramp between Bents 1 and 9 under water. When a portion of the top of the deck is under water, it is assumed that the bridge is out of service until adaptations or remedial actions are put in place.

Design guidance documents used in the analysis of the effects of storm surge (and sea level rise) on bridges include the following:

- American Association of State Highway and Transportation Officials (AASHTO) *Load and Resistant Factor Design (LRFD) Bridge Design Specifications*[270]
- AASHTO *Guide Specifications for Bridges Vulnerable to Coastal Storms*[271]

The *LRFD Bridge Design Specifications* provides the equations used for a structural analysis of a bridge whereas the *Guide Specification for Bridges Vulnerable to Coastal Storms* provides the equations used to develop the wave forces acting on a bridge during a storm surge.

The remainder of this section discusses the assessment of each failure mode for the existing bridge under each of the storm surge scenarios described in Step 4.

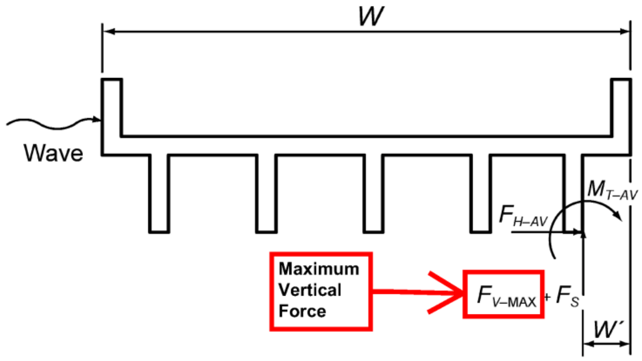
Failure Mode One - Superstructure Failure

A storm surge (with its associated waves) that encounters a bridge imparts vertical and horizontal forces on the superstructure. Typically, it is the combination of both forces that causes a superstructure to fail. The vertical forces include dynamic forces from the waves, a buoyancy force if it submerges the bridge, and a vertical slamming force, which can impact the underside of the superstructure. The horizontal force is the sum of the wave and current induced forces. The *Guide Specification for Bridges Vulnerable to Coastal Storms* provides three Design Cases for a superstructure and the equations (for each Design Case) to determine the vertical and horizontal forces on a superstructure using data from storm surge and wave models (see Figure 71).

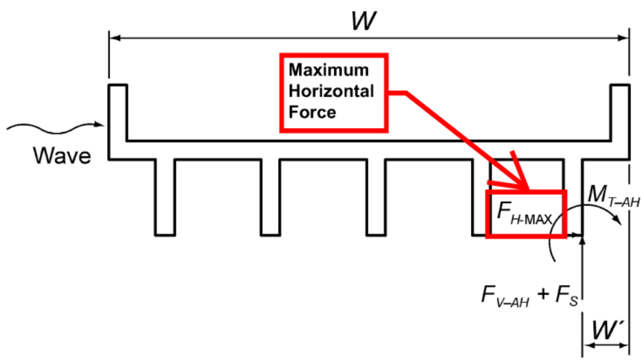
Design Case I maximizes the vertical wave force and is used to design the resistance of the superstructure from the substructure. Design Case II maximizes the horizontal wave force and is used to design the resistance of bents and horizontal restraints. Design Case III determines the forces on the overhang and parapet portion of the bridge. Design Case III will not be investigated in this case study because the damage of an overhang or parapet is a localized failure that would not cause the bridge to collapse or place it completely out of service: one might wish to investigate this if a full adaptation assessment were being conducted.

[270] AASHTO, 2012
[271] AASHTO, 2008

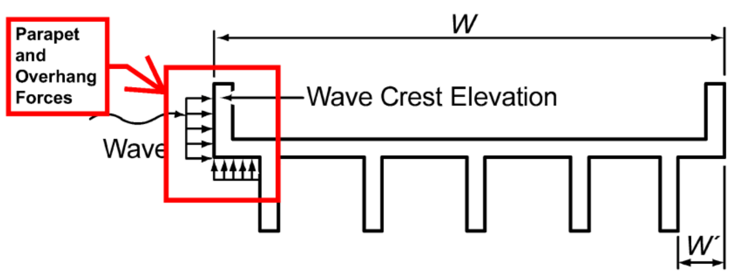
Figure 71: Illustration of Superstructure Force Design Cases[272]



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



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



(c) Case III—Assumed Prorated Overhang Design Forces

It should be noted that determining all the possible modes of concrete failure[273] in the concrete beams and concrete pile cap was beyond the scope of this study. Therefore, it was assumed in this failure mode that concrete failures in the pile cap or in the concrete beams would not occur. For this case study and in all concrete structures, reinforcing bars embedded in properly designed concrete pile caps and in properly designed concrete beams are assumed to sufficiently prevent a concrete failure. Also, a majority of the bridge failures caused by Hurricane Katrina typically failed due to the connection between the superstructure and substructure.[274]

[272] Source: AASHTO, 2008 (Figure 6.1.2.1-1) (as modified)
[273] Concrete failure would be a large spall or shear crack
[274] Curtis, 2007

In this case study, failure is assumed to occur at either the anchor bolts (embedded into the concrete pile cap, see Figure 69) or horizontal through bolts (located at the bottom of the concrete girders, see Figure 69). For a typical 50 foot (15.2 meter) span, the vertical bolt capacity was 542.6 kips[275] (2,413.6 kilonewtons) and the horizontal bolt capacity was 1,221.4 kips (5,433.1 kilonewtons). These capacities were based on the type and size of bolts as specified in the ALDOT drawings (see Figure 69) and in accordance with the 2012 *AASHTO LRFD Bridge Design Specifications.*

The data shown in Table 31 and Table 32 from the storm surge and wave models were used to calculate wave forces. The various forces were calculated and the load factors[276] in the *LRFD Bridge Design Specifications* were applied before comparing to the capacities of the bolts securing the bridge. Table 33 illustrates the compiled superstructure forces at Bent 11 and Table 34 illustrates the compiled superstructure forces at Bent 13 under each of the storm surge scenarios. Failure of the bolts to secure the superstructure occurs when the applied forces (vertical or horizontal) from the waves exceed the bolt capacity. The tables show the bolts failing from the vertical forces under all the storm surge scenarios in Design Case I for Bents 11 and 13, under the Hurricane Katrina Base Case Scenario in Design Case II for Bent 11, and under all the storm surge scenarios in Design Case II for Bent 13. The bolts do not fail from the horizontal loads in any of the storm surge scenarios.

Vertical and horizontal forces vary between the different storm surge scenarios, Bents 11 and 13, and the two Design Cases. This is due to dynamic wave forces in play, how far under water the superstructure is submerged and the maximum forces each of the Design Cases are trying to calculate. The farther below the water the superstructure is, the dynamic horizontal wave forces are less intense and the water velocity would be less, thus causing the magnitude of the lateral forces to be less. The vertical wave and vertical slamming forces increase the farther below the water superstructure is submerged thus increasing the vertical uplift forces.

To summarize, the superstructure at Bents 11 and 13 would likely have bolt failures at the bottom of the girders and lift off and wash away under all of the storm surge scenarios investigated. Because Bent 11 is typical of the other bents in the case study segment, it is likely that this failure could occur at any of the spans within the study section of the ramp.

[275] One kip, also referred to as a kilopound, is equal to 1,000 pounds-force (4.4 kilonewtons).
[276] Load factors may be thought of as safety factors that are applied to the loads.

Table 33: Superstructure Forces at Bent 11 on the Bridge to I-10 Eastbound at Exit 30

Storm Surge Scenario	Design Case I		Design Case II	
	Applied Vertical Forces kips (kilonewtons)	Applied Horizontal Forces kips (kilonewtons)	Applied Vertical Forces kips (kilonewtons)	Applied Horizontal Forces kips (kilonewtons)
Hurricane Katrina Base Case	5,450.4 (24,244.6)	85.7 (381.2)	1,374.4 (6,113.6)	141.9 (631.2)
Hurricane Katrina Shifted	10,030.5 (44,617.9)	72.3 (321.6)	533.2 (2,371.8)	85.5 (380.3)
Hurricane Katrina Shifted + Intensified + SLR	11,754.3 (52,285.7)	39.1 (173.9)	512.2 (2,278.4)	57.5 (255.8)
Bolt Capacity For Four Girders in a Single Bridge Span, kips (kilonewtons)	542.6 (2,413.6)	1,221.4 (5,433.1)	542.6 (2,413.6)	1,221.4 (5,433.1)
Failure (Yes/No)	Yes, for all scenarios	No, for all scenarios	Yes, for Katrina Base Case	No, for all scenarios

Table 34: Superstructure Forces at Bent 13 on the Bridge to I-10 Eastbound at Exit 30

Storm Surge Scenario	Design Case I		Design Case II	
	Applied Vertical Forces kips (kilonewtons)	Applied Horizontal Forces kips (kilonewtons)	Applied Vertical Forces kips (kilonewtons)	Applied Horizontal Forces kips (kilonewtons)
Hurricane Katrina Base Case	3,591.5 (15,975.8)	107.8 (479.5)	1,478.4 (6,576.3)	181.9 (809.1)
Hurricane Katrina Shifted	8,637.0 (38,419.3)	83.8 (372.8)	787.1 (3,501.2)	102.7 (456.8)
Hurricane Katrina Shifted + Intensified + SLR	10,245.2 (45,572.9)	38.6 (171.7)	756.6 (3,365.5)	62.8 (279.3)
Bolt Capacity For Four Girders in a Single Bridge Span, kips (kilonewtons)	542.6 (2,413.6)	1,221.4 (5,433.1)	542.6 (2,413.6)	1,221.4 (5,433.1)
Failure (Yes/No)	Yes, for all scenarios	No, for all scenarios	Yes, for all scenarios	No, for all scenarios

Failure Modes Two and Three - Substructure Failure

Failure in the substructure during a storm surge is caused by vertical and / or horizontal loads transferred from the superstructure to the substructure and subsequently overloading elements (in this case study, the concrete piles) of the substructure. In this case study, the vertical and horizontal forces applied to the substructure are derived from the superstructure forces and from the *Guide Specification for Bridges Vulnerable to Coastal Storms*.[277] A geotechnical computer model (simulating soil conditions) was created using GROUP 8.0 software. It uses the forces as inputs, and the results are compared to allowable geotechnical and structural capacities of the pile.

Possible failure modes of the substructure investigated in this case study were:

- **Uplift of the piles and pile cap from vertical wave forces:** Vertical forces are applied and the soil uplift resistance[278] and axial tensile (structural) capacity[279] of the pile are checked for sufficiency
- **Failure of piles from lateral forces:** Lateral forces are applied and piles are checked for shear[280] and bending[281] sufficiency. The geotechnical software, GROUP 8.0, determined the maximum lateral force (shear and bending) on a single pile based on calculated lateral wave forces on the pile cap and the assumed soil profile (with and without scour). This lateral force was then compared to the structural pile lateral capacity (shear and bending) to determine if it passed or failed.

The lateral and axial pile loading analyses were performed in accordance with the procedures outlined in the *Guide Specification for Bridges Vulnerable to Coastal Storm*[282] and the *LRFD Bridge Design Specifications*[283] using various combinations of the wave loads provided for the superstructure and the bents.

This failure modes assume the superstructure is attached to the substructure up to the failure point of the bolts as described in Failure Mode 1. Thus, if the owner of the bridge should decide to reinforce the bolts (or change the bolt configuration) as an adaptation option in response to Failure Mode 1, this mode of failure would need to be re-evaluated. This particular study did not include the effects of a ship collision on a pier (this is covered in the *LRFD Bridge Design Specifications* document) because it is not over a navigable channel. However, the owner may

[277] AASHTO, 2008
[278] Soil resistance for concrete piles is defined as the resistance derived from soil friction and soil cohesion.
[279] Axial tensile capacity of the pile is defined as the structural capacity of the pile to deal with tension. The concrete piles have a calculated structural tensile capacity of 386 kips (1,717 kilonewtons) in accordance to the *LRFD Bridge Design Specifications* document.
[280] The tendency for a pile to be sheared is caused by the applied horizontal loads. The concrete piles have a calculated structural shear capacity of 55 kips (245 kilonewtons) in accordance to the *LRFD Bridge Design Specifications* document.
[281] Bending is the flexure of the pile caused by the application of horizontal loads. The concrete piles have a calculated structural bending moment capacity (the highest stress experienced within a material under bending at its moment of rupture) of 233 feet-kips (316 kilonewton-meters) in accordance to the *LRFD Bridge Design Specifications* document.
[282] AASHTO, 2008
[283] AASHTO, 2012

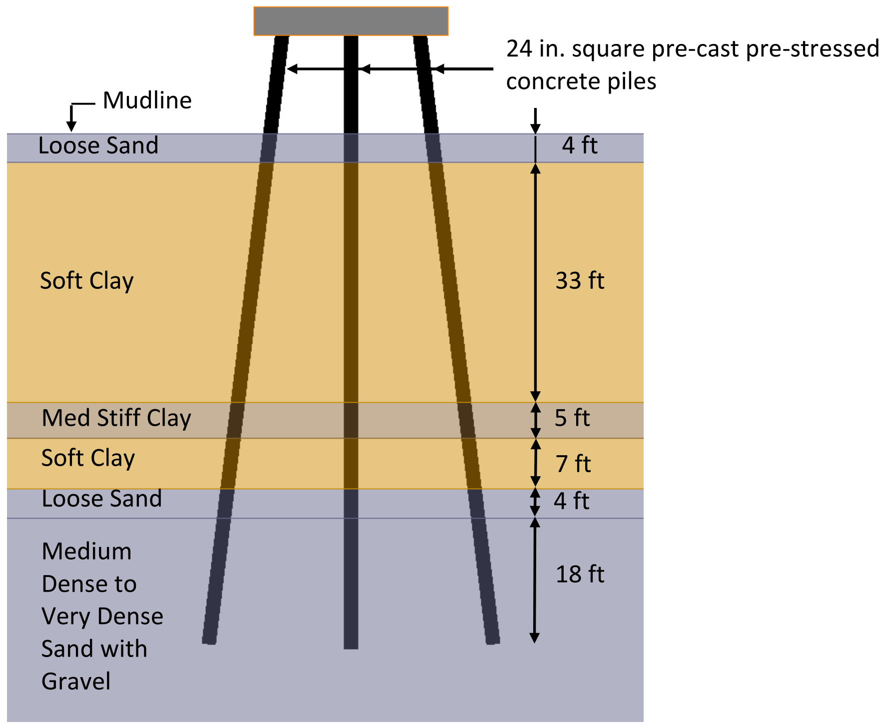
need to evaluate whether vessels or large debris in a storm surge scenario might impact the bridge and whether such an impact should be considered in the bridge design.

The pile dimensions and length were developed based on the as-built plans and the pile driving records. The as-built plans indicate that Bent 11 consists of a single row of three 24 inch (61 centimeter) square pre-cast pre-stressed concrete piles while Bent 13 consists of two rows of three 24 inch (61 centimeter) square pre-cast pre-stressed concrete piles. The pile driving records indicate that the piles were driven to a depth of 71 feet (21.6 meters) and 72 feet (21.9 meters) below the mudline for Bents 11 and 13, respectively.

The subsurface profiles at Bents 11 and 13 were developed from the boring logs provided on the as-built plans. Because no information other than qualitative descriptions of the soil strata is provided on the boring logs (i.e., no Standard Penetration Test (SPT) [284] blow counts or laboratory test results), the engineering properties for the soil layers used in the lateral and axial pile capacity analyses were derived using typical values reported in the geotechnical engineering literature.[285] An initial analysis was performed to calibrate the assumed engineering properties based on an assumption that the foundations at Bents 11 and 13 performed acceptably during Hurricane Katrina (i.e., there was no pull-out of the piles or piles were not forced out of alignment or rotated under the Hurricane Katrina Base Case Scenario). Because the subsurface characteristics were assumed to be similar for Bents 11 and 13, the generalized subsurface profile at Bent 11 is shown as an example below in Figure 72.

[284] The standard penetration test is an in-situ dynamic penetration test designed to provide information on the geotechnical engineering properties of soil.
[285] Duncan, Horz, and Yang, 1989; Holtz and Kovacs, 1981

Figure 72: Generalized Subsurface Profile at Bent 11 on the Bridge to I-10 Eastbound at Exit 30[286]



Scour of the bridge piers is another key consideration in any analysis of substructure failure. Scour occurs by water forces eroding the soil support where a pile and mudline meet, reducing each pier's capacity to handle the forces on it by reducing the embedment length of a pile. Reduction of the embedment length exposes the pile to more stream and wave forces and subjects the pile to increased stresses and reduces pile lateral capacity if the piles are embedded in competent soils. Scour computations were performed for each of the bents following the guidance provided in the Federal Highway Administration's (FHWA) HEC-18[287] publication. Scour at bridge piers is computed as the sum of **contraction scour, long term trends,** and **local pier scour**.

Contraction scour is the general erosion of the channel bed due to the contraction and acceleration of flows as they transition from a wider area into a narrower bridge crossing. The theoretical basis of contraction scour is centered on the acceleration of flows through a bridge opening. Because the study area bridge is bounded to the north by the causeway and a bulkhead shoreline, flows will not be accelerating through this area and contraction scour will be negligible. Pressure scour is also a form of contraction scour that is often a consideration for

[286] Note: No scour conditions is shown and, for clarity, the superstructure is not shown
[287] Arneson et al., 2012

bridges where flooding (or storm surge) conditions result in the bridge deck impeding the flood flow. Under pressure scour, a contraction of flows occurs vertically and the same general theory of accelerating flows causing general scouring of the channel bed occurs. However, in the case of the study site, there is sufficient opportunity for the storm surge to flank the bridge opening, thus raising the water surface on the opposite side of the bridge opening and negating the primary mechanisms for vertical contraction and formation of pressure flow; thus pressure scour is also assumed to be negligible.

A **long term trend** refers to the process where a water body undergoes a long term change in its bed elevation. This change can be thought of as occurring over the course of years or decades, as opposed to occurring during a single storm event as is the case with the other types of scour. In general for most Eastern estuaries, the processes of sea level rise and associated sedimentation is generally expected to result in increasing sea bed elevations.[288] Thus, some degree of sedimentation at the case study bridge is assumed to occur over the long term. However, because of the uncertainty in quantifying future sea bed levels, the analysis of the bridge foundations focuses on current sea bed levels to ensure foundation stability under current conditions, with the potential of improved conditions in the future. Another example of a long term trend would be lateral migration of barrier islands and resulting changes to seabed elevations as inlets expand, close up, and migrate laterally. The Bonner Bridge in the Outer Banks of North Carolina is a classic example. In this particular case, the shoreline has been stabilized with timber bulkheads which minimizes the potential of lateral migration. This type of long term trend is not a factor at this case study facility.

The last component is **local pier scour**. Local pier scour is caused by the vertical obstruction to flows caused by the physical presence of the pier in the water. Local scour occurs as horizontal flow velocity is converted into vertical turbulence caused by water flows going around a pier. The vertical turbulence, known as a horseshoe vortex, effectively removes soil at the base of the pier. Additional turbulent formations known as wake vortices form downstream of the pier in a flow vacuum created opposite the water flow. The wake vortices have a similar effect as the horseshoe vortex in the removal of soil from the backside of a pier foundation. In the case study, the depth of local pier scour was estimated using the Sheppard pier scour equation (also known as the Florida DOT Pier Scour Methodology) which is published in FHWA’s HEC-18 manual[289] and Florida’s *Bridge Scour Manual*.[290] The Sheppard pier scour methodology has been documented as an improvement over traditional HEC-18 methods as it provides improved considerations for soil material types while still incorporating considerations for flow depth, velocity, and angle of attack into the pier scour computations.

[288] Duncan, Goff, Austin, and Fulthorpe, 2000
[289] Arneson et al., 2012
[290] FDOT, 2011

The axial-lateral loading analysis of the Bent 11 and 13 foundations were performed using a three-dimensional soil-structure interaction analysis software called GROUP 8.0.[291] The results of these analyses show the maximum lateral loads,[292] vertical loads,[293] and bending moment[294] acting on the pile cap and are presented in terms of maximum bending moment,[295] maximum shear force,[296] and maximum tension load[297] action on each individual pile. The maximum tension load is compared with total pile uplift resistance[298] for each pile. The maximum structural axial, shear, and moment loads were compared to the corresponding structural capacity. Table 35 and Table 36 show the inputs to the GROUP 8.0 software, including the calculated scour depths. Results are shown in Table 37 and Table 38 with and without scour considerations for each scenario. Failure of the substructure occurs when either vertical uplift forces exceed the vertical soil resistance or structural tensile capacity of the piles causing a pile to pull out or lateral forces exceed lateral geotechnical or structural capacity of the pile causing a pile to deflect excessively or break.

[291] Reese, Wang, Arrellaga, Hendrix, and Vasquez, 2010
[292] Lateral loads are the applied horizontal loads.
[293] Vertical loads are the uplift forces derived from upward wave forces acting on the superstructure.
[294] Bending moment is a flexure force on a pile section caused by the applied horizontal loads.
[295] Maximum bending moment at each pile is determined after compiling all loads (for a particular storm surge scenario) into the GROUP 8.0 computer model. This process is performed for each of the three storm surge scenarios.
[296] Maximum shear force at each pile is determined after compiling all loads (for a particular storm surge scenario) into the GROUP 8.0 computer model. This process is performed for each of the three storm surge scenarios.
[297] Maximum tension load at each pile is determined after compiling all loads (for a particular storm surge scenario) into the GROUP 8.0 computer model. This process is performed for each of the three storm surge scenarios.
[298] Total pile uplift resistance is derived from soil friction and soil cohesion acting on the pile.

Table 35: Substructure Input Loads at Bent 11 on the Bridge to I-10 Eastbound at Exit 30

Storm Surge Scenario	Scour Depth Feet (Meters)[299]	Axial Tension Load kips (kN)	Lateral Load kips (kN)	Bending Moment Kip-Ft (kN-meter)
Hurricane Katrina Base Case	No Scour	543 (2,415)	165 (734)	62 (84)
	3.1 (0.9)			
Hurricane Katrina Shifted	No Scour	543 (2,415)	117 (520)	100 (136)
	4.7 (1.4)			
Hurricane Katrina Shifted + Intensified + SLR	No Scour	543 (2,415)	75 (334)	60 (81)
	4.7 (1.4)			

Table 36: Substructure Input Loads at Bent 13 on the Bridge to I-10 Eastbound at Exit 30

Storm Surge Scenario	Scour Depth Feet (Meters)[300]	Axial Tension Load kips (kN)	Lateral Load kips (kN)	Bending Moment Kip-Ft (kN-meter)
Hurricane Katrina Base Case	No Scour	543 (2,415)	239 (1,063)	282 (382)
	4.3 (1.3)			
Hurricane Katrina Shifted	No Scour	543 (2,415)	175 (778)	464 (629)
	5.0 (1.5)			
Hurricane Katrina Shifted + Intensified + SLR	No Scour	543 (2,415)	94 (418)	215 (292)
	5.0 (1.5)			

[299] The analysis was run with and without scour conditions.
[300] The analysis was run with and without scour conditions.

Table 37: Substructure Results at Bent 11 on the Bridge to I-10 Eastbound at Exit 30

Storm Surge Scenario	Scour Depth Feet (Meters)	Results from GROUP Analysis: Maximum Bending Moment Kip-Feet (kN-Meter)	Results from GROUP Analysis: Maximum Shear kips (kN)	Results from GROUP Analysis: Maximum Tension Load per Pile kips (kN)	Total Pile Soil Axial Tensile Resistance kips (kN)	Pile Soil Axial Tensile Resistance Sufficient?[301]	Pile Structural Axial Capacity Sufficient?[302]	Pile Structural Shear Capacity Sufficient?[303]	Pile Structural Bending Moment Capacity Sufficient?[304]
Hurricane Katrina Base Case	No Scour	1,142 (1,549)	63 (280)	200 (890)	232 (1,032)	Yes	Yes	No	No
	3.1 (0.9)	1,250 (1,695)	69 (307)	200 (890)	222 (988)	Yes	Yes	No	No
Hurricane Katrina Shifted	No Scour	792 (1,074)	45 (200)	193 (859)	232 (1,032)	Yes	Yes	Yes	No
	4.7 (1.4)	917 (1,243)	54 (240)	190 (845)	216 (961)	Yes	Yes	Yes	No
Hurricane Katrina Shifted + Intensified + SLR	No Scour	475 (644)	29 (129)	188 (836)	232 (1,032)	Yes	Yes	Yes	No
	4.7 (1.4)	563 (763)	36 (160)	188 (836)	216 (961)	Yes	Yes	Yes	No

[301] Total pile tensile resistance is compared to maximum tension load per pile to determine if the soil can resist uplift forces acting on the pile without failure.
[302] The concrete piles have a calculated structural tensile capacity of 386 kips (1,717 kilonewtons) in accordance to the *LRFD Bridge Design Specifications* document. This value is compared to the maximum tension load per pile to determine if the pile will break due to the tensile load.
[303] The concrete piles have a calculated structural shear capacity of 55 kips (245 kilonewtons) in accordance to the *LRFD Bridge Design Specifications* document. This value is compared to the maximum shear load per pile to determine if the pile will break due to the shear load.
[304] The concrete piles have a calculated structural bending moment capacity of 233 feet-kips (316 kilonewton-meters) in accordance to the *LRFD Bridge Design Specifications* document. This value is compared to the maximum bending moment load per pile to determine if the pile will break due to the shear load.

Table 38: Substructure Results at Bent 13 on the Bridge to I-10 Eastbound at Exit 30

Storm Surge Scenario	Scour Depth Feet (Meters)	Results from GROUP Analysis			Total Pile Soil Axial Tensile Resistance kips (kN)	Pile Soil Axial Tensile Resistance Sufficient?[305]	Pile Structural Axial Capacity Sufficient?[306]	Pile Structural Shear Capacity Sufficient?[307]	Pile Structural Bending Moment Capacity Sufficient?[308]
		Maximum Bending Moment Kip-Feet (kN-Meter)	Maximum Shear kips (kN)	Maximum Tension Load per Pile kips (kN)					
Hurricane Katrina Base Case	No Scour	958 (1,299)	54 (240)	100 (445)	242 (1,076)	Yes	Yes	Yes	No
	4.3 (1.3)	1,083 (1,469)	63 (280)	100 (445)	227 (1,010)	Yes	Yes	No	No
Hurricane Katrina Shifted	No Scour	708 (960)	40 (178)	90 (400)	242 (1,076)	Yes	Yes	Yes	No
	5.0 (1.5)	808 (1,096)	49 (218)	90 (400)	222 (988)	Yes	Yes	Yes	No
Hurricane Katrina Shifted + Intensified + SLR	No Scour	333 (452)	21 (93)	90 (400)	242 (1,076)	Yes	Yes	Yes	No
	5.0 (1.5)	417 (565)	27 (120)	90 (400)	222 (988)	Yes	Yes	Yes	No

305 Total pile tensile resistance is compared to maximum tension load per pile to determine if the soil can resist uplift forces acting on the pile without failure.
306 The concrete piles have a calculated structural tensile capacity of 386 kips (1,717 kilonewtons) in accordance to the *LRFD Bridge Design Specifications* document. This value is compared to the maximum tension load per pile to determine if the pile will break due to the tensile load.
307 The concrete piles have a calculated structural shear capacity of 55 kips (245 kilonewtons) in accordance to the *2012 AASHTO LRFD Bridge Design Specifications* document. This value is compared to the maximum shear load per pile to determine if the pile will break due to the shear load.
308 The concrete piles have a calculated structural bending moment capacity of 233 feet-kips (316 kilonewton-meters) in accordance to the *2012 AASHTO LRFD Bridge Design Specifications* document. This value is compared to the maximum bending moment load per pile to determine if the pile will break due to the shear load.

The results indicate the piles have sufficient axial capacity to resist the uplift force on the superstructure (up to the point the anchor bolts on the superstructure fail) under all three surge scenarios, but the piles are not able to resist the lateral forces (shear and moment) under any of the scenarios and may fail due to shear and / or bending. However, during Hurricane Katrina, the storm surge only caused superstructure damage and did not damage the piles. The discrepancy between the modeled results and what actually occurred can likely be attributed to the lack of detailed geotechnical data, such as the shear strength parameters and physical properties (e.g., plasticity characteristics, unit weight) of soils, in the vicinity. A full analysis, beyond the scope of this study, should involve development of the shear strength parameters and the physical properties of the soils based on soil borings with SPT and / or cone penetration test soundings along with geotechnical laboratory tests on collected soils samples during investigation.

Step 6 – Identify Adaptation Option(s)

Under all three scenarios, Step 5 indicated that adaptation options are required for dealing with the potential of deck uplift and float away (Failure Mode One) and also for dealing with the excessive lateral forces on the piles (Failure Mode Two), actual performance during Hurricane Katrina notwithstanding. No separate adaptation options are required under any of the scenarios to deal with scour (Failure Mode Three).

Viable adaptive design options that would attempt to prevent Failure Modes One or Two from occurring include:

- One of the adaptation options recommended in *Guide Specification for Bridges Vulnerable to Coastal Storms* is to design the superstructure to break away from the substructure in a significant storm surge. While, the designers of the case study bridge did not intend the superstructure to breakaway in a significant storm surge, this is what occurred in Katrina. A breakaway superstructure would allow for a much shorter and less expensive rebuild period than if the substructure was allowed to be damaged. In essence, the facility is designed for a controlled failure if surge and wave forces become too great. This is an important adaptation concept that might be worth considering for similar types of bridges in comparable environments.
- Design the anchor bolts and horizontal through girder bolts to fail at a lower load level resulting in lower loads transmitted to the piles in a significant storm surge. Although the bolts would fail at a lower level and alleviate some of the shear and bending stresses on the piles, more spans could breakaway when compared to the current bolt configuration.
- Improve the connection between the superstructure and substructure and also investigate if the substructure foundations require strengthening after obtaining all geotechnical information
- Investigate the use or partial use of installing open grid decks to possibly reduce the vertical loads imposed by a storm surge. Note that this would also help prevent damage from uplift and may be able to be used in lieu of the severability approach whereby the deck is designed to float off if uplift forces are too strong. Open grid decks are lighter and would reduce the vertical forces imposed on the superstructure, but may have corrosion problems. Open grid

decks also sometimes develop cracks (fatigue and stress) and have much higher maintenance needs than concrete decks.

- In order to lower the magnitude of the lateral wave forces, one might consider replacement of the girders with others having a shallower section and/or replace the parapets with a more open railing system. A shallower girder can be installed by adding more girders or replacing with a slab type system, but this might prove costly. Replacing a parapet with a more open railing system may not follow standards that are typically used for such ramps. Benefits and disadvantages would need to be evaluated.
- Redesign the superstructure to have removable deck sections for portions of the bridge which are closer to water level. Sufficient advance notice of an upcoming storm surge is needed for this repair option to be successful especially given the time needed to remove the sections and possible high winds associated with an expected storm. The girders can also be removable but would add to the length of advance notice time needed to have them removed.
- Replace the bridge with a raised/protected embankment section up to a point high enough along the ramp where a bridge section can be used.
- Investigate eliminating the entire ramp and interchange after the next major storm causes severe damage. Discussions with stakeholders would be needed before this decision could be made.

Step 7 – Assess Performance of the Adaptation Option(s)

In this step, the performance of each adaptive design option mentioned in Step 6 would be determined under each of the storm surge scenarios. This would aid in the development of effective adaptation solutions and serve as a basis for a benefit-cost analysis. This step was not completed for this particular case study due to resource limitations.

Step 8 – Conduct an Economic Analysis

An economic analysis was not included in this case study but is recommended for facility-level adaptation assessments. See Section 4.4.1 for an example of how an economic analysis was applied to a culvert exposed to changes in precipitation due to climate change.

Step 9 – Evaluate Additional Decision-Making Considerations

Additional factors that will influence decision-making on what adaptation option to consider include:

- How much a significant storm surge or future sea level rise would affect the land uses served by the ramp. It is conceivable that the next significant storm surge could significantly damage the superstructure and substructure of the ramp and may also eliminate the land uses served by the ramp. If those land uses are not re-built, the need for the ramp may be lessened to the point that it is no longer needed and expensive reconstructions or adaptations are not necessary. Elimination of the interchange may also bring up the planning question of whether or not to increase the capacity of I-10. An important concept in transportation adaptation planning is that transportation facilities should not be adapted beyond the viability of the land uses they serve. Public meetings with nearby property owners would need to be held to discuss this issue.

- The public acceptance of an open-bridge deck or more open parapet walls and possible increased safety hazards from both adaptation options.

Step 10 – Select a Course of Action

The course of action recommended for this case study is the eventual elimination of all or a portion of the Exit 30 interchange the next time a storm causes major damage to the facility. The near-term course of action should be a detailed study exploring the implications of this option. The study should include an economic analysis of maintaining the interchange versus closing it down. The issues of how the closure would affect volume of traffic on I-10 (dependent on how much of the interchange is to be demolished) should be made a part of the study.

Step 11 – Plan and Conduct Ongoing Activities

Specific ongoing activities will depend upon the adaptation option selected. Generally, ongoing adaptation options for a coastal bridge would include monitoring the performance of the adaptive actions during future storm events and documentation of instances when the adaptation might have saved money from what would have happened had the current design remained in place.

Conclusions

This case study has, using the *General Process for Transportation Facility Adaptation Assessments*, demonstrated how a bridge can be analyzed for potential storm surge scenarios, including those where sea level rise has been factored in. Both the deck and the substructure of the portion of the ramp being studied were found to be vulnerable to each of the three surge scenarios tested. To address these vulnerabilities, several adaptation options were presented for consideration. Also, the overall area was considered and the adaptive option of eliminating the interchange was presented.

The two important lessons learned in this case study are that (1) the worst case storm surge scenario does not necessarily translate to the worst effect on the facility and that (2) one should examine the overall viability of a facility as part of an adaptation assessment and consider elimination / retreat as an adaptive option in some situations.

4.4.6 Road Alignment Exposure to Storm Surge – I-10 (Mileposts 24 to 25)

Introduction

Stronger storms and higher sea levels increase the risk of water surging over the paved surface of coastal highways. Known as overtopping, such an event due to storm surge can cause a variety of impacts to road and rail alignments located along coastlines. This chapter illustrates how the *General Process for Transportation Facility Adaptation Assessments* can be used to analyze the potential risks of this happening on I-10 on the south side of Mobile (between mileposts 24 and 25). The assessment determines the potential for roadway overtopping, the risk of roadway embankment failure, the degree of inland flooding caused by potential storm surge scenarios, and the implications of flow velocities through bridge underpasses.

It was found that all of the storm surge scenarios tested present some threat of inundation and erosion to the roads and railroad passing through the underpasses in the study corridor. Two of the three surge scenarios tested overtop I-10 and one of them causes enough erosion to fully breach the highway embankment. An adaptation option is recommended that armors the I-10 embankment at its low spot and hardens the road and rail infrastructure passing through the underpasses.

Case Study Highlights

Purpose: To evaluate the potential for roadway overtopping, embankment failure, and inland flooding, and determine the implications of flow velocities through bridge underpasses due to storm surge.

Approach: The potential for overtopping was evaluated by overlaying the projected surge flood elevations onto the roadway profile and cross-sections of the underpasses. Failure (breaching) of the roadway was evaluated by modeling the road as a barrier across the flow, then calculating erosion potential and erosion rates based on estimated flow rates. The potential and degree of inland flooding of the nearby Oakdale neighborhood was determined using a time-step analysis and the storm surge hydrographs. Then, the maximum flow velocities through bridge underpasses were estimated using a FHWA tidal hydraulics orifice approach combined with the time-step analysis of the storm surges.

Findings: The segment analyzed have the potential to overtop and be vulnerable to erosion under certain storm scenarios.

Viable Adaptation Options:

- Harden underpasses
- Armor roadway embankment
- Raise the roadway

Other Conclusions: There has been limited research on methods for predicting roadway breaches.

Application of the General Process for Transportation Facility Adaptation Assessments

Step 1 – Describe the Site Context

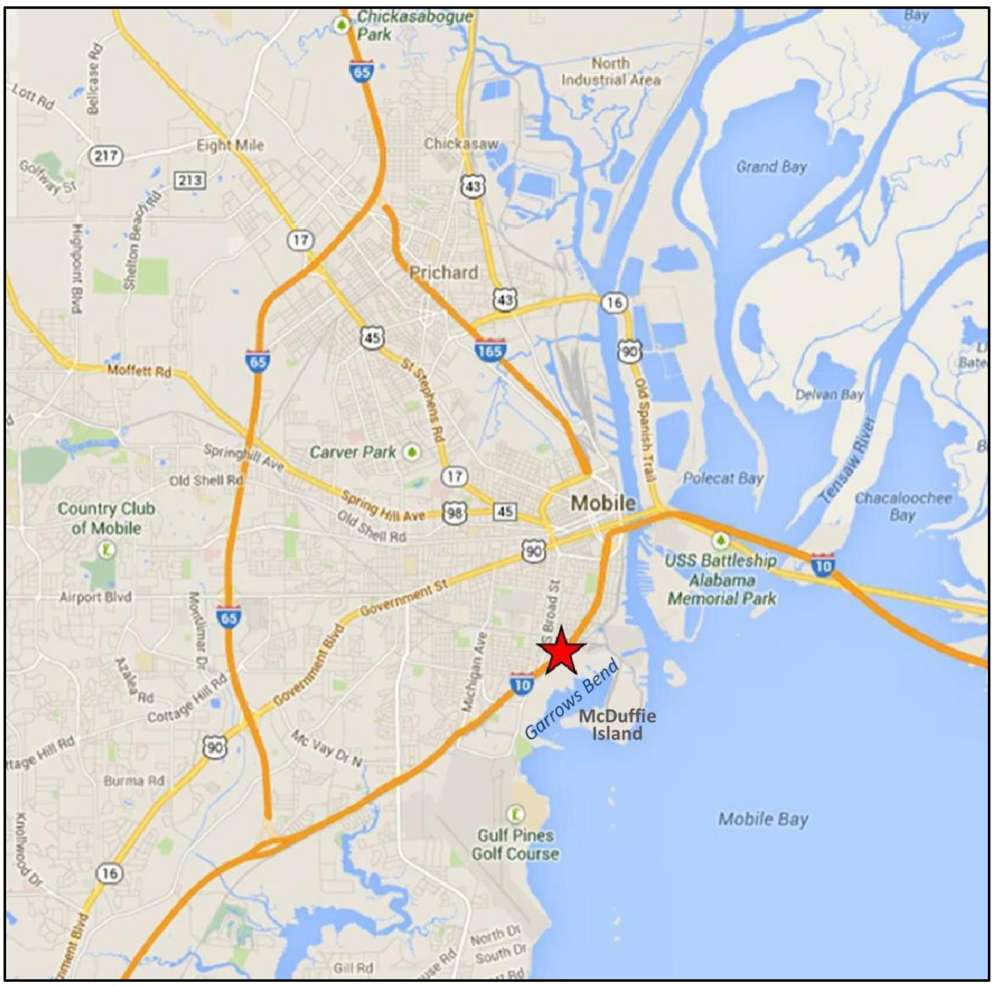
I-10 is a cross-country highway that runs through eight states connecting Los Angeles, CA to Jacksonville, FL. In Alabama, I-10 traverses Mobile and Baldwin Counties and is a critical route for traffic crossing Mobile Bay and accessing downtown Mobile from the south and west. The study segment is located approximately one mile south of downtown Mobile (see Figure 73). In this segment, I-10 is a 10-lane freeway running parallel to Garrows Bend, an estuary within Mobile Bay that bounds the west side of McDuffie Island. Garrows Bend is well connected to Mobile Bay and is thus expected to experience storm surge flooding equivalent to that experienced in Mobile Bay. I-10 is offset approximately 1,000 to 2,000 feet (305 to 610 meters) from the shoreline of Garrows Bend. Surrounding land uses include the residential neighborhood of Oakdale to the north and west of the road and industrial facilities located to the east, between the roadway and the shoreline. I-10 is owned and maintained by ALDOT.

Step 2 – Describe the Existing Facility

The study segment lies between mileposts 24 and 25, between the Broad Street and Warren Street bridges (see Figure 74). A third bridge crossing lies on the northern end of the study segment where I-10 crosses Tennessee Street and a railroad line. A stream is piped underneath the roadway embankment near this crossing.

The study segment is approximately 4,300 feet (1,311 meters) in length and 170 feet (52 meters) wide with 10 travel lanes and four shoulders. Each travel lane is 11 to 12 feet (3.4 to 3.7 meters) wide and each shoulder is 12 to 14 feet (3.7 to 4.3 meters) wide.

Figure 73: Location of the I-10 Study Segment in Relation to Mobile Bay and the Mobile Metropolitan Area[309]



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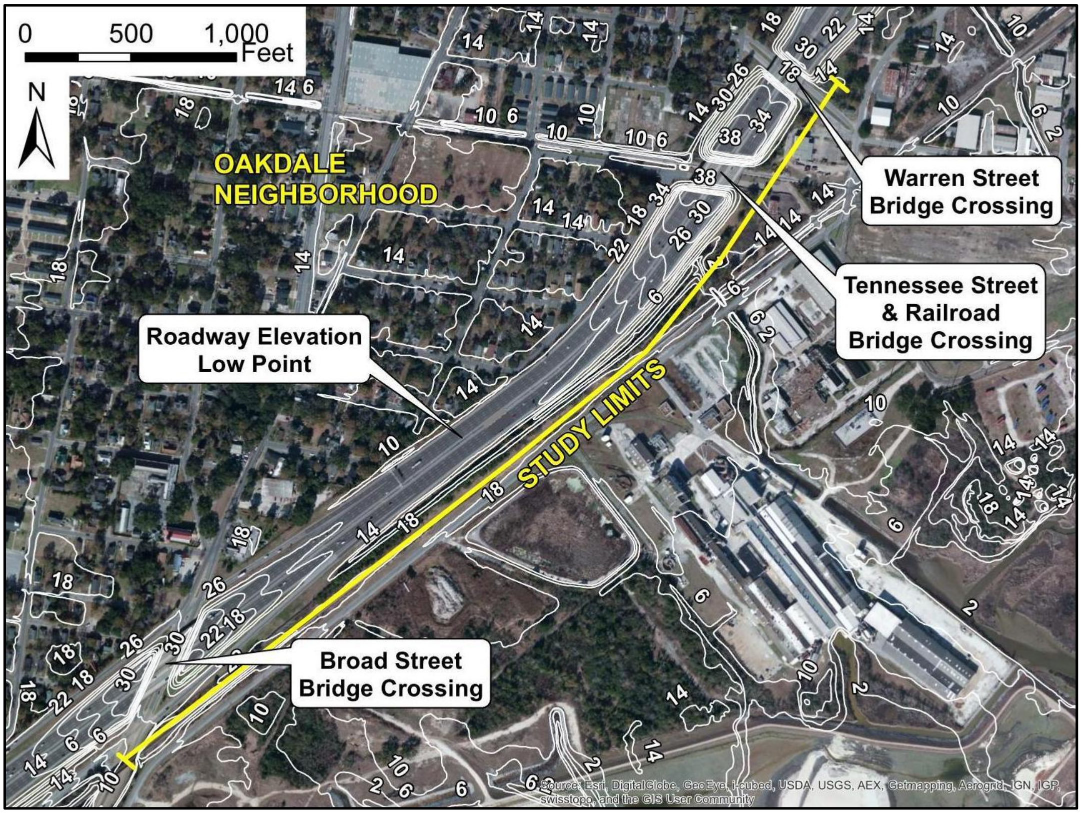
309 Source of base map: Google Maps (as modified)

The roadway profile has a low point at the center of the study segment, midway between the Tennessee Street and Broad Street crossings. This low point has an elevation of 14.6 feet (4.5 meters),[310] whereas the bridges at Broad Street and Tennessee Street are at the highest points along the segment with elevations of 34 feet (10.4 meters) and 40 feet (12.2 meters), respectively. Table 39 summarizes the bridge and underpass dimensions at the three crossings. All dimensions described are measured using two-foot (0.6 meter) contour data maintained by the City of Mobile and GIS analysis of aerial imagery; an adaptation analysis for an actual project would require site surveys to gather more accurate information.

[310] All elevations reported in this case study are in relation to the North American Vertical Datum 1988 (NAVD88).

Figure 75 shows ground level views of the roadway and the three underpasses.

Figure 74: Plan View of I-10 Showing Study Limits, Bridge Crossings, and Elevation Contours[311]



311 The contour elevations shown in this map do not include the elevations on the bridge overpasses.

Table 39: Dimensions of the Bridge Underpasses along the I-10 Study Segment[312]

	Broad Street Crossing Feet (Meters)	Tennessee Street and Railroad Crossing Feet (Meters)	Warren Street Crossing Feet (Meters)
I-10 Bridge Top Elevation	34.0 (10.4)	40.0 (12.2)	30.0 (9.1)
Road / Railroad Elevation Under the Bridge	12.6 (3.8)	11.6 (3.5)	13.1 (4.0)
Road / Railroad Width Under the Bridge	78 (23.8)	94 (28.7)	92 (28.0)
Estimated Vertical Clearance Above Road / Railroad[313]	16.4 (5.0)	23.4 (7.1)	11.9 (3.6)

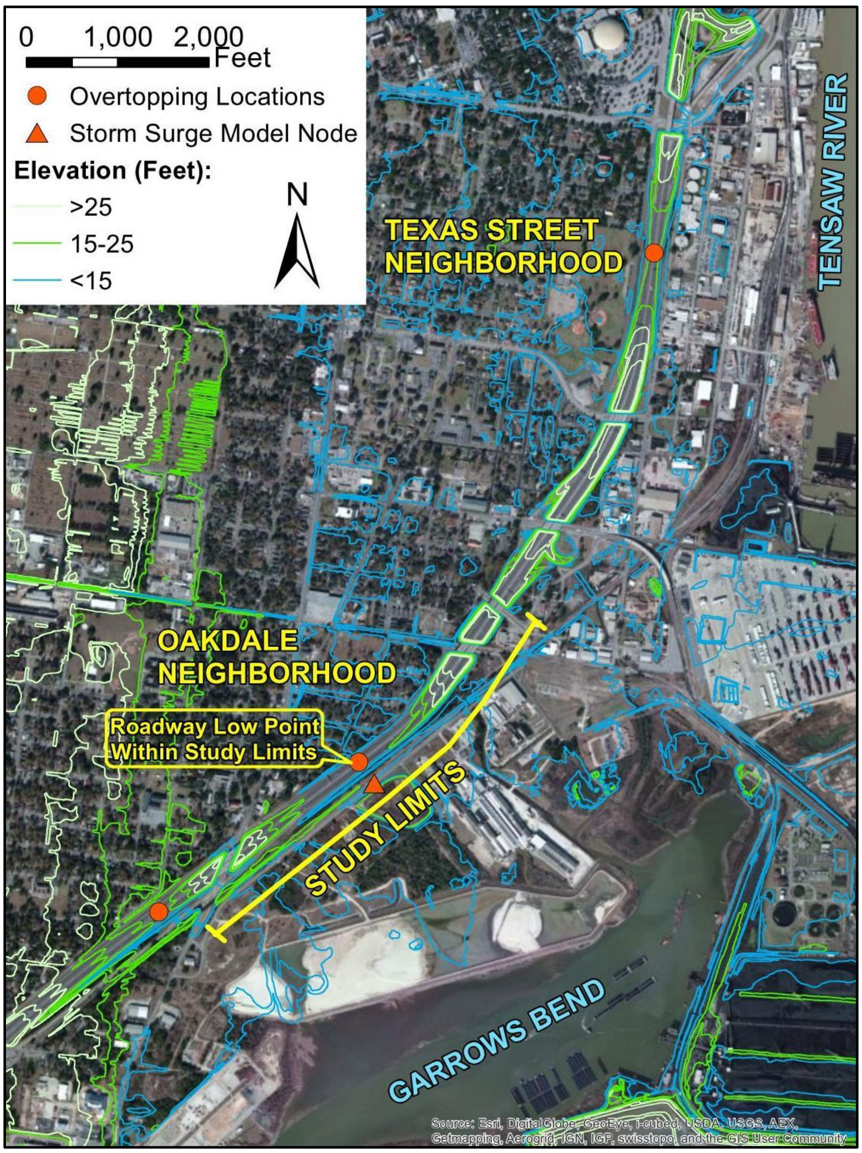
[312] Note: All elevations in this study are in relation to the North American Vertical Datum of 1988 (NAVD88).
[313] Assumes a five foot (1.5 meter) depth between the top of deck and the bottom of girders.

Figure 75: Study Area Images
(Top to Bottom: I-10 Roadway Looking East from North of the Embankment Near the Low Point, Broad St. Underpass, Tennessee St. and Railroad Underpass, and Warren St. Underpass Looking North)[314]

[314] Source: Google Maps Street View

The study limits for this assessment were chosen to analyze the effects of storm surge overtopping at a well-defined low point in the roadway and storm surge flooding impacts through the bridge underpass pathways. The low-lying areas that are vulnerable to flooding continue northward as shown in Figure 76 and it is expected that similar storm surge flooding dynamics will occur along the length of I-10 in these areas. However, to provide an example assessment, this case study was limited to the one roadway low point near the Oakdale neighborhood and nearby crossings; a flooding analysis for that neighborhood was also undertaken. In a typical analysis of storm surge impacts on road and rail alignments, one might instead set the assessment limits based on the entire affected segment.

Figure 76: Topography in the Vicinity of the I-10 Study Segment



Step 3 – Identify Climate Stressors That May Impact Infrastructure Components

For this roadway segment, storm surge and the resulting flooding are the primary climate change-related environmental factors likely to affect the roadway. In addition, this assessment also considered sea level rise as an added factor in the most extreme storm surge scenario analyzed.

Additional related roadway components not considered in this assessment that might be impacted by changing climatic conditions include the roadway pavement drainage system, culvert / storm sewer system, embankment slope protection, and roadway sub-base. Roadway pavement drainage could be impacted by increased precipitation intensity where the flat slope of a roadway may not meet the design spread conditions for a given storm event because of its location. The culvert / storm sewer system could also be impacted by changing tailwater[315] conditions caused by sea level rise. Increased tailwater will decrease the ability of the system to handle water flows. Embankment slope protection for the roadway could be at risk if sea levels or riverine floodplain conditions were to rise and expose the approach roadway to erosive flow conditions. Lastly, the roadway sub-base could become compromised if sea level rise were to result in permanent inundation of the roadway sub-base. Inundation of the sub-base would result in a loss of bearing strength and potentially cause hydraulic forcing of voids in the base itself that were originally caused from repeated loading / unloading of the roadway from normal traffic.

Step 4 – Decide on Climate Scenarios and Determine the Magnitude of Changes

The following three storm surge scenarios were considered for this adaptation assessment:

- **Hurricane Katrina Base Case Scenario**: This scenario represents the surge conditions that actually occurred in Mobile with Hurricane Katrina making landfall at the Louisiana-Mississippi border. The effects of Hurricane Katrina on the Mobile area were not as severe as they were at the Louisiana-Mississippi border.
- **Hurricane Katrina Shifted Scenario**: This scenario estimates the surge levels that would occur if Hurricane Katrina's path was shifted east to make landfall in Mobile.
- **Hurricane Katrina Shifted + Intensified + Sea Level Rise (SLR) Scenario**: This scenario estimates the surge levels that would occur if Hurricane Katrina was shifted, intensified with stronger winds due to climate change, and came on top of 2.5 feet (0.8 meters) of sea level rise.

A more detailed description of each scenario and how it was developed can be found in Section 4.4.4 of this document (under Step 4) and in the *Climate Variability and Change in Mobile, Alabama* report.[316]

For this case study, site-specific flood elevations under each scenario were calculated for a point located along the I-10 roadway study segment, approximately 290 feet (88 meters) southeast of

[315] Tailwater refers to the water at a system (e.g., culvert) outlet.
[316] USDOT, 2012

the roadway low point. This point, shown in Figure 76 as the storm surge model node, is the node for which flooding elevations were calculated as part of the ADvanced CIRCulation (ADCIRC) storm surge modeling and was chosen to be representative of flood elevations for the study site. The three storm surge elevations for each surge scenario (calculated at the storm surge model node location) and the current FEMA 100-year flood elevation for the study site location are summarized in Table 40.

Table 40: Flood Elevations for the Storm Surge Scenarios and the Current FEMA 100-Year Flood at the Storm Surge Model Node near the I-10 Study Segment[317]

	FEMA 100-year Flood Feet (Meters)[318]	Hurricane Katrina Base Case Scenario Feet (Meters)	Hurricane Katrina Shifted Scenario Feet (Meters)	Hurricane Katrina Shifted + Intensified + SLR Scenario Feet (Meters)
Storm Surge / Flood Elevation	11.8 (3.6)	13.0 (4.0)	20.3 (6.2)	25.0 (7.6)

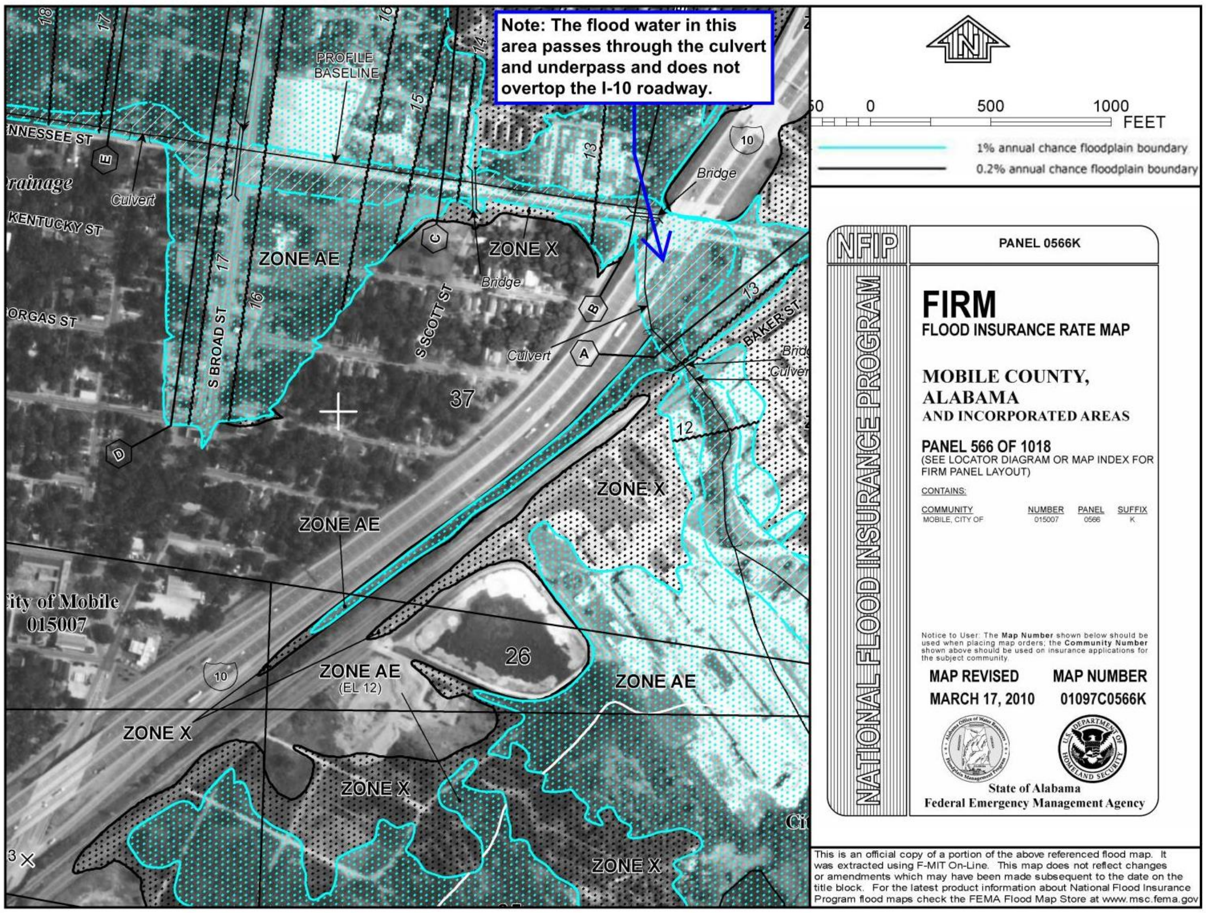
According to the FEMA flood insurance study of Mobile County, Alabama, the 100-year flood elevation due to coastal flooding in the vicinity of the study site is 11.8 feet (3.6 meters).[319] Comparing this to the predicted storm surge elevations, all three storm surge scenarios produce floods with return periods in excess of the current 100-year event. Figure 77 shows the FEMA 100-year flood boundary based on the flood insurance study. Note that the extent of flooding shown in the FEMA flood map includes both tidal and riverine flooding sources. Potential riverine flooding of the stream channel along Tennessee Street causes the flood elevations along this channel to be higher moving further inland. As evident in Figure 77, the flood elevation along Tennessee Street near its intersection with South Broad Street is almost 17 feet (5.2 meters); a value much higher than the coastal flooding elevation of 11.8 feet (3.6 meters). This suggests that the potential increased impacts due to riverine flooding, in addition to coastal flooding, may need to be considered in areas near rivers or streams. Such an effort was beyond the scope of this case study but should be considered if actual adaptation work were to be done in the area.

Analysis of historical flooding events provides background insight into the local impacts resulting from past climate events. During Hurricane Katrina, flood elevations of up to 11.5 feet (3.5 meters) were reported at the Mobile State Docks, located approximately three miles (4.8 kilometers) north of the study site.[320] This actual flood elevation is less than the storm surge flood elevation of 13 feet (4 meters) predicted in the Hurricane Katrina Base Case Scenario

[317] Note: The storm surge elevations shown here include wave run-up effects.
[318] See the Mobile County Flood Insurance Study (FEMA, 2010d)
[319] The FEMA 100-year flood elevation of 11.8 feet was obtained from the Mobile County Flood Insurance Study (FEMA, 2010d) documentation of stillwater elevations based on coastal transects. This study site is located along the coastline between the Hannon Road and US 90.
[320] NOAA, 2013d

model. However, information regarding actual flood elevations at the study site during Hurricane Katrina is limited. Flood elevations at the study site may differ from the State Docks value due to its location relative to Mobile Bay and wave run-up effects. The FEMA flood insurance study also cites a number of significant historical storms that have caused severe flooding in Mobile. Of note are Hurricane Camille of 1969 that produced a recorded peak tide of 7.4 feet (2.3 meters) in Mobile, and a 1926 hurricane that produced tides of 10.9 feet (3.3 meters) above normal tide in Mobile.[321]

Figure 77: FEMA Flood Insurance Rate Map in the Vicinity of the I-10 Study Segment [322]



Step 5 – Assess Performance of the Existing Facility

Four potential impacts of storm surge flooding were evaluated to assess the performance of the existing roadway under each of the potential storm surge scenarios. These include:

- Storm surge overtopping of the I-10 roadway and underpasses

[321] FEMA, 2010d
[322] Source: FEMA, 2010e (as modified). Note: The elevations shown are in NAVD88.

- I-10 roadway embankment breaching due to overtopping flows
- Inland flooding impacts to the Oakdale neighborhood in terms of total flood volume entering the neighborhood and time to drain during the ebb of the surge
- Implications of flow velocities through the underpasses at the three bridge crossings.

Each of these impacts is discussed in the sub-sections that follow.

Storm Surge Overtopping of I-10 Roadway and Underpasses

The degree of overtopping of the I-10 roadway and underpasses was evaluated by overlaying the predicted storm surge flood elevations onto the roadway profile and cross-sections of the underpasses (see Figure 78). The profile and cross-section elevations were produced using AutoCAD Civil 3D with two foot (0.6 meter) planimetric contour data maintained by the City of Mobile. The brown lines in the figure represent the highest ridge along the roadway and the underpasses, while the blue dashed lines represent the highest flood level that is reached in each of the three storm surge scenarios.

As shown in Figure 78, only the Tennessee Street and Railroad Underpass location is expected to experience flooding under the Hurricane Katrina Base Case Scenario. Although the Broad Street underpass is also located at a slightly lower elevation than the Hurricane Katrina Base Case Scenario flood elevation, there are no overland flow paths into this underpass from the seaside direction for flood elevations lower than approximately 14 feet (4.3 meters), and thus flooding is not expected on this segment of Broad Street for this scenario.

Figure 78 also shows that the Hurricane Katrina Shifted Scenario and the Hurricane Katrina Shifted + Intensified + SLR Scenario will cause overtopping of the I-10 roadway and flooding of all three underpasses. Based on the flood elevations, the sequence of flooding under both of these scenarios proceeds as follows: first the Tennessee Street and Railroad Underpass is flooded, followed by the Warren Street underpass, and then the Broad Street underpass and I-10 roadway at around the same time.

It is important to evaluate whether the existing roadway is currently meeting design standards for roadway flooding. Although ALDOT does not indicate a specific return period storm for design of roadways against flooding, the Code of Federal Regulations states that the design flood for through lanes of interstate highways is the 50-year flood.[323] The FEMA 50-year flood elevation in the vicinity of the study site is 10.6 feet (3.2 meters).[324] As discussed earlier, the FEMA 100-year flood elevation is 11.8 feet (3.6 meters), which does not overtop the I-10 roadway study segment as shown in Figure 77. Therefore, the I-10 study segment is currently meeting the federal design requirement as well as accommodating the 100-year storm event.

[323] USDOT, 1994.
[324] FEMA, 2010d

Figure 78: Elevation Views of the I-10 Study Segment Roadway Profile and Underpass Cross-Sections

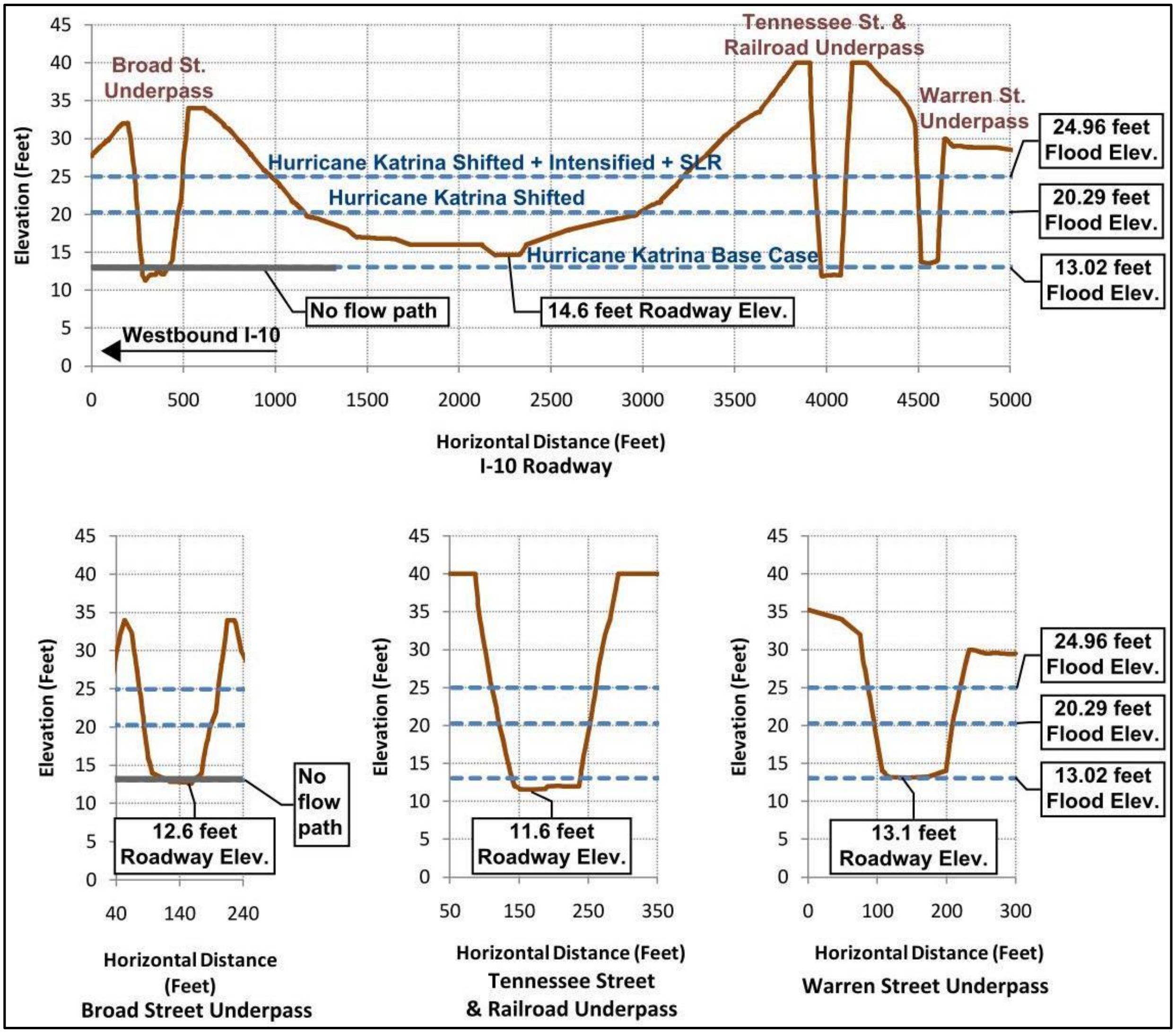


Figure 79, Figure 80, and Figure 81 show potential inundation zones under each of the storm surge scenarios. Comparing these figures to the FEMA 100-year flood map (Figure 77), it appears that the flooding extent predicted for the Hurricane Katrina Base Case Scenario storm surge is comparable to that of the 100-year storm, whereas the Hurricane Katrina Shifted Scenario and the Hurricane Katrina Shifted + Intensified + SLR Scenario have much larger impact areas.

Figure 79: Hurricane Katrina Base Case Scenario Flood Zone in the Vicinity of the I-10 Study Segment, Flood Elevation 13 Feet (Four Meters)

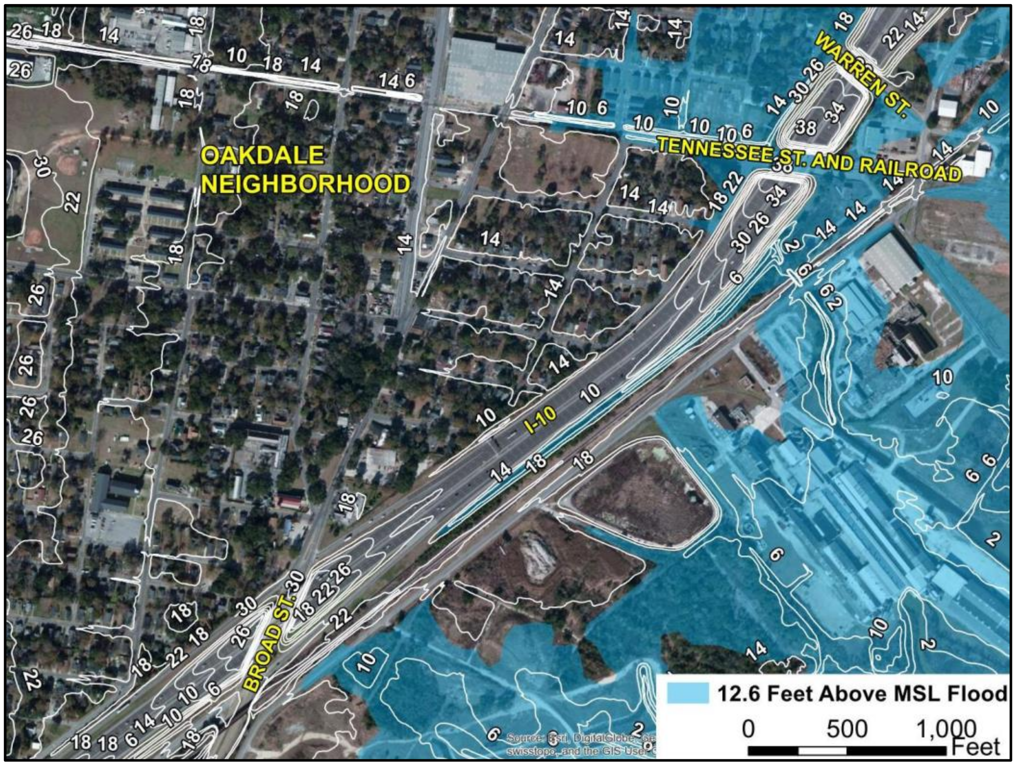


Figure 80: Hurricane Katrina Shifted Scenario Flood Zone in the Vicinity of the I-10 Study Segment, Flood Elevation 20.3 Feet (6.2 Meters)

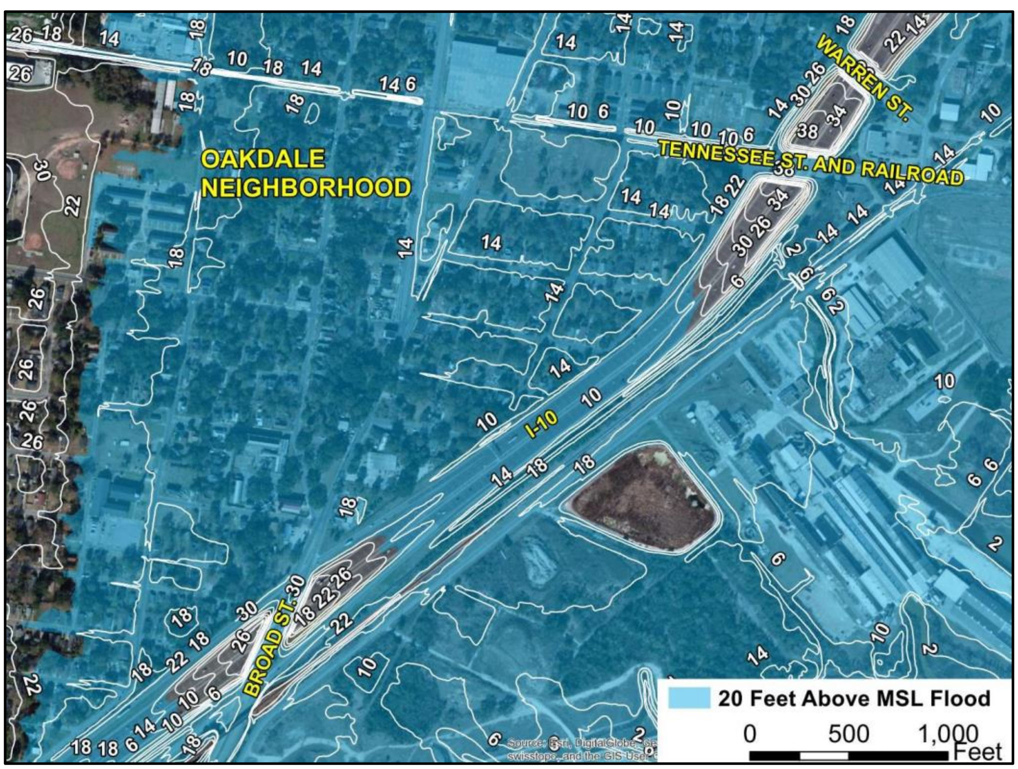
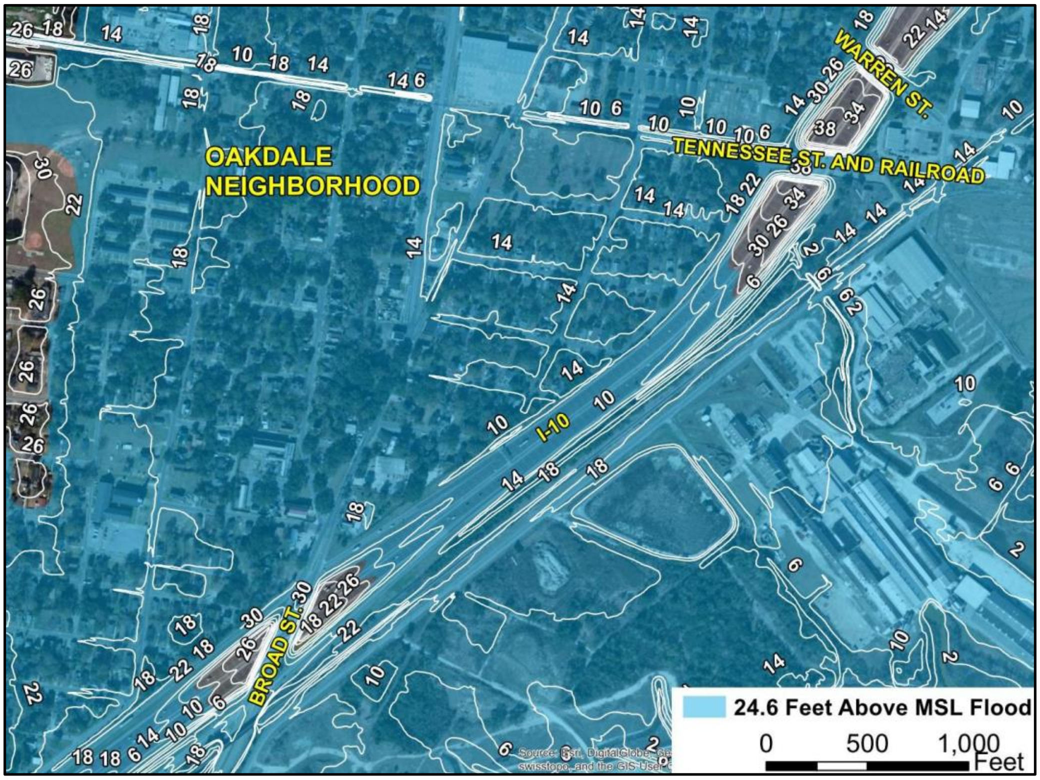


Figure 81: Hurricane Katrina Shifted + Intensified+ SLR Flood Zone in the Vicinity of the I-10 Study Segment, Flood Elevation 25 Feet (7.6 Meters)



<u>I-10 Roadway Embankment Breaching</u>

The most common type of embankment failure during flooding is erosion of the embankment due to overtopping.[325] Erosion will occur if erosive forces created by high flow velocities over the embankment exceed the ability (that is, strength) of the embankment to resist erosion. Erosion can begin at the break point between the downstream shoulder and earthen slope and progress toward the roadway and down the slope. Alternatively, erosion can also begin at the toe (base) of the downstream slope, typically under low tailwater[326] elevation conditions. For embankments consisting of non-cohesive materials such as sand, breaching typically occurs as progressive surface erosion.[327] For embankments composed of cohesive materials such as clay, the erosion typically begins as rills[328] which start a headcutting[329] process that causes erosion to progress toward the crest of the embankment.[330]

[325] Chen and Anderson, 1986
[326] Tailwater elevation in this case refers to the water surface elevation on the downstream side of the embankment. This can also be described as the flood elevation inland of the roadway during surge flow and the flood elevation seaside of the roadway during ebb flow.
[327] Progressive surface erosion is the gradual erosion of soil particles as they are washed away by moving water.
[328] Rills are narrow channels that form in the surface of the embankment as flows begin to erode the embankment soil.
[329] Headcutting is a channel erosion process where erosion downcutting of the channel bed starts at the low point of the channel and progresses upstream / up-hill.
[330] ASCE EWRI, 2011

As noted above, the Hurricane Katrina Base Case Scenario is not expected to overtop the I-10 roadway, so no embankment erosion is expected with this scenario. Thus, the embankment erosion analysis was only performed for the Hurricane Katrina Shifted Scenario and the Hurricane Katrina Shifted + Intensified + SLR Scenario.

A key prerequisite to modeling the amount of erosion and the potential for breaching of the roadway is understanding the flow rates over the embankment. Because flow rates vary over the duration of a storm, a time-step analysis was performed at half-hour increments for each of the storm surge flooding scenarios. First, storm surge hydrographs for each scenario were developed to determine the flood elevation at varying time steps (see Figure 82). Next, stage[331]-discharge curves were developed to capture varying flow area geometries and head differentials[332] across the roadway at varying flood elevations. The stage-discharge curves were developed by calculating the discharge at various flood elevations to create a relationship between flood elevation and flow rate.

To calculate the discharges over the roadway, the roadway was modeled as a broad-crested weir[333] using the following formula:

$$Q_f = C\,bH^{3/2}$$

Where,

Q_f = The flow rate over the roadway (cubic feet per second)

b = Width of the submerged portion of the roadway (feet)

H = Hydraulic depth[334] (feet)

C = Discharge coefficient that accounts for the effects of contraction, velocity, and fluid properties (3.1 [English units] is used in this study assuming a rounded upstream edge due to the gradually sloped embankment and shoulder lane)

Topographic data showed that the roadway has a relatively low embankment at the low point. The height of the lowest point of the roadway embankment is no greater than three to four feet (0.9 to 1.2 meters) higher than the lowest ground elevation in the neighborhood. It was found after a time-step analysis of the storm surges, that the tailwater elevation rises rapidly during the surge and the roadway reaches submerged weir conditions[335] quickly. From this point on, the

[331] Stage refers to water elevation.
[332] Head differential is the difference between the storm surge flood elevation and the inland flood elevation above the top of the roadway.
[333] A weir is a barrier across a river (smaller than a dam) characterized by the allowance of water to flow over the top of the barrier. The broad-crested weir formula is a simplified approach to modeling the roadway embankment which assumes a rectangular cross-sectional flow area. To ensure that correct flow areas are used, in this study the hydraulic depth (see next footnote) is used as "H" in the equation that follows.
[334] Hydraulic depth is equivalent to the flow area divided by the width of the submerged section of roadway ("b" in the equation). The weir equation typically uses the difference between the water elevation and the weir crest as "H" in the equation, but using the hydraulic depth ensures that the correct flow areas are accounted for.
[335] Weir hydraulics is based on a condition of rapidly changing flow with fast flow conditions over the weir. Submergence occurs when the speed of the flow is limited by high tailwater depths.

roadway was modeled as a submerged weir using the following formula to calculate discharges over the roadway:

$$Q_s = Q_f \left[1 - \left(\frac{H_2}{H_1} \right)^{1.5} \right]^{0.385}$$

Where,

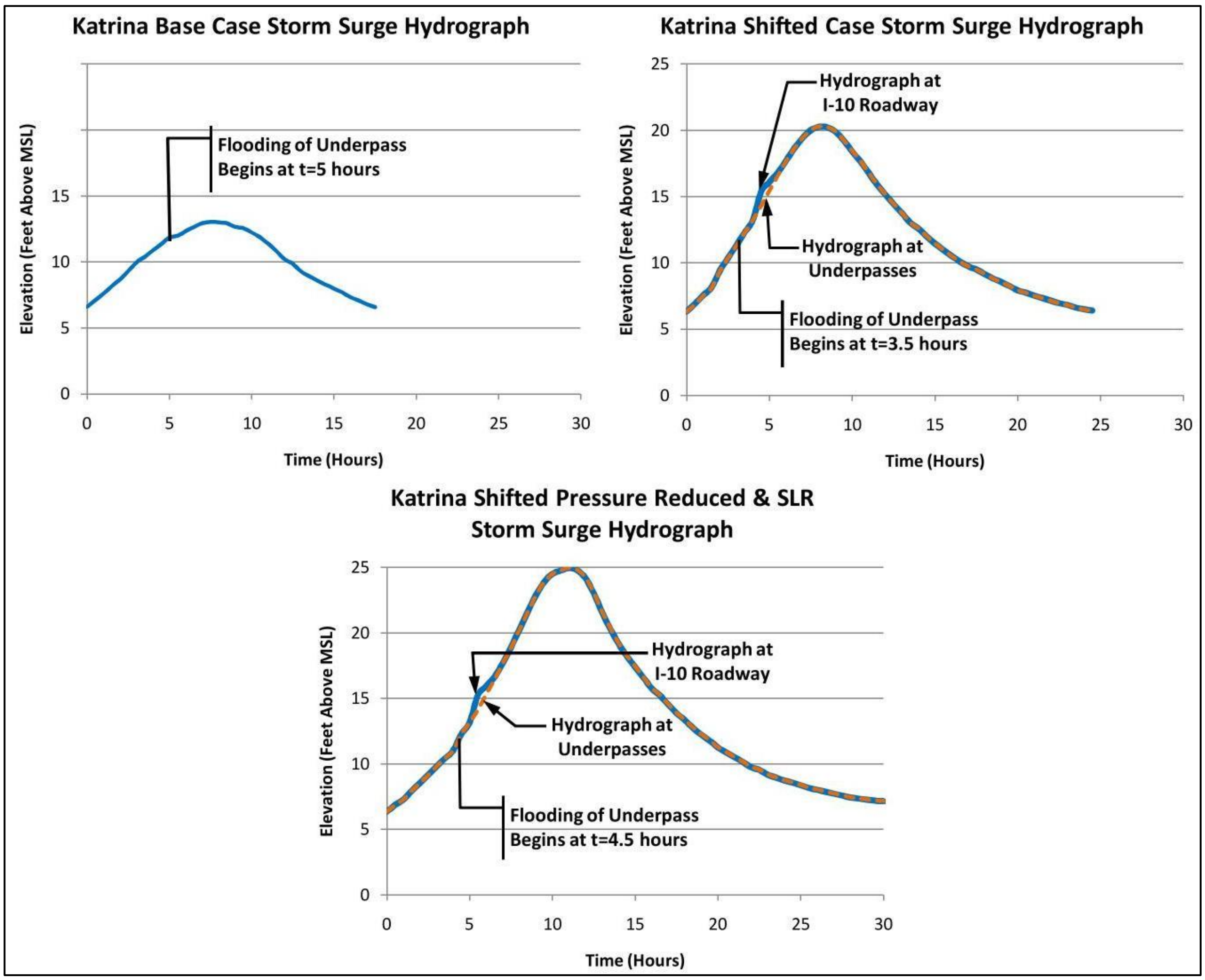
Q_s = Submerged weir flow rate (cubic feet per second)

Q_f = Unsubmerged weir flow rate (from previous formula, cubic feet per second)

H_1 = The upstream head elevation[336] above the crest (feet)

H_2 = The downstream head elevation above the crest (feet)

Figure 82: Storm Surge Hydrographs for the I-10 Study Segment



[336] Head elevation refers to the difference between the water surface elevation and the weir crest (top of weir) elevation.

Given the calculated flow rates over the embankment, the next step was to calculate erosion potential and erosion rate. Various models have been developed to predict the erosion potential of embankments. Many of these studies have aimed to correlate peak flow rates to hydraulic parameters for breached dams where the flow rate depends on the dimensions of the breached section. In this case, however, the flow rates are tied to storm surge flood elevations, which increase independently of the breach dimensions. As such, the embankment breaching analysis for this case study was conducted in two parts: first, a determination was made as to whether embankment erosion will begin based on permissible velocities and shear stresses[337] and, if so, an estimate of the erosion rate was made assuming breaching had begun.

The potential for erosion to begin on the embankment was evaluated based on the permissible shear stress of the materials comprising the embankment. For this study site, design plans showed that the embankment subgrade along this reach of I-10 was composed of loamy borrow material. Silt loam was therefore used for this analysis. However, if one were conducting an actual project, the soil type would need to be verified through field surveys for a more accurate estimation of erosion potential.

The threshold velocity and shear stress for erosion were determined through use of the critical velocity and critical shear stress of the soil, respectively. A critical velocity of two feet per second (0.6 meters per second) and a critical shear stress of 0.1 pounds per square foot (2.4 pascals), applicable to silt loam, were used in the analysis.[338] Aerial imagery shows that the embankment also contains vegetative growth, which appears to be short grass on both slopes,[339] but that bare patches of soil may also be present. Because erosion can initiate at weak points along the embankment slope, the values for soil were used instead of the higher permissible values provided by vegetation. If the shear stress caused by the flows overtopping the embankment exceeded the permissible shear stress, erosion would begin. The level of shear stress that determines if erosion will occur was calculated from the flow velocity according to the following formula:[340]

$$\tau = \frac{1}{8} f \rho V^2$$

Where,

τ = Shear stress (pounds per square foot)

f = The Darcy-Weisbach coefficient (0.02 for a smooth soil surface was used in this analysis)

ρ = Water density (pounds per cubic foot)

[337] Shear stress is the force per unit area acting parallel to the surface of an object, measured in pounds per square foot in this study.
[338] Chen and Anderson, 1986
[339] The critical shear stress of short grass is 0.6 pounds per square foot (28.7 pascals) according to Chen and Anderson, 1986.
[340] Chen and Anderson, 1986

V = Local velocity[341] (feet per second)

Once the initial breaching has begun, the erosion rate was estimated using two methods that were developed to determine the erosion of highway embankments based on field and laboratory test data. The first method is based solely on erodibility of the embankment soil and is calculated from shear stresses. This method does not account for vegetative cover. The erosion rate for non-cohesive soils using this method is given by the following formula:[342]

$$E = 0.00324(\tau - \tau_c)^{1.3}$$

Where,

E = Erosion rate (cubic feet per square foot per second)

τ = Shear stress (pounds per square foot, from previous formula)

τ_c = Critical shear stress of soil (0.1 pounds per square foot [2.4 pascals] applicable to silt loam was used in this analysis)

The second method is an empirical method considering soil type, vegetative cover, embankment height, overtopping depth, and head differential. [343] The erosion rates under both methods are calculated in terms of cubic feet of soil per second per square foot of embankment area, and then converted to inches per second of erosion in the direction progressing into the embankment.

For the I-10 roadway segment, it is expected that erosion may occur on either the inland side of the embankment due to storm surge flows or on the seaside of the embankment due to ebb flows as the surge retreats back to the ocean. Therefore, flow rates were evaluated over the entire storm surge hydrograph and the maximum discharges were selected for breach analysis. It is also possible that local velocities on the embankment slope may vary; however, for simplification, the velocities calculated from the peak discharges based on the above weir equations were used. The relevant parameters and outputs of the embankment breaching analysis are provided in Table 41.

Both overtopping storm surge scenarios have the potential to create sufficient flow velocities and shear stresses to cause erosion of the embankment. Based on the embankment breaching analysis, the maximum local shear stress produced by the Hurricane Katrina Shifted Scenario and the Hurricane Katrina Shifted + Intensified + SLR Scenario storm surge flows are 1.7 and 2.7 pounds per square foot (81.4 and 129.3 pascals), respectively, which both exceed the permissible shear stress of 0.1 pounds per square foot (2.4 pascals) applicable to silt loam.

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[341] The local velocity is defined as the flow velocity near the surface of the embankment, which may differ from the average velocity based on the discharges calculated from the weir equations. Chen and Anderson, 1986 provide velocity profiles of flow down an embankment. For simplicity, this study uses the velocity derived from the discharges calculated from the weir equations.

[342] Chen and Anderson, 1986

[343] See Chen and Anderson, 1986 for more details on this method. This method involves the use of a series of nomographs (graphical calculating devices) based on mathematical models to determine the erosion rate of an embankment.

Table 41: Summary of the I-10 Roadway Embankment Breaching Parameters

	Hurricane Katrina Base Case Scenario[344]	Hurricane Katrina Shifted Scenario	Hurricane Katrina Shifted + Intensified + SLR Scenario
Max. Discharge Over Roadway in cfs (m^3/s)	--	13,693 (388)	48,431 (1,371)
Velocity Based on Max. Discharge in ft/s (m/s)	--	3.3 (1.0)	4.2 (1.3)
Max. Local Shear Stress in lb/sq ft (Pa)	--	1.7 (81.4)	2.7 (129.3)
Estimated Max. Rate of Embankment Erosion Based on Soil Erodibility in Inch/s (cm/s)	--	0.1 (0.2)	0.1 (0.3)
Estimated Max. Rate of Embankment Erosion Based on Empirical Method in Inch/s (cm/s)	--	0.1 (0.13)	0.2 (0.5)

Also, as shown in the table, the two methods of estimating rate of embankment erosion produced comparable erosion rates. Taking the more extreme case of the two estimation methods, the Hurricane Katrina Shifted Scenario erodes the embankment at a rate of 0.1 inches per second (0.2 centimeters per second) and the Hurricane Katrina Shifted + Intensified + SLR Scenario erodes the embankment at a rate of 0.2 inches per second (0.5 centimeters per second).

It is important to note that the erosion rate values presented above are based on maximum discharges and headwater elevations over the embankment and are therefore maximum erosion rates, which are not likely sustained over the entire flood duration. To calculate the total erosion of the embankment during the entire storm surge, it is possible to sum the amount of erosion at various time steps over the flood duration based on the erosion rates calculated from the velocities and shear stresses at each time step. However, for simplification in this analysis, the maximum erosion rates listed above were multiplied by the amount of time during which 80% to 100% of the maximum velocity was sustained over the storm surge hydrograph according to the time-step analysis. These duration times were the same for both surge flows and ebb flows: 2.5 hours for the Hurricane Katrina Shifted Scenario and two hours for the Hurricane Katrina Shifted + Intensified + SLR Scenario. This approximation may underestimate the actual total erosion that could occur, but it provides an idea of the scale of possible erosion. If adaptation actions were to actually be contemplated at this facility, a calculation of erosion rates by time step should be conducted.

[344] The analysis was not performed for this scenario since the Hurricane Katrina Base Case Scenario surge is not expected to overtop I-10.

At these erosion rates and estimated duration times, it was estimated that the Hurricane Katrina Shifted Scenario storm surge would erode 60 feet (18.3 meters) into both the inland and seaside embankments, causing failure of the shoulder lane and four travel lanes on both sides of the roadway. The Hurricane Katrina Shifted + Intensified + SLR Scenario storm surge would erode 114 feet (34.7 meters) into both the inland and seaside embankments, which would result in breaching of the entire width of the roadway. Note that this analysis does not account for pavement interaction with flows (it is assumed that erosion of the embankment slopes will immediately undermine the pavement).

Flood Volume Entering Oakdale Neighborhood and Time to Drain

The degree of potential inland flooding was analyzed for the Oakdale neighborhood in order to determine flooding elevations and the time period during which the neighborhood would be flooded. These are important factors in predicting flood damage and emergency response planning. In order to determine the total volume of storm surge flows entering the Oakdale neighborhood, a time-step analysis was performed using the storm surge hydrographs (see Figure 82). The primary flow paths into the neighborhood included overtopping of the I-10 roadway and discharge through the three underpasses for Broad Street, the Tennessee Street and Railroad Underpass, and Warren Street. Discharges over the I-10 roadway were calculated based on the weir equations (see previous sub-section) and discharges through the underpasses were calculated using the Federal Highway Administration (FHWA) tidal hydraulics orifice approach.[345] The base equation is similar to the orifice equation; however, the discharge coefficient includes entrance, exit, and friction losses. The formulas for flow rate through the underpasses and for the discharge coefficient are as follows:

$$Q = C_d A\sqrt{2g\Delta H}$$

$$C_d = \left[1 \div \left(K_o + K_b + \frac{2gn^2 L_c}{K^2 h_c^{\,4/3}}\right)\right]^{1/2}$$

Where,

Q = Flow rate (cubic feet per second)

C_d = Discharge coefficient

A = Flow area

ΔH = Head differential[346]

K_o = Velocity head loss coefficient on the ocean side[347]

[345] The tidal hydraulics orifice approach was developed by FHWA for the hydraulic evaluation of bridges in tidal waterways (see FHWA, 2004). This flow rate approximation is typically applicable to waterways such as inlets connecting the ocean to a bay.
[346] In this formula, the head differential is taken as the difference between the storm surge flood elevation and the inland flood elevation above the bottom of the underpass. The head differential varies over the duration of the storm surge.

K_b = Velocity head loss coefficient on the inland side[348]

g = Gravitational constant (32.2 feet per second squared [9.8 meters per second squared)

n = Manning's roughness coefficient of the underpass (0.02 used for concrete)

L_c = Length of underpass

h_c = Depth of flow in underpass

To incorporate the discharge formulas into the time-step analysis, stage-discharge curves were developed to capture varying flow area geometries and head differentials across the underpasses at varying flood elevations. A stage-volume curve was also developed for the Oakdale neighborhood to determine the inland flood elevation at various volumes of flows entering the neighborhood, which was also incorporated into the time-step analysis. Table 42, Table 43, and Table 44 provide a summary of the time-step analysis of storm surge flows. The tables summarize seaward and inland flood elevations, flow rates and velocities over the I-10 roadway embankment and all three underpasses, and the erosion rate of the I-10 embankment based on the flow velocities. The erosion rates documented in these tables are calculated using only the first estimation method presented above, based on erodibility of embankment soil and the erosion rate equation. The positive flow rates and velocities indicate flows approaching inland, and the negative flow rates and velocities indicate flows returning seaward. Table 45 summarizes the total volume of storm surge flows entering the Oakdale neighborhood, the time to reach the peak flood elevation in the neighborhood, and the time for the flows to drain from the neighborhood under the three storm surge scenarios.

The results of the time-step storm surge flooding analysis allow for several conclusions to be made. First, the flow rates through the underpasses and over the I-10 roadway were significant enough for the inland flood elevations to grow at rates almost equal to that of the storm surge flood elevations. In other words, the rate of surge rise is slow enough where the underpasses can handle all the flow with little attenuation, and the limited inland capacity to hold the flood volume allows inland flood elevations to rise quickly. As a result, the time to reach peak flood elevation in the inland neighborhood was close to the time to the storm surge peak, as denoted by the crest in the storm surge hydrographs (see Figure 82). As such, the I-10 roadway embankment does not provide significant attenuation of storm surge flows as flooding enters the Oakdale neighborhood.

[347] Velocity head loss coefficient on the ocean side is taken as one if the velocity goes to zero.
[348] Velocity head loss coefficient on the inland side is taken as one if the velocity goes to zero.

Table 42: Time-Step Analysis of Storm Surge Flows Along the I-10 Study Segment under the Hurricane Katrina Base Case Scenario

Time (hrs)	Flood Elevation Seaward (ft)	Flood Elevation Inland (ft)	Water Depth Above I-10 Roadway Low Point (ft)	Flow Rate Over I-10 Roadway (cuft/s)	Velocity Over I-10 Roadway (ft/s)	Flow Rate Through Broad St. (cuft/s)	Velocity Through Broad St. (ft/s)	Flow Rate Through TN St. (cuft/s)	Velocity Through TN St. (ft/s)	Flow Rate Through Warren St. (cuft/s)	Velocity Through Warren St. (ft/s)	Erosion Rate of I-10 Embankment (inch/s)
4.5	11.35	11.50	0	0	0	0	0	0	0	0	0	0
5.0	11.84	11.62	0	0	0	0	0	48	1.5	0	0	0
5.5	11.99	11.75	0	0	0	0	0	83	1.9	0	0	0
6.0	12.33	12.06	0	0	0	0	0	215	3.1	0	0	0
6.5	12.65	12.54	0	0	0	0	0	329	3.4	0	0	0
7.0	12.92	12.92	0	0	0	0	0	344	2.9	0	0	0
7.5	13.02	13.02	0	0	0	0	0	186	1.5	0	0	0
8.0	13.00	13.00	0	0	0	0	0	-69	-0.5	0	0	0
8.5	12.93	12.93	0	0	0	0	0	-168	-1.3	0	0	0
9.0	12.65	12.65	0	0	0	0	0	-292	-2.5	0	0	0
9.5	12.55	12.55	0	0	0	0	0	-137	-1.4	0	0	0
10.0	12.22	12.26	0	0	0	0	0	-219	-2.5	0	0	0
10.5	11.87	12.04	0	0	0	0	0	-155	-2.4	0	0	0
11.0	11.38	11.85	0	0	0	0	0	-130	-2.7	0	0	0
11.5	10.83	11.71	0	0	0	0	0	-86	-2.6	0	0	0
12.0	10.21	11.62	0	0	0	0	0	-45	-1.9	0	0	0
12.5	9.85	11.50	0	0	0	0	0	-11	-0.6	0	0	0

Table 43: Time-Step Analysis of Storm Surge Flows Along the I-10 Study Segment under the Hurricane Katrina Shifted Scenario

Time (hrs)	Flood Elevation Seaward @ I-10 Roadway (ft)	Flood Elevation Seaward @ Underpasses (ft)	Flood Elevation Inland (ft)	Water Depth Above I-10 Roadway Low Point (ft)	Flow Rate Over I-10 Roadway (cuft/s)	Velocity Over I-10 Roadway (ft/s)	Flow Rate Through Broad St. (cuft/s)	Velocity Through Broad St. (ft/s)	Flow Rate Through TN St. (cuft/s)	Velocity Through TN St. (ft/s)	Flow Rate Through Warren St. (cuft/s)	Velocity Through Warren St. (ft/s)	Erosion Rate of I-10 Embankment (inch/s)
3.0	11.32	11.32		0	0	0	0	0	0	0	0	0	0
3.5	12.30	12.30	11.50	0	0	0	0	0	238	3.5	0	0	0
4.0	13.25	13.25	11.92	0	0	0	0	0	828	5.7	0	0	0
4.5	15.38	14.33	13.05	0.78	163	2.6	526	5.3	1,460	5.9	497	5.4	0.040
5.0	16.10	15.48	13.97	1.50	948	2.9	1,208	6.3	2,421	6.6	1,300	6.4	0.053
5.5	16.81	16.59	15.48	2.21	2,751	3.3	1,595	5.6	2,805	5.8	1,764	5.7	0.076
6.0	17.70	17.70	16.59	3.10	5,085	2.9	2,210	5.7	3,614	5.8	2,461	5.8	0.054
6.5	18.64	18.64	17.70	4.04	8,627	2.9	2,522	5.3	3,960	5.4	2,808	5.3	0.053
7.0	19.43	19.43	18.64	4.83	11,945	2.8	2,714	4.9	4,151	4.9	3,017	4.9	0.050
7.5	20.02	20.02	19.43	5.42	13,693	2.6	2,612	4.2	3,928	4.3	2,897	4.3	0.040
8.0	20.29	20.29	20.02	5.69	11,195	2.0	1,834	2.8	2,739	2.9	2,032	2.9	0.018
8.5	20.24	20.24	20.29	5.64	-6,018	-1.1	-815	-1.3	-1,218	-1.3	-903	-1.3	0.003
9.0	19.90	19.90	20.24	5.30	-11,985	-2.1	-2,040	-3.2	-3,051	-3.2	-2,260	-3.2	0.023
9.5	19.22	19.22	19.90	4.62	-13,753	-2.8	-2,742	-4.5	-4,137	-4.6	-3,042	-4.6	0.046
10.0	18.42	18.42	19.22	3.82	-10,922	-2.8	-2,628	-4.9	-4,046	-5.0	-2,923	-5.0	0.049
10.5	17.65	17.65	18.42	3.05	-7,071	-2.7	-2,168	-4.8	-3,434	-4.8	-2,415	-4.8	0.042
11.0	16.80	16.80	17.65	2.20	-4,475	-2.7	-1,913	-5.0	-3,135	-5.1	-2,129	-5.1	0.042
11.5	15.91	15.91	16.80	1.31	-2,125	-2.6	-1,527	-5.0	-2,641	-5.2	-1,692	-5.1	0.040
12.0	15.17	15.17	15.91	0.57	-660	-2.8	-1,023	-4.5	-1,927	-4.7	-1,117	-4.6	0.049
12.5	14.44	14.44	15.17	0	-90	-2.5	-715	-4.3	-1,519	-4.6	-756	-4.4	0.035

Time (hrs)	Flood Elevation Seaward @ I-10 Roadway (ft)	Flood Elevation Seaward @ Underpasses (ft)	Flood Elevation Inland (ft)	Water Depth Above I-10 Roadway Low Point (ft)	Flow Rate Over I-10 Roadway (cuft/s)	Velocity Over I-10 Roadway (ft/s)	Flow Rate Through Broad St. (cuft/s)	Velocity Through Broad St. (ft/s)	Flow Rate Through TN St. (cuft/s)	Velocity Through TN St. (ft/s)	Flow Rate Through Warren St. (cuft/s)	Velocity Through Warren St. (ft/s)	Erosion Rate of I-10 Embankment (inch/s)
13.0	13.77	13.77	14.44	0	0	0	-417	-3.9	-1,103	-4.3	-404	-3.9	0
13.5	13.05	13.05	13.77	0	0	0	0	0	-835	-4.3	-142	-3.4	0
14.0	12.60	12.60	13.05	0	0	0	0	0	-418	-3.2	0	0	0
14.5	12.01	12.01	12.60	0	0	0	0	0	-312	-3.4	0	0	0
15.0	11.45	11.45	12.17	0	0	0	0	0	-180	-3.1	0	0	0
15.5	10.98	10.98	11.91	0	0	0	0	0	-105	-2.7	0	0	0
16.0	10.53	10.53	11.75	0	0	0	0	0	-55	-2.1	0	0	0
16.5	10.09	10.09	11.65	0	0	0	0	0	-21	-1.1	0	0	0
17.0	9.75	9.75	11.58	0	0	0	0	0	0	0	0	0	0

Table 44: Time-Step Analysis of Storm Surge Flows Along the I-10 Study Segment under the Hurricane Katrina Shifted + Intensified + SLR Scenario

Time (hrs)	Flood Elevation Seaward @ I-10 Roadway (ft)	Flood Elevation Seaward @ Underpasses (ft)	Flood Elevation Inland (ft)	Water Depth Above I-10 Roadway Low Point (ft)	Flow Rate Over I-10 Roadway (cuft/s)	Velocity Over I-10 Roadway (ft/s)	Flow Rate Through Broad St. (cuft/s)	Velocity Through Broad St. (ft/s)	Flow Rate Through TN St. (cuft/s)	Velocity Through TN St. (ft/s)	Flow Rate Through Warren St. (cuft/s)	Velocity Through Warren St. (ft/s)	Erosion Rate of I-10 Embankment (inch/s)
4.0	11.01	11.01	11.50	0	0	0	0	0	0	0	0	0	0
4.5	12.28	12.28	11.91	0	0	0	0	0	231	3.5	0	0	0
5.0	13.17	13.17	12.96	0	0	0	0	0	762	5.5	0	0	0
5.5	15.26	14.15	14.15	0.66	113	2.6	425	5.0	1,298	5.7	380	5.0	0.037
6.0	15.94	15.32	15.32	1.34	707	2.8	985	5.5	2,030	5.8	1,052	5.6	0.050
6.5	16.65	16.43	16.35	2.05	2,228	3.2	1,516	5.6	2,703	5.8	1,673	5.7	0.070
7.0	17.69	17.69	17.69	3.09	5,379	3.1	2,424	6.3	3,965	6.4	2,698	6.4	0.064
7.5	18.89	18.89	18.89	4.29	10,757	3.2	3,008	6.0	4,681	6.1	3,348	6.1	0.069
8.0	20.25	20.25	20.25	5.65	20,286	3.6	4,121	6.4	6,161	6.5	4,566	6.5	0.095
8.5	21.58	21.58	21.58	6.98	31,090	3.8	5,023	6.4	7,278	6.4	5,530	6.4	0.110
9.0	22.87	22.87	22.87	8.27	42,907	4.0	5,915	6.3	8,340	6.4	6,467	6.4	0.123
9.5	23.88	23.88	23.88	9.28	48,431	3.8	5,924	5.6	8,184	5.6	6,441	5.6	0.105
10.0	24.51	24.51	24.51	9.91	45,242	3.2	5,001	4.4	6,821	4.4	5,418	4.4	0.067
10.5	24.77	24.77	24.77	10.17	33,835	2.3	3,319	2.8	4,503	2.9	3,591	2.9	0.027
11.0	24.96	24.96	24.96	10.36	31,220	2.0	2,930	2.5	3,960	2.5	3,167	2.5	0.020
11.5	24.80	24.80	24.80	10.20	-29,301	-1.9	-2,697	-2.3	-3,645	-2.3	-2,915	-2.3	0.017
12.0	24.19	24.19	24.19	9.59	-47,020	-3.2	-5,096	-4.3	-6,910	-4.4	-5,512	-4.4	0.066
12.5	23.02	23.02	23.02	8.42	-54,136	-4.0	-6,601	-6.0	-9,062	-6.1	-7,164	-6.1	0.123
13.0	21.59	21.59	21.59	6.99	-46,049	-4.2	-6,341	-6.7	-8,914	-6.7	-6,927	-6.7	0.138
13.5	20.32	20.32	20.32	5.72	-30,636	-3.8	-4,911	-6.2	-7,114	-6.3	-5,406	-6.3	0.105

Time (hrs)	Flood Elevation Seaward @ I-10 Roadway (ft)	Flood Elevation Seaward @ Underpasses (ft)	Flood Elevation Inland (ft)	Water Depth Above I-10 Roadway Low Point (ft)	Flow Rate Over I-10 Roadway (cuft/s)	Velocity Over I-10 Roadway (ft/s)	Flow Rate Through Broad St. (cuft/s)	Velocity Through Broad St. (ft/s)	Flow Rate Through TN St. (cuft/s)	Velocity Through TN St. (ft/s)	Flow Rate Through Warren St. (cuft/s)	Velocity Through Warren St. (ft/s)	Erosion Rate of I-10 Embankment (inch/s)
14.0	19.18	19.18	19.18	4.58	-19,569	-3.4	-3,825	-5.9	-5,708	-6.0	-4,236	-5.9	0.081
14.5	18.23	18.23	18.23	3.63	-11,416	-3.0	-2,843	-5.3	-4,381	-5.4	-3,161	-5.4	0.058
15.0	17.41	17.41	17.41	2.81	-6,469	-2.7	-2,147	-4.9	-3,427	-5.0	-2,392	-5.0	0.043
15.5	16.59	16.59	16.59	1.99	-3,653	-2.6	-1,756	-4.9	-2,916	-5.0	-1,953	-4.9	0.039
16.0	15.77	15.77	15.77	1.17	-1,642	-2.5	-1,375	-4.8	-2,418	-5.0	-1,521	-4.9	0.036
16.5	15.23	15.23	15.23	0.63	-482	-2.8	-825	-3.8	-1,583	-4.0	-897	-3.9	0.046
17.0	14.53	14.53	14.53	0	-104	-2.5	-718	-4.2	-1,507	-4.5	-762	-4.3	0.037
17.5	13.88	13.88	13.88	0	0	0	-448	-3.9	-1,140	-4.3	-442	-4.0	0
18.0	13.35	13.35	13.35	0	0	0	0	0	-762	-3.8	-159	-3.1	0
18.5	12.74	12.74	12.74	0	0	0	0	0	-598	-3.9	-7	-2.1	0
19.0	12.24	12.24	12.28	0	0	0	0	0	-333	-3.2	0	0	0
19.5	11.77	11.77	12.02	0	0	0	0	0	-186	-2.8	0	0	0
20.0	11.26	11.26	11.83	0	0	0	0	0	-132	-2.9	0	0	0
20.5	10.88	10.88	11.70	0	0	0	0	0	-75	-2.3	0	0	0
21.0	10.52	10.52	11.62	0	0	0	0	0	-37	-1.6	0	0	0
21.5	10.16	10.16	11.50	0	0	0	0	0	-11	-0.6	0	0	0

Table 45: Flood Volumes Entering Oakdale Neighborhood and Time to Drain with Existing Structures

	Hurricane Katrina Base Case Scenario	Hurricane Katrina Shifted Scenario	Hurricane Katrina Shifted + Intensified + SLR Scenario
Total Volume in ac-ft (m^3)	40 (51,700)	1,300 (1,581,300)	2,800 (3,411,900)
Time to Peak Flood Elevation in Hours	3	5	7
Time to Drain after Peak in Hours	5	9	10.5

It can also be observed that the time to drain is longer than the time to peak, which may be partly due to the slower rate at which the storm surge subsides compared to the rate at which it rises. During the ebb of the surge when it retreats to the sea, the flood volume in the neighborhood drains at a slower rate than the flood levels south and east of the highway. It takes approximately 1.5 to two hours for the water elevations inland of the roadway (in the Oakdale neighborhood) to reach a comparable elevation to that on the seaward side. It is important to note, however, that the estimated time to drain may be less if a larger cross sectional flow area due to the eroded portion of the I-10 roadway embankment is taken into account.

Flow Velocities through Underpasses

Flow rates and velocities through the Broad Street, the Tennessee Street and Railroad, and the Warren Street bridge underpasses were examined to evaluate potential impacts to the bridge abutments and roadways under the bridge. The flow calculations were performed using the FHWA tidal hydraulics orifice approach which was incorporated into the time-step analysis of the storm surges (see previous section) to determine the maximum flow velocities through the underpasses. The results are summarized in Table 46.

Table 46: Peak Flow Velocities through the I-10 Study Segment Underpasses with Existing Structures

	Hurricane Katrina Base Case Scenario ft/s (m/s)	Hurricane Katrina Shifted Scenario ft/s (m/s)	Hurricane Katrina Shifted + Intensified + SLR Scenario ft/s (m/s)
Broad Street Underpass	--	6.3 (1.9)	6.7 (2.0)
Tennessee Street and Railroad Underpass	3.4 (1.0)	6.6 (2.0)	6.8 (2.1)
Warren Street Underpass	--	6.4 (1.9)	6.7 (2.0)

All three bridge crossings within the study site have concrete abutments and concrete roadways under the bridges. Since concrete has permissible velocities up to 18 feet per second (5.5 meters per second) or greater,[349] the majority of the roadway is protected from erosive flow velocities for all storm surge scenarios. However, small sections of median consist of soil and grass, with maximum permissible velocities of only two to four feet per second (0.6 to 1.2 meters per second); lower than or within the range of the projected velocities under each of the surge scenarios. This may be of concern because the bridge piers are located in the grass median. In addition, the Tennessee Street underpass also contains a railroad line. Railroad ballast, assuming two inch (5.1 centimeter) average stone size, has a maximum permissible velocity of three to six feet per second (0.9 to 1.8 meters per second).[350] Thus, the rail line might be vulnerable to the flow velocities under each surge scenario as well.

Step 6 – Identify Adaptation Option(s)

Multiple adaptive concepts were evaluated to address the issues identified with the existing facility in Step 5. The concepts consider what could be done to limit overtopping of I-10, embankment erosion, flooding impacts to the Oakdale neighborhood, and damage to the underpasses from high flow velocities.

The possibility of improvements to I-10 for the purpose of providing flooding protection to the Oakdale neighborhood was immediately ruled out because use of the roadway in this manner exceeds the overall design considerations and standards for the roadway. Additionally, the repurposing of any roadway as a flood protection structure will open the owner / agency up to additional liability concerns in the event that an extreme event breaches the roadway. Given that flood protection is not the primary function of a roadway and that a roadway will fall short of the design standards necessary for a flood protection structure, FHWA currently recommends against owner agencies pursuing this manner of adaptation. For these reasons, this adaptation option was not evaluated in this case study.

The adaptive concepts are first presented with their advantages and disadvantages. This is followed by a more in-depth discussion of the adaptation options that were developed considering differing combinations and variants of the adaptive concepts.

- **Widen Underpass(es) by Lengthening the Bridge(s)**

  Description:

  This concept involves lengthening one or more of the bridges to widen the underpass(es).

  Advantages:

  – Decreases flow velocities through the underpasses

  Disadvantages:

  – Relatively large capital and maintenance cost

[349] Fischenich, 2001
[350] Fischenich, 2001

- Major disruption to traffic for long periods of time during construction

Analysis showed that widening the underpasses decreases flow velocities through the underpasses, but not by a significant amount. This is due to the fact that the existing underpasses have capacity to convey the storm surge flows inland without significant attenuation, and the limited inland volume capacity causes the inland flood elevation to rise quickly, keeping the head differential[351] across the underpasses low. Thus, widening the underpasses does not cause a significant decrease in the head differential. Even with significant widening (for example by five times the original width), maximum velocities through the underpasses only decrease from 3.4 to 2.5 feet per second (one to 0.8 meters per second) for the Hurricane Katrina Base Case Scenario storm surge, from 6.6 to 5.8 feet per second (two to 1.8 meters per second) for the Hurricane Katrina Shifted Scenario, and 6.8 to 6.7 feet per second (2.1 to two meters per second) for the Hurricane Katrina Shifted + Intensified + SLR Scenario. Underpass widening may be a more reasonable option at project sites with more flow constriction through the underpasses or larger inland volume capacity. However, at this study site, this concept was excluded from the subsequent evaluation of adaptation options because it would be a large project undertaking with large capital cost and minimal effect on flow velocities.

- **Harden Underpass**

Description:

This concept involves hardening grassed areas within one or more of the underpasses with concrete and converting the segment of the railroad track at the Tennessee Street and Railroad underpass to a direct-fixation track.[352]

Advantages:

- Protects the bridge structures by preventing erosion of the areas within the underpasses
- Relatively low capital and maintenance cost

Disadvantages:

- Disruption to traffic during construction
- Requires installation of concrete approach slabs beneath the ballasted track to transition between the ballasted and the direct-fixation segments of the track

- **Raise I-10 Roadway**

Description:

This involves raising the I-10 roadway at the low point to protect the embankment from overtopping flows.

Advantages:

- Protects I-10 roadway from flood overtopping depending on proposed roadway elevation

[351] Head differential in this case is the difference between the storm surge flood elevation and the inland flood elevation above the top of the roadway.
[352] A direct fixation track involves rail attached to concrete plinths (platforms) mounted on concrete slabs

Disadvantages:

- Relatively large capital cost
- Major disruption to traffic for long periods of time during construction
- May require retaining wall if lateral right-of-way is insufficient for proposed roadway elevation

- **Armor I-10 Roadway Embankment**

  Description:

  This involves armoring the I-10 roadway embankment with vegetated, permanent reinforcement matting.

  Advantages:

  - Protects the I-10 roadway embankment slopes from erosion and prevents breaching
  - Relatively low capital cost
  - Low disruption to traffic

  Disadvantages:

  - Does not prevent flood overtopping
  - Requires periodic maintenance and replanting when necessary

The option of raising the base of the underpass by approximately one foot (0.3 meters) to prevent the Hurricane Katrina Base Case Scenario storm surge from entering was also considered as an alternative. However, the stream channel crossing under the I-10 embankment in the vicinity of the underpass would allow flood flows to enter despite raising the bottom elevation of the underpass. Doing this may also impose upon the FEMA regulated floodplain along Tennessee Street. This option was therefore excluded.

The discussions of adaptation options below provide examples of how the above adaptation concepts can be used in combination to mitigate the impacts of predicted storm surge flooding.

<u>Option One – Harden Underpasses (Tennessee Street and Railroad Underpass Only)</u>

Analysis in Step 5 showed that flooding of the underpasses will likely produce sufficiently high flow velocities through the underpasses that may undermine the grass median and eventually the bridge piers. The Tennessee Street and Railroad underpass is particularly vulnerable due to its lower elevation compared to the other two underpasses as well as the railroad track that passes through the underpass. To address these issues, this adaptation option involves hardening the grassed areas within the Tennessee Street and Railroad underpass with concrete and converting the segment of the railroad track to a direct-fixation track.

This option protects this underpass from erosive flow velocities that would be created by the Hurricane Katrina Base Case Scenario storm surge, which reaches a flood elevation of 13 feet (four meters) and causes flooding of the Tennessee Street and Railroad underpass that leads to the flooding of a portion of the Oakdale neighborhood. Since no major changes to the dimensions and geometry of the underpasses or the I-10 roadway are proposed for this option, the flow rates, velocities, and degree of inland flooding remain unchanged compared to the existing facility. Hardening the grassed areas with a non-erodible material such as concrete would help to prevent erosion of the soil and undermining of the bridge structure. Concrete, with a permissible flow velocity of over 18 feet per second (5.5 meters per second),[353] can resist the shear forces caused by flow velocities of up to 3.4 feet per second (one meter per second) experienced under the Hurricane Katrina Base Case Scenario at this underpass. The railroad ballast, assuming two inch (5.1 centimeter) diameter stone with a maximum permissible velocity of three to six feet per second (0.9 to 1.8 meters per second), may also be vulnerable to these flows. Converting the segment of railroad track at the underpass to a direct-fixation track will provide higher resistance to erosive forces compared to a ballasted track. This would also involve installation of concrete approach slabs beneath the ballasted track to transition between the ballasted and the direct-fixation segments of the track.

<u>Option Two – Harden (All) Underpasses and Raise I-10 up to 21 feet (6.4 meters)</u>

This option addresses the impacts associated with erosive flows through all three underpasses and overtopping of the I-10 roadway which can cause erosion of the embankment. This option involves the following measures:

- Elevating I-10 to 21 feet (6.4 meters) through raising the lowest point of the roadway by 6.4 feet (two meters).
- Hardening all grassed areas within all three underpasses with concrete and converting the segment of the railroad track at the Tennessee Street and Railroad underpass to a direct-fixation track

This option protects all three underpasses from erosive flow velocities that would be created by the Hurricane Katrina Shifted Scenario, which reaches a flood elevation of 20.3 feet (6.2 meters), causing flooding of all three underpasses and overtopping the existing I-10 roadway by 5.7 feet (1.7 meters). Analysis showed that raising the roadway to 21 feet (6.4 meters) only increases maximum flow velocities through the underpasses from approximately 6.6 feet (2.0 meters per second) to 6.9 feet per second (2.1 meters per second) during the Hurricane Katrina Shifted Scenario. Hardening the grassed areas within all three underpasses with a non-erodible material such as concrete, and converting the segment of railroad track at the Tennessee Street and Railroad underpass to a direct-fixation track in the same manner described under Option One, can resist shear forces caused by flow velocities at the underpasses of the Option Two facility.

This option would also prevent any overtopping of the I-10 roadway by the Hurricane Katrina Shifted Scenario storm surge flood and eliminate the potential for embankment erosion under this scenario. Analysis also showed that raising the roadway does not help to prevent storm surge flows from entering the neighborhood because the existing underpasses have significant capacity to convey storm surge flows inland, and the limited inland volume capacity causes the inland flood elevation to rise quickly. The underpasses have the capacity to convey all the flow with

[353] Fischenich, 2001

little attenuation, and thus the amount of inland flooding caused by the storm surge is virtually unaffected by the height of the roadway embankment. Therefore, this adaptation options does not significantly alter the amount of flooding in the neighborhood. This may not be the case at project sites with more flow constriction through the underpasses or larger inland volume capacity.

<u>Option Three – Harden (All) Underpasses and Raise I-10 up to 26 feet (7.9 meters)</u>

This option addresses the impacts associated with erosive flows through all three underpasses and overtopping of the I-10 roadway which can cause erosion of the embankment. This option involves the following measures:

- Elevating I-10 to 26 feet (7.9 meters) through raising the lowest point of the roadway by 11.4 feet (3.5 meters)
- Hardening all grassed areas within the underpasses with concrete and converting the segment of the railroad track at the Tennessee Street and Railroad underpass to a direct-fixation track

This option protects all three underpasses from erosive flow velocities that would be created by the Hurricane Katrina Shifted + Intensified + SLR Scenario, which reaches a flood elevation of 25 feet (7.6 meters), causing flooding of all three underpasses, and overtopping the existing I-10 roadway by 10.4 feet (3.2 meters). Analysis showed that raising the roadway to 26 feet (7.9 meters) only increases maximum flow velocities through the underpasses from approximately 6.8 feet (2.1 meters per second) to 7.2 feet per second (2.2 meters per second) during the Hurricane Katrina Shifted + Intensified + SLR Scenario. Hardening the grassed areas within all three underpasses with a non-erodible material such as concrete, and converting the segment of railroad track at the Tennessee Street & Railroad underpass to a direct-fixation track in the same manner described under Options One and Two, can resist shear forces caused by flow velocities at the underpasses of the Option Three facility.

This option would also prevent any overtopping of the I-10 roadway by the Hurricane Katrina Shifted + Intensified + SLR Scenario storm surge flood and eliminate the potential for embankment erosion under this scenario. As true in the case of Option Two, analysis also showed that raising the roadway does not help to prevent storm surge flows from entering the neighborhood because the rate of surge rise is slow enough where the underpasses can handle all the flow with little attenuation, and the amount of inland flooding caused by the storm surge is virtually unaffected by the height of the roadway embankment. Therefore, this adaptation option does not significantly alter the amount of flooding in the neighborhood. This may not be the case at project sites with more flow constriction through the underpasses or larger inland volume capacity.

<u>Option Four – Harden (All) Underpasses and Armor I-10 Roadway Embankment</u>

This option addresses the impacts associated with erosive flows through all three underpasses and over the I-10 roadway embankment. This option involves the following measures:

- Armoring both sides of the I-10 roadway embankment with vegetated, permanent reinforcement matting, within the segment that is subject to flooding
- Hardening all grassed areas within the underpasses with concrete and converting the segment of the railroad track at the Tennessee Street and Railroad Underpass to a direct-fixation track

This option protects all three underpasses and the I-10 roadway embankment from erosive flow velocities that would be created by the Hurricane Katrina Shifted + Intensified + SLR Scenario, which reaches a flood elevation of 25 feet (7.6 meters), causing flooding of all three underpasses and overtopping the existing I-10 roadway by 10.4 feet (3.2 meters). Instead of raising the roadway embankment, this option allows the roadway to be overtopped by storm surge flows while protecting the embankment slopes from the erosive flows with less erodible material.

Since no major changes to the dimensions and geometry of the underpasses or the I-10 roadway are proposed for this option, the flow rates, velocities, and time to drain inland flood waters remain unchanged compared to the existing facility. The material proposed for lining the embankment slopes is vegetated reinforcement matting composed of woven synthetic fibers or other permanent material. This material has a maximum permissible velocity of five to seven feet per second (1.5 to 2.1 meters per second) and shear stress of three to five pounds per square foot[354] (144 to 239 pascals) depending on the type. These materials are designed to a sufficient strength to withstand the maximum flow velocity of 4.2 feet per second (1.3 meters per second) and shear stress of 1.7 pounds per square foot (81.4 pascals) expected to be produced by the Hurricane Katrina Shifted + Intensified + SLR Scenario storm surge on the embankment. Using vegetation to resist erosive flows on the slopes will require periodic maintenance and possible replanting to preserve the integrity of the vegetation; however, frequent mowing is not necessary if seed mixes with certain herbaceous species are used, such as annual ryegrass, bermudagrass, tall fescue, weeping lovegrass, or annual lespedeza.[355] The areas requiring lining are both slopes of the embankment within the anticipated submerged segment of roadway, approximately 2,240 feet (683 meters) in length and three to 13.5 feet (0.9 to 4.1 meters) in height.

Hardening the grassed areas within all three underpasses with a non-erodible material such as concrete, and converting the segment of railroad track at the Tennessee Street and Railroad underpass to a direct-fixation track in the same manner described under Options One through Three, can resist shear forces caused by flow velocities at the underpasses.

Step 7 – Assess Performance of the Adaptation Option(s)

Table 47 summarizes how well each of the three proposed adaptation options performs under each of the storm surge scenarios. If these adaptation options actually were being considered for design, a full analysis quantifying the performance of each option under each scenario would need to be conducted and the results used in the economic analysis in Step 8.

[354] Fischenich, 2001
[355] ALDOT, 2012

Table 47: Performance Summary of the Adaptation Options for the I-10 Study Segment

	Hurricane Katrina Base Case Scenario	Hurricane Katrina Shifted Scenario	Hurricane Katrina Shifted + Intensified + SLR Scenario
Option One: Harden Underpass (Tennessee Street & Railroad Underpass Only)	• Protects the Tennessee Street & Railroad Underpass against erosive flow velocities	• Does not protect I-10 roadway embankment, Broad Street Underpass, or Warren Street Underpass against erosive flow velocities.	• Does not protect I-10 roadway embankment, Broad Street Underpass, or Warren Street Underpass against erosive flow velocities.
Option Two: Harden (All) Underpasses and Raise I-10 Roadway up to 21 feet (6.4 meters)	• Protects Tennessee Street & Railroad Underpass against erosive flow velocities.	• Prevents storm surge from overtopping I-10 roadway • Protects all three underpasses against erosive flow velocities.	• Protects all three underpasses against erosive flow velocities. • Does not protect I-10 roadway embankment from erosive flow velocities
Option Three: Harden (All) Underpasses and Raise I-10 Roadway up to 26 feet (7.9 meters)	• Protects Tennessee Street & Railroad Underpass against erosive flow velocities.	• Prevents storm surge from overtopping I-10 roadway • Protects all three underpasses against erosive flow velocities.	• Prevents storm surge from overtopping I-10 roadway • Protects all three underpasses against erosive flow velocities.
Option Four: Harden (All) Underpasses and Armor I-10 Roadway Embankment	• Protects Tennessee Street & Railroad Underpass against erosive flow velocities.	• Protects I-10 roadway embankment and all three underpasses from erosive flow velocities • Does not prevent storm surge from overtopping I-10 roadway	• Protects I-10 roadway embankment and all three underpasses from erosive flow velocities • Does not prevent storm surge from overtopping I-10 roadway

Step 8 – Conduct an Economic Analysis

An economic analysis was not conducted for this case study. Refer to Section 4.4.1 for an example of how an economic analysis can be conducted for an adaptation study. That said, a general comparisons of the benefits and relative costs of each adaptation option can provide a great deal of insight toward decision-making, even before a formal economic analysis is conducted. Priority should be given to protecting the assets with the highest value, which, for this study, are the bridges at the three underpasses and the railroad track. As shown in Table 47, hardening one or more the underpasses by replacing the grassed areas with concrete and replacing the railroad ballast with a direct fixation track will provide protection of the underpasses under all three storm surge scenarios. At a relatively low cost compared to the other options, this course of action is promising.

As for protection of the I-10 roadway, while raising the roadway would prevent storm surge flows from overtopping the roadway embankment, analysis has shown that this would have very little effect on the amount of inland flooding. The main concern would be potential erosion and breaching of the embankment, which can be mitigated by armoring the embankment slopes with vegetated reinforcement matting. A raised roadway may allow I-10 to remain functional during a storm surge event, but the relative benefit of being able to keep I-10 functioning as opposed to allowing it to flood would need to be conducted. Analysis may reveal that raising the roadway would provide minimal added benefit at much greater capital costs and disruption to traffic during construction compared to armoring the embankment.

Step 9 – Evaluate Additional Decision-Making Considerations

Additional considerations that address environmental and social concerns may influence the decision-making process. The following are potential action items specific to this study site that may need to be addressed when selecting adaptation alternatives:

- Community outreach and public involvement are necessary courses of action, particularly if the underpass widening option is considered. Widening bridges may cause concern among public stakeholders about potential increased flooding, construction disturbances, and costs. As such, public meetings would need to be held to explain the impacts and convey the facts to the public. Coordination with individual property owners would also be necessary if the adaptation is expected to impact specific properties.
- It may be necessary to develop (or revise existing) evacuation plans to reflect the storm surge flooding events predicted by this study. Evacuation schedule and emergency response coordination can be planned based on predicted storm surge peaking times and time for inland areas to drain. These activities should be closely coordinated with the local emergency management agency.
- Continued maintenance and asset management are necessary to ensure the integrity of the new facility. For example, if vegetative plantings and reinforcement matting are used on the embankment slopes, regular inspection and maintenance would be required to identify areas for replanting and ensure that the embankment is sufficiently protected against erosive flows.

Step 10 – Select a Course of Action

Although a formal economic analysis was not performed for the adaptation alternatives proposed, Option 4 provides the most protection against the impacts of storm surge flows for relatively low capital costs. Hardening the underpasses by replacing the grassed areas with concrete and replacing the railroad ballast with a direct fixation track provides protection of the underpasses under all three storm surge scenarios. Although some maintenance is necessary, armoring the embankment slopes with vegetated reinforcement matting can provide significant protection of the I-10 roadway embankment under all three storm surge scenarios.

Step 11 – Plan and Conduct Ongoing Activities

Whatever option is chosen, performance of the new facility should be monitored after completion of the project, and impacts to the facility should be periodically assessed. These activities will help to determine if design thresholds are being met and if so, whether the facility is meeting design goals. Potential additional improvements can then be evaluated based on monitoring and assessment findings, and any lessons learned can be applied to future projects. Agencies should also continue to monitor climate projections as they change with the advancement of climate knowledge and modeling capabilities.

Also, if one of the adaptation options involving vegetative plantings and reinforcement matting are used on the embankment slopes, regular inspection, maintenance, mowing and replanting would be required to ensure that the embankment is sufficiently protected against erosive flows.

Conclusions

This case study provides an example of how a road or rail alignment can be evaluated for climate change impacts resulting from storm surge flooding and sea level rise. The I-10 alignment between mileposts 24 and 25 was studied under three plausible future storm surge scenarios. It was determined that all storm surge scenarios could impact the Oakdale neighborhood north and west of the highway and cause potential erosion problems at the Tennessee Street and Railroad Underpass. The Hurricane Katrina Shifted Scenario and the Hurricane Katrina Shifted + Intensified + SLR Scenario would, in addition, present erosion problems at the other two underpasses, overtop I-10 and cause erosion concerns along the embankment at the location of overtopping. A variety of adaptation options were proposed to lessen the impacts of each storm surge scenario.

During the analysis it was found that the area that was most lacking in current research was the topic of embankment breaching. Many studies have aimed to establish estimates of flow rates and breach dimensions for earthen dams and levees, but not many have developed methods to predict the onset of embankment breaching, or that focus on highway embankments. This is an area of future research that would be needed in order to more accurately predict the impact of storm surge flooding on highway embankments.

4.4.7 Coastal Tunnel Exposure to Storm Surge – The I-10 (Wallace) Tunnel

Introduction

Underwater coastal tunnels are particularly vulnerable to storm surge. This section contains a brief summary of the study *Storm Surge Analysis for the I-10 Tunnel* performed by Douglass et al. (2007) on the I-10 tunnel under the Mobile Ship Channel, in which a "design storm" method for risk-based coastal design decisions was developed that closely matches the *General Process for Transportation Facility Adaptation Assessments*. This method was applied and the tunnel was found to have vulnerability to flooding in a hurricane. Various approaches to dealing with this issue have been developed, including replacing the tunnel with a bridge, and these are being assessed by ALDOT.

> **Case Study Highlights**
>
> **Purpose:** Evaluate whether an underwater coastal tunnel could be flooded during hurricane events due to surge entering air vents or the non-gated tunnel entrance.
>
> **Approach:** This case study was adapted from a previous study conducted by Douglass et al (2007). Using a three-step modeling process to quantify the risk of flooding under present day conditions. The USACE ADCIRC model was used to simulate storm surge. The USACE EST model was used to estimate the storm surge frequency relationship. Then, a weir flow model and EurOtop (wave overtopping model) were used to model a flood hydrograph. These modeling efforts allowed Douglass et al. to estimate surge elevations for the 100-year and 150-year storm, and then compare the surge elevations to the engineering design of the tunnel.
>
> **Findings:** Douglass et al (2007) found that the tunnel could be flooded during storms equal to or greater than the 75-year storm.
>
> **Viable Adaptation Options:**
>
> - Raise the west portal wall elevation
> - Raise all approach walls
> - Install temporary flood gates
>
> **Other Conclusions:** The Saffir-Simpson Hurricane Category scale is not particularly useful for engineering decisions because hurricanes are assigned categories based on wind speeds, not storm surge.

Note that the Douglass study did not include an assessment of future sea level rise and accompanying higher storm surge. However, the analysis is included as a case study in this report to illustrate how the *General Process for Transportation Facility Adaptation Assessments* can be applied not only to climate change analyses but also to situations where estimates of current storm surge return periods are known to be out of date and new (higher) estimates of current surge probabilities are developed to reassess present day risk.[356]

Application of the General Process for Transportation Facility Adaptation Assessments

Step 1 – Describe the Site Context

The I-10 Tunnel (also known as the George C. Wallace Tunnel) crosses under the Mobile Ship Channel (the Mobile River) north of Mobile Bay (see Figure 83 and Figure 84). The tunnel is part of the interstate highway system and is located at the west end of a seven-mile elevated

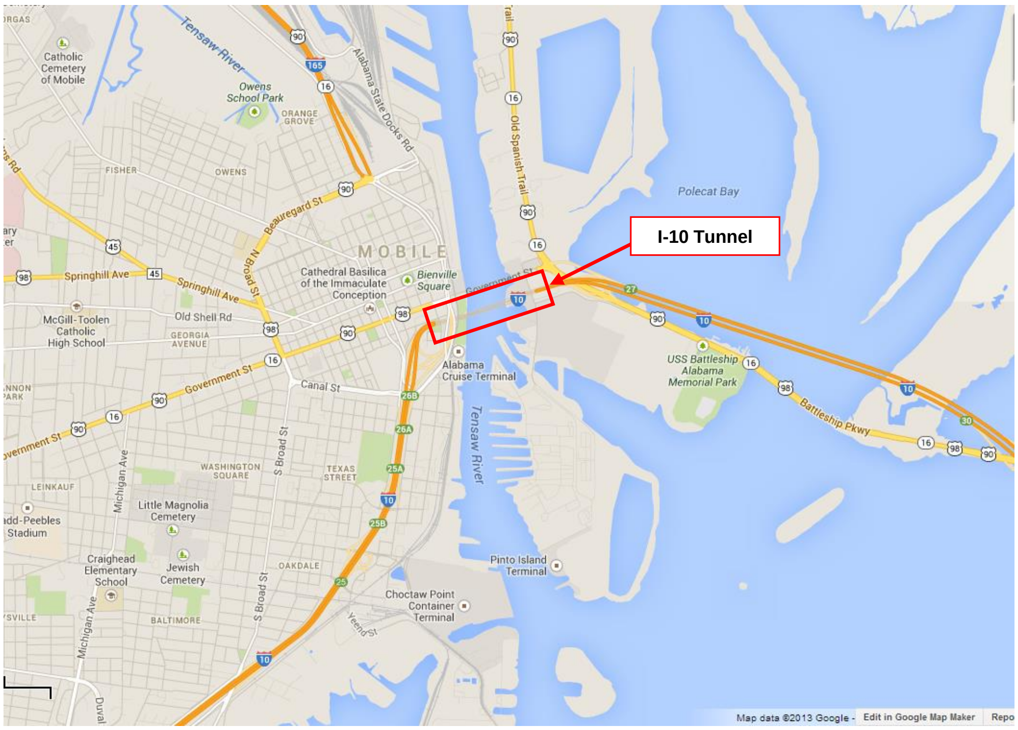
[356] Changes in flood elevations are a common situation when FEMA updates their flood mapping (both in coastal and riverine environments) or when NOAA updates their precipitation return period estimates as with the release of NOAA Atlas 14. As illustrated by this case study, the *General Process for Transportation Facility Adaptation Assessments* can be applied to reassess risks to facilities when these changes in present day flood elevation estimates are released.

causeway across the north end of Mobile Bay. The I-10 Tunnel carries Annual Average Daily Traffic (AADT) of about 70,000 vehicles per day and was designated as a critical asset by the analysis presented in the *Assessing Infrastructure for Criticality in Mobile, Alabama* report.[357]

Step 2 – Describe the Existing Facility

The I-10 Tunnel (opened in 1972) is 3,000 feet (914.4 meters) long and consists of twin tubes carrying two lanes of traffic each in each direction. The approaches total 1,300 feet (396.2 meters) in length. The existing crest elevation of the west portal wall (see Figure 85) is 16 feet[358] (4.9 meters) and the crest elevation of the east portal wall is 19 feet (5.9 meters).

Figure 83: Map Showing the Location of the I-10 Tunnel within the Mobile Metropolitan Area[359]



[357] USDOT, 2011
[358] All elevations in this study are relative to the North American Vertical Datum of 1988 (NAVD88) unless otherwise noted.
[359] Source of basemap: Google Maps (as modified)

Figure 84: Aerial Photograph of the I-10 Tunnel Location[360]

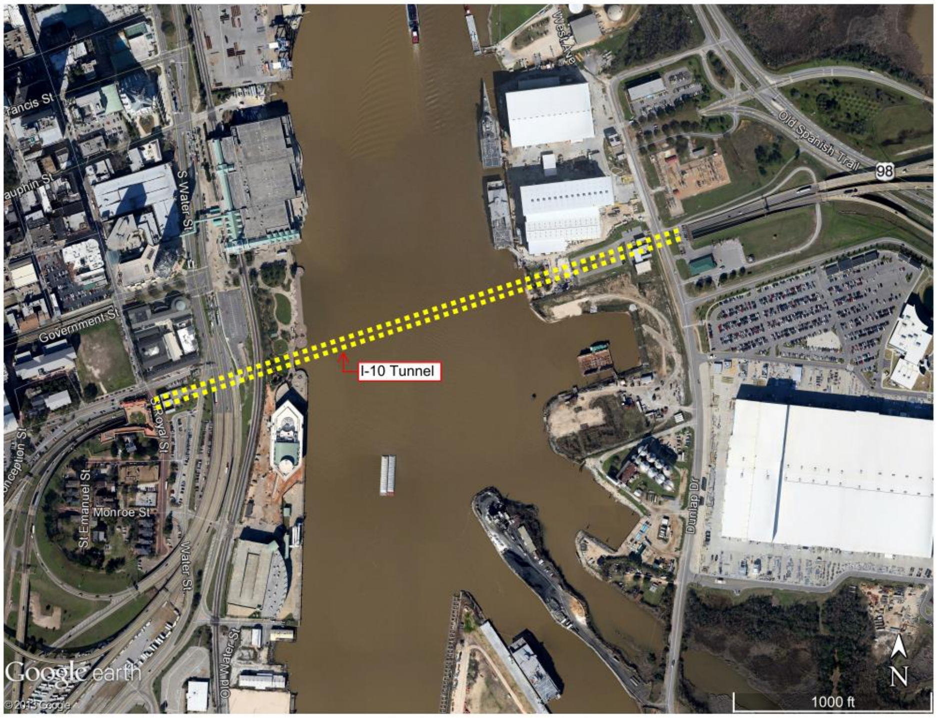


Figure 85: West Portal of the I-10 Tunnel[361]

[360] Source of basemap: Google Earth (as modified)
[361] Douglass, Scheffner, and Kellogg Brown & Root Services, Inc., 2007

Step 3 – Identify Environmental Stressors That May Impact Infrastructure Components

Storm surges, the primary environmental stressor that could affect the I-10 Tunnel, are the focus of this case study.

Step 4 – Decide on Climate Scenarios and Determine the Magnitude of Changes

The I-10 Tunnel reports, *Storm Surge Analysis for the I-10 Tunnel*[362] and *Wave Overtopping Study Report for the I-10 Tunnel*,[363] analyzed storm surge and wave overtopping as the principal threat to the tunnel. The studies evaluated the vulnerability of the transportation asset to extreme events, however, they did not specifically consider climate change scenarios. That said, the methods and results have some aspects similar to climate change impacts. A component of the studies was the re-evaluation of the relationship between risk and storm surge and the result is a peak 100-year storm surge elevation three to four feet (0.9 to 1.2 meters) higher than the existing FEMA maps. This is similar in magnitude to a reasonable sea level rise scenario assumption for this area. At the time of this study, the existing FEMA flood maps were over 25 years old, the adjacent county to the west had just been restudied by FEMA, resulting in significant increases in the estimated flood levels, and FEMA efforts to restudy the basic surge-frequency relationship in Mobile County were years away from being completed. Hurricane Katrina had just caused the highest storm surge in the previous 50 years and it made landfall 100 miles from Mobile County. Thus, there was concern that the storm surge at the tunnel would have been much more severe had the storm made landfall closer to Alabama.

<u>Water Level Values</u>

Table 48 shows several of the more recent, measured, high water levels around the tunnel.

---

[362] Douglass et al., 2007
[363] Douglass, 2008

Table 48: Historic High Water Marks near the I-10 Tunnel[364]

Rank	High Water Mark Elev. (ft. NAVD)	Storm Name	Date
1	12.4[365]	Katrina	2005
2	11.7	Frederic	1979
3	9.4	Georges	1998
4	7.4	Camille	1969
5	4.9	Ivan	2004
6	3.8	Elena	1985

Realizing the tunnel's vulnerability after Hurricane Katrina, Douglass et al. (2007) were tasked with developing a risk-based approach to coastal design decisions: specifically, the likelihood of the tunnel flooding during hurricanes. It is well understood that storm surge at a specific site is more sensitive to the track of the storm than the storm's "category" as measured on the Saffir-Simpson scale. The Saffir-Simpson scale is based on wind speed rather than storm surge, making it problematic for engineering decisions when storm surge is the damage mechanism. Traditional risk-based design return periods can be used effectively, however. The study team used a three-step modeling procedure to develop quantitative estimates of the risk of flooding in the existing tunnel according to present day climate conditions. The first two related modeling steps quantify the coastal storm surge–frequency relationship at any coastal location where tropical storms are a dominant phenomenon. The procedure has been developed by USACE and FEMA for coastal flood analysis and mapping. Two USACE computer models – ADCIRC (ADvanced CIRCulation Model for Oceanic, Coastal and Estuarine Waters) and the Empirical Simulation Technique (EST) model were used in these studies.

The ADCIRC model can be used to simulate tidal circulation and / or coastal storm surge. Other hydrodynamic models exist but ADCIRC is in the public domain and has proven capable of modeling coastal hydrodynamics at a high-resolution very well in a variety of situations. The study team included a coastal numerical modeling specialist (Dr. Norman Scheffner) as is typically required for the use of high-performance, high-resolution, hydrodynamic models like ADCIRC. Model validation for this study was done with both tidal simulations and storm surges. Storm events were verified by comparing simulated peak surge elevations to historical observations at the location of the tunnel. The model was then used to simulate all storms that

[364] Source: Douglass et al., 2007. Note: High-water marks are within the general vicinity of the I-10 Tunnel. These values are the higher elevation values reported along the north end of the bay. The values are relative to different datums and include NAVD88, mean sea level (the average of the water elevations recorded at each hour of the day over the current National Tidal Datum Epoch [the specific 19-year period over which NOAA uses to obtain observations that are used to develop tidal statistics]), and the National Geodetic Vertical Datum of 1929 (NGVD). The difference between these datums is less than 0.5 feet (15.2 centimeters).
[365] FEMA, 2006

significantly impacted the study area since 1886. In order to insure that the most possible severe events were included, simulations included hypothetical events that could occur. For example, the tracks of the most intense events were shifted along the coast such that the study area experienced maximum surge elevations.

In the second step in the modeling procedure, following all numerical simulations, the database of computed surges were used as input for the EST – a statistical model that simulates life-cycle sequences of cyclic but nondeterministic multi-parameter systems (such as storm events) and their corresponding environmental impacts. This approach can be used in place of a joint probability method for developing the storm surge–frequency relationship. A basic assumption of either method of assigning frequency to specific surge elevations is that future events will be statistically similar in magnitude and frequency to past events: again, climate change was not considered in this study.

The third step in the modeling procedure was flood hydrograph modeling using a combination of a weir[366] flow model and EurOtop, a wave overtopping model. The Douglass et al. (2007) study estimated the storm surge high water elevations are 16.7 feet (5.1 meters) for the 100-year (one percent annual exceedance probability) storm return period and 19 feet (5.8 meters) for the 150-year (0.7% annual exceedance probability) storm return period.[367]

Step 5 – Assess Performance of the Existing Facility

Past performance is a starting point to understanding the impacts under potential future storms. During Hurricane Katrina, the I-10 Tunnel experienced some limited flooding, including flooding of the air shafts due to failure of the outflow pipe, which led to back flow. Filling of air shafts can lead to closing of the tunnel because of the potential for carbon monoxide poisoning.

The following subsection discusses the critical flooding thresholds with respect to various potential storms as determined by Douglass et al. (2007).

<u>Flooding Threshold</u>

The results of the analyses show that the tunnel could flood during hurricanes as shown in Table 49. The 100-year storm surge elevation (i.e., the surge level with a 1% chance of exceedance in any year) is estimated at 16.7 feet (5.1 meters). The fourth and fifth columns of Table 49 give the risk of occurrence (of flooding) for two different design lives, 20 years and 50 years, for each of the return periods. These design lives, 20 years and 50 years, were selected for demonstration purposes only. The probability of that flood level being exceeded during a 20-year or 50-year

[366] A weir is a barrier across a river (smaller than a dam) characterized by the allowance of water to flow over the top of the barrier.
[367] Douglass et al., 2007

time period is 18% and 39%, respectively.[368] The risk values shown in these columns are derived using a form of the binomial distribution common in quantitative risk analysis.[369]

Table 49: Projected Storm Surge Flooding Elevations by Exceedance Probabilities for the I-10 Tunnel over Hypothetical Remaining Design Lifespans[370]

Storm Return Period (years)	Storm Surge Elevation (Ft. NAVD)	Probability of Exceedance in any Year	Probability of Exceedance in a 20-Year Design Life	Probability of Exceedance in a 50-Year Design Life	Estimated Flooding Levels in the I-10 Tunnel (Ft.)
25	9.8	4.0%	56%	87%	0
50	13.4	2.0%	33%	64%	0
75	15.4	1.3%	24%	49%	0-3
100	16.7	1.0%	18%	39%	Full
150	19.0	0.7%	13%	28%	Full
200	20.2	0.5%	10%	22%	Full

Step 6 – Identify Adaptation Option(s)

Previous work by others had recommended raising the elevation of the west portal wall to 19 feet (5.8 meters) to match the elevation of the east portal wall (**Option A**). The initial, primary goal of this study was to quantify the additional level of flood protection provided by such an approach. Interestingly, this study showed that such an approach alone would only provide a relatively limited level of additional flood protection because of wave overtopping at the more exposed east portal. Thus, the study evolved to consider several other adaptation options including:

- **Option B:** Raise all approach walls to elevation 19 feet (5.8 meters) and construct a berm / seawall around the east portal to reduce wave overtopping
- **Option C:** Install temporary flood gates. The surge-frequency analysis from this study indicates that extremely high surge levels could occur at this site. The only completely "storm-proof" alternative would be to close the tunnel with temporary flood gates to be deployed as large storms approached.[371]

Step 7 – Assess Performance of the Adaptation Option(s)

The study found the following results for the three adaptation options across the surge scenarios developed:

[368] Douglass, 2008
[369] See for example Equation 4.81 in FHWA, 2002a.
[370] Source: Douglass, 2008. Note: The estimated flooding levels are water depths in the low-point in the tunnel. These estimated depths include both wave overtopping and weir flow over the crest of the portal walls. "Full" means the entire tunnel will be filled with seawater.
[371] Douglass, 2010

- **Option A:** Raising all approach walls to elevation 19 feet (5.8 meters) would result in a relatively small additional level of flood protection: no flooding for the 75-year event, some very limited flooding for the 100-year event, and complete flooding for the 150-year event.[372]
- **Option B:** Raising all approach walls to elevation 19 feet (5.8 meters) and constructing a berm / seawall near the east portal could provide significantly more protection: no flooding for all scenarios up to the 150-year event.[373] A berm or seawall that extended to a high enough elevation would reduce or eliminate waves at the approach wall and thus reduce wave overtopping. This option is meant to reduce the waves incident on the approach walls, but will not prevent the surge from reaching the approach walls. Several things should be noted for this adaptation option. First, this option was developed to take advantage of some of the site-specific characteristics. Wave heights may be much higher immediately to the east of the east portal (see Figure 86) due to the lower ground elevation and because the service access road is not present to break up the waves (i.e., the waves are not depth-limited). Second, the potential for wave uplift forces on the first bridge span to the east of the east portal may be enough to cause failure of this seven-mile stretch of highway regardless of the protection provided by this adaptation option.[374]
- **Option C:** Temporary flood gates would protect the tunnel from all storms including catastrophic storms such as the 500-year event. ALDOT has experience with temporary flood gates used to protect the adjacent, older, smaller Bankhead Tunnel under the Mobile Shipping Channel. Significant operational issues related to closing an interstate highway during hurricane approach would have to be addressed.

The results for flooding under adaptation options A and B, as compared to existing conditions, are provided in Table 50.

[372] Douglass, 2008
[373] Douglass, 2010
[374] Douglass et al., 2007; Douglass, 2008; Douglass, 2010

Figure 86: Portion of I-10 Immediately East of the I-10 Tunnel Portal[375]

Table 50: Projected Water and Flooding Levels at the I-10 Tunnel with Adaptation Options A and B[376]

Storm Return Period (Years)	Storm Surge Elev. (ft. NAVD)	Existing I-10 Tunnel Lowest Wall Crest Elevation (Ft. NAVD)	Estimated Flooding Levels: Existing I-10 Tunnel (Ft. NAVD)	Estimated Flooding Levels: Option A – All Walls Raised to Elev. 19 (Ft. NAVD)	Estimated Flooding Levels: Option B – All Walls Raised to Elev. 19 + Berm / Seawall (Ft. NAVD)
25	9.8	16	0	0	0
50	13.4	16	0	0	0
75	15.4	16	0-3 ft.	0	0
100	16.7	16	Full	0-6 ft.	0
150	19.0	16	Full	Full	0-6 ft.
200	20.2	16	Full	Full	Full

Step 8 – Conduct an Economic Analysis

An economic analysis was not included in this portion of this case study but is recommended for facility-level adaptation assessments. Following this study, the tunnel's owner, ALDOT,

[375] Douglass et al., 2007
[376] Source: Douglass, 2008. Note: The estimated flooding levels are water depths in the low-point in the tunnel. These estimated depths include both wave overtopping and weir flow over the crest of the portal walls. "Full" means the entire tunnel will be filled with seawater.

developed construction cost estimates for each of the adaptation design options mentioned above.

Step 9 – Evaluate Additional Decision-Making Considerations

A number of additional decision-making considerations unique to this facility were required. First, the west portal of this tunnel is located in downtown Mobile immediately adjacent to several historically significant buildings including Fort Conde. This leads to several technically challenging issues related to the geotechnical and structural engineering which would be required for any adaptation option. Second, traffic congestion is an issue at the tunnel: in fact, there is an effort underway to replace the I-10 Tunnel under the Mobile Ship Channel with a large bridge to address the traffic delays typical at the tunnel. Third, as mentioned above, there are significant operational issues related to decisions to close an interstate highway tunnel with temporary flood gates if this adaptation option were to be chosen. The closure takes some time after the decision and thus would have to be done prior to the peak of the storm.

Step 10 – Select a Course of Action

A final design has not been selected for this case study. Selection of a final design will require further economic analysis and additional decision-making considerations be taken into account. ALDOT is still working on determining a course of action for the tunnel.

Step 11 – Plan and Conduct Ongoing Activities

Once a course of action has been decided on, ongoing monitoring activities should be conducted to assess whether the facility is performing as planned. Monitoring activities could include:

- Installing a recording tidal gage
- Establishing a log, that would record the details and dates of any climate stressor-related incident, and the performance of adaptation option pursued
- Noting updates of sea level projections such as those provided by the U.S. Army Corps of Engineers

If an adaptation option is pursued, estimates of the cost savings due to the adaptations should be calculated to track the value they are providing.

Conclusions

Several lessons can be learned from this coastal tunnel case study. First, adaptation decisions should be based upon sound science and site-specific engineering analysis. This includes selection of appropriate storm surge and wave computer models by experienced coastal engineers on the study team who know how to quantify risk (from storm surge), know the physical processes and damage mechanisms to look at, and which models will give most accurate results. Second, seemingly logical design options may not effectively achieve the primary goal; increasing the portal wall elevation just to account for storm surge alone would not have increased the level of flood protection much. Wave impacts on top of the surge are important in this coastal situation because of wave overtopping at the more exposed portal. Third, the use of the most commonly understood measure of storm strength, the Saffir-Simpson Hurricane "Category" Scale, is not particularly valuable for engineering decisions related to storm surge as there is not a one-to-one relationship between storm surge and storm "category." And finally, integrating vulnerability into decision-making will typically include some iteration or "feedback-loop" process such as the search for more effective alternative design options in this case study.

The study conducted on the Wallace Tunnel does not consider the potential impacts that future sea level rise and accompanying higher storm surge could have on the tunnel. An additional analysis of the vulnerability of the tunnel to sea level rise and accompanying storm surge is recommended to fully understand the potential risks to the tunnel. This study stemmed from an approach to modify an existing tunnel to the extent practicable and assess how it performed under storm conditions, rather than to define a design criterion and determine what would be necessary to meet that standard. For large, expensive, highly constrained projects this type of assessment might be a useful practical way to approach the problem. This can help decision makers consider a variety of questions. How much resilience can we practically build into our project? How much time or reduction of risk will a particular adaption option buy us? When does it become too expensive to modify the existing tunnel and another type of structure or route needs to be considered?

4.4.8 Shipping Pier Exposure to Storm Surge – Dock One at the McDuffie Coal Terminal

Introduction

Piers[377] are an important linkage between maritime and land-based transportation networks. Higher storm surges resulting from rising sea levels and potential increases in hurricane intensity with climate change pose a potential threat to near shore piers and other port infrastructure. This case study explores possible future storm surge impacts on a pier at the McDuffie Coal Terminal in Mobile. This particular port facility was chosen for study because of its exposure to storm surge and the economic importance of maintaining continuity of operations. According to the Alabama State Port Authority staff, the Authority receives 50% of its revenue from the McDuffie Coal Terminal.[378] Also, if the McDuffie Coal Terminal is out of service for more than 30 days the shortage of coal can result in "brown-outs"[379] in the area.[380] This case study applies the 11-step *General Process for Transportation Facility Adaptation Assessments* to one of the piers at the McDuffie Coal Terminal as an example of how owners / operators of similar facilities might evaluate and take steps to minimize climate change and extreme weather risks. The study found that the pier studied, Dock One, was not vulnerable to the storm surge scenarios tested and no adaptation options are recommended at this time.

Case Study Highlights

Purpose: To evaluate whether a shipping pier structure could be vulnerable to wave impacts from selected surge scenarios.

Approach: A methodology by Cuomo et al (2007) was used to estimate the *quasi-static* hydraulic force of a wave colliding with the pier, and whether the pier's design was sufficient for withstanding these forces.

Findings: According to the analysis, the pier's design is likely sufficient to withstand the modeled storm scenarios.

Viable Adaptation Options: No specific adaptation options evaluated since the pier does not appear vulnerable to this stressor.

Other Conclusions: Although the pier itself could withstand the modeled surges, critical equipment on and around the pier and ancillary services will need protection from any event that overtops the pier.

Application of the General Process for Transportation Facility Adaptation Assessments

Step 1 – Describe the Site Context

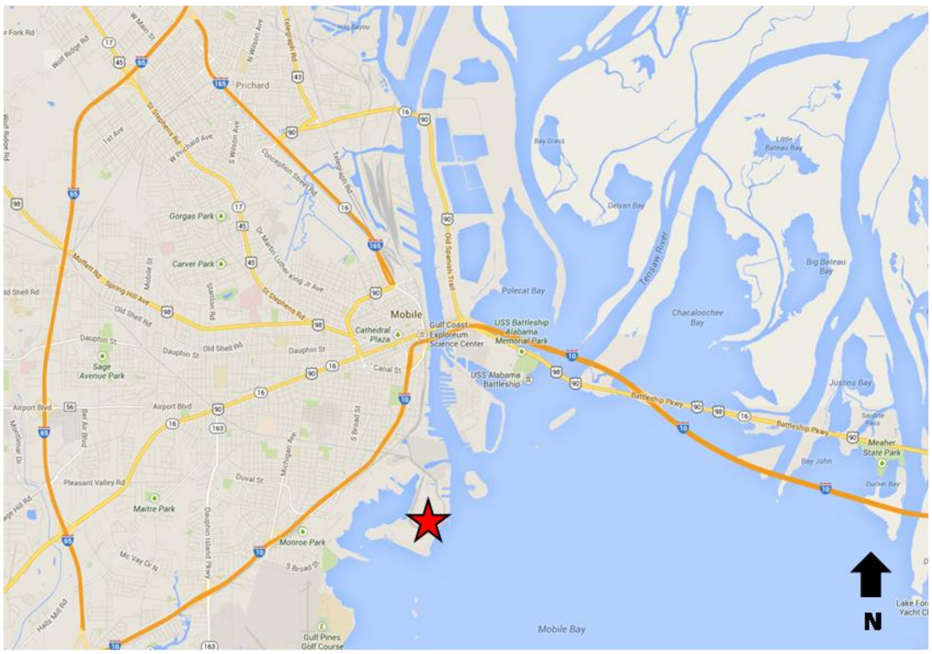
The McDuffie Coal Terminal is located on McDuffie Island, 2.5 miles (four kilometers) south of downtown Mobile (see Figure 87). The facility is one of the largest import-export coal terminals

[377] Piers, also known as docks, are elevated structures for mooring ships that connect to land and extend out into the water; they differ from wharves / quays which are also used for mooring ships but generally run parallel to the shoreline providing continuous access from the shoreline edge. The terms "pier" and "dock" are used interchangeably in practice and in this case study.
[378] Kichler, 2013
[379] A brown-out is a temporary cutback in electricity supply.
[380] Kichler, 2013

in the United States with an annual throughput capacity of 30 million tons.[381] Total import export tonnage in 2012 was 13.9 million tons.[382]

This case study will focus on one specific portion of the facility, Dock One, which is the southernmost pier at the terminal (see Figure 88). Dock One is exposed to Mobile Bay to the east and south with the main shipping channel into Mobile located just east of the berth. Dock One functions as a ship and barge loading facility for coal. Two ship loaders on the pier transfer coal from storage areas to the hulls of vessels berthed alongside the dock for distribution to domestic power plants or foreign ports.

Figure 87: Location of the McDuffie Coal Terminal[383]



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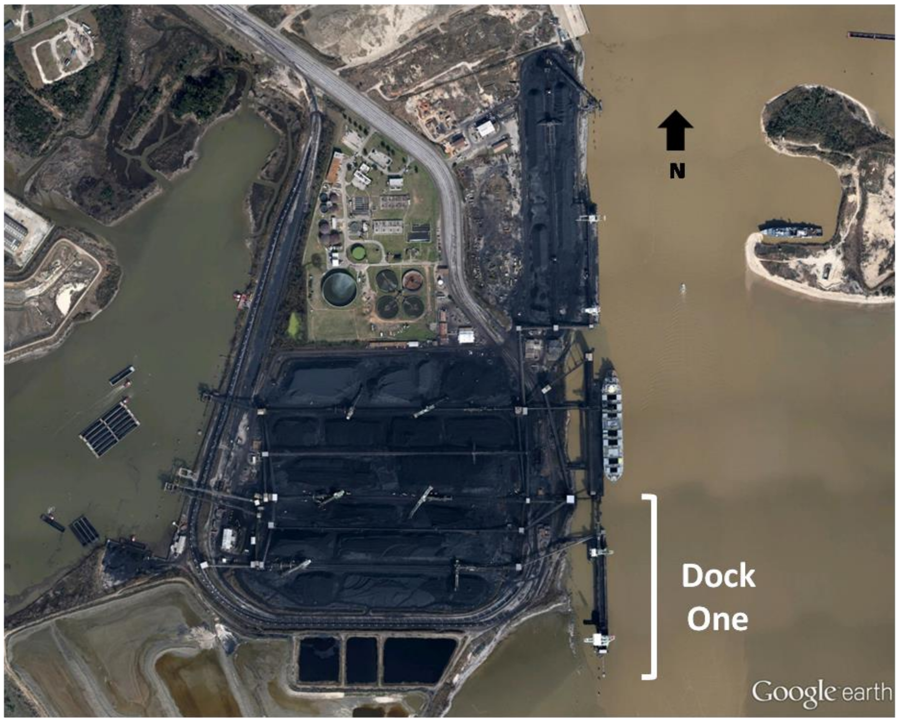
[381] ASPA, 2014
[382] ASPA, 2014
[383] Source of base map: Google Maps (as modified)

Figure 88: Aerial View of the McDuffie Coal Terminal Showing Dock One[384]



Step 2 – Describe the Existing Facility

Dock One was originally constructed in 1973 and has undergone major renovations and expansions in 1994 and 2004. The dock is generally used as a single berth for larger vessels on the waterside with capability to berth multiple barges of various sizes on both the waterside and landside of the dock.

Components of Dock One include:

- The main dock itself which is 648 feet (197.5 meters) long and 62 feet (18.9 meters) wide
- A single 16 square foot (1.5 square meter) mooring dolphin[385]
- A 148 foot (45.1 meter) long by four foot (1.2 meter) wide access walkway from the pier to the mooring dolphin
- A 240 foot (73.2 meter) long by 24 foot (7.3 meter) wide two-lane access bridge from the shore to the pier.

Figure 89 shows a plan (overhead) view of the dock with all of these features marked and Figure 90 shows an oblique aerial image of the facility.

[384] Source of aerial photo: Google Earth (as modified)
[385] A mooring dolphin is a standalone man-made structure above the water level that ships can tie up to. Dolphins typically consist of a series of vertical wood, steel, or concrete piles driven into the seabed that are tied together at the top by wire rope or a concrete or steel cap.

Figure 89: Aerial View of Dock One[386]

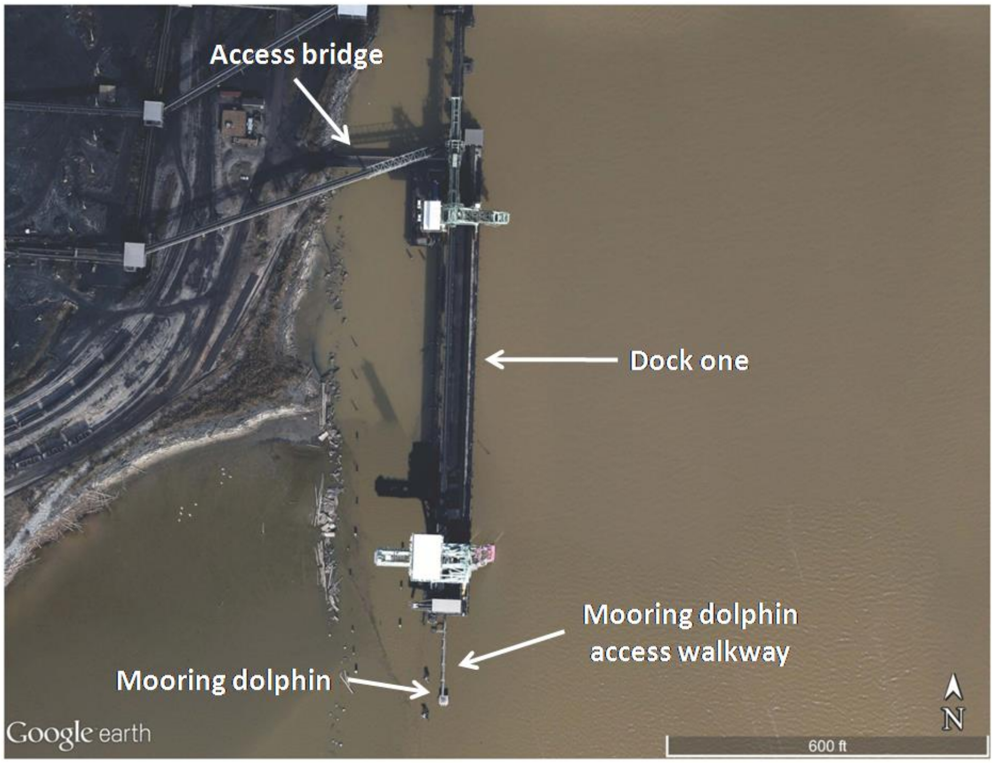
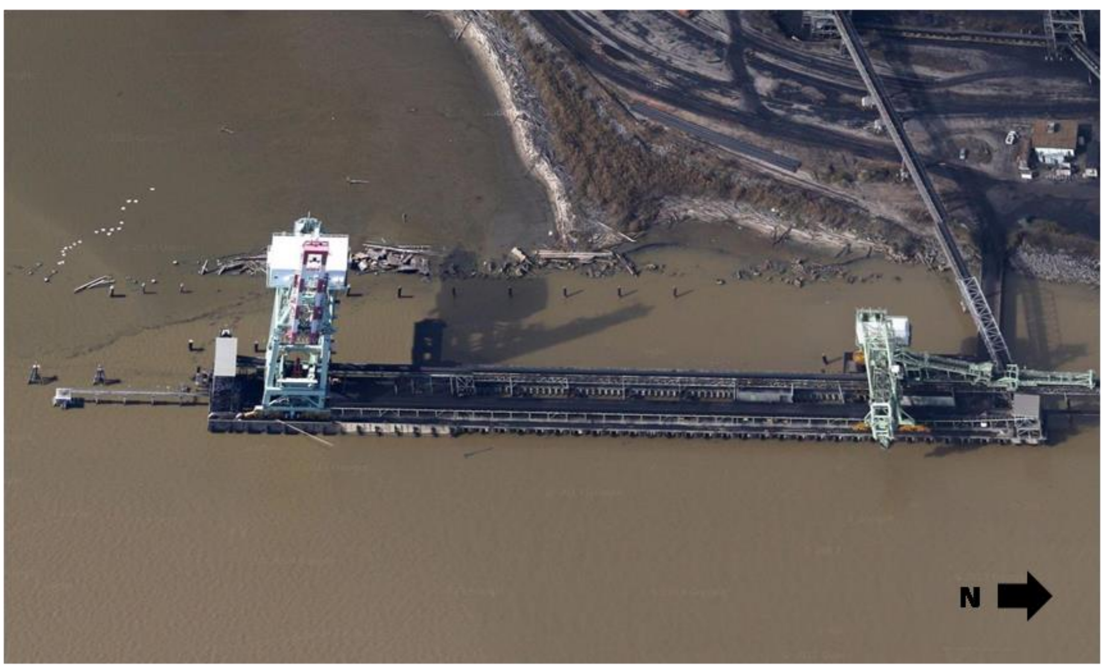


Figure 90: Oblique Aerial Image of Dock One[387]



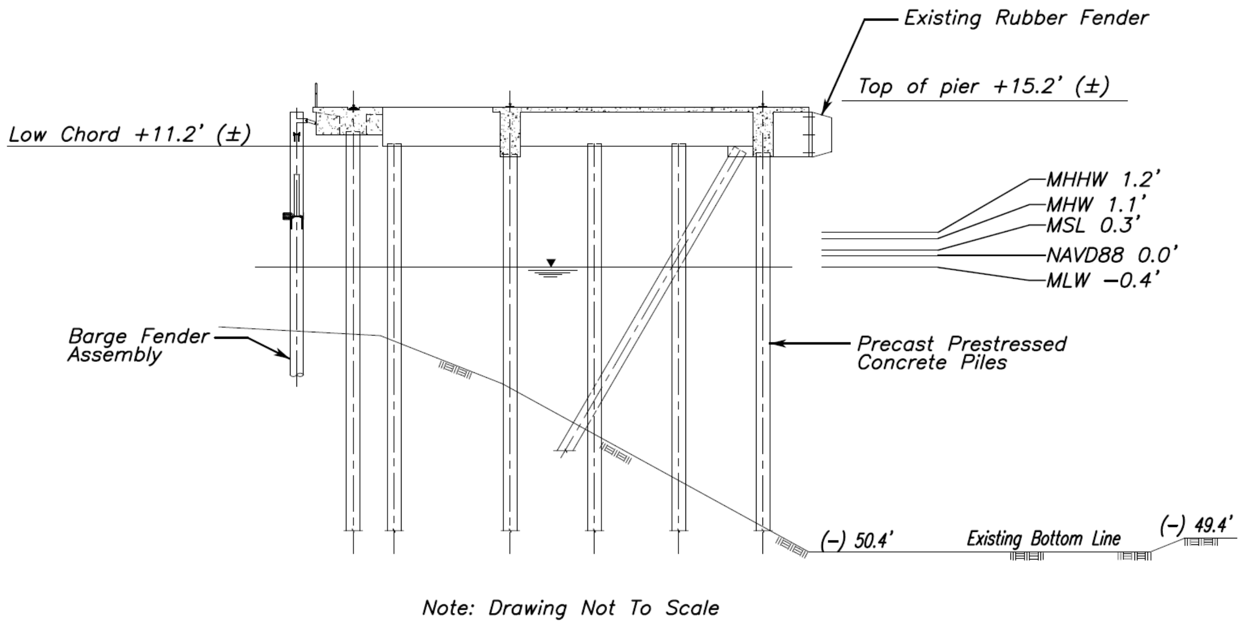
[386] Source of aerial photo: Google Earth (as modified)
[387] Source of aerial photo: Google Maps (as modified)

The dock's construction consists of a cast-in-place reinforced concrete deck supported by cast-in-place reinforced concrete beams supported by precast prestressed concrete piles.[388] The top of pier elevation is approximately 15.2 feet (4.6 meters)[389] with a low chord elevation (bottom of cap beam) at approximately 11.2 feet (3.4 meters). Mean Low Water[390] (MLW) is at -0.4 feet (-0.1 meters), Mean Sea Level[391] (MSL) is at 0.3 feet (0.1 meters), Mean High Water[392] (MHW) is at 1.1 feet (0.3 meters), and Mean Higher High Water[393] (MHHW) is at 1.2 feet (0.4 meters). Figure 91 provides a typical section for the dock showing each of these water elevations.

Industrial piers like the one at McDuffie are designed for very large loads. Dock One is designed for crane wheels loads of 27,000 pounds per linear foot (40,180.4 kilograms per linear meter) of rail, deck uniform live loads[394] of 750 pounds per square foot (3,661.9 kilograms per square meter), and concentrated loads[395] of 100,000 pounds (45,392 kilograms). Loads from ship impact berthing and mooring lines are very large as well. The mooring bollards[396] at Dock One are located at 60 foot (18.3 meter) intervals and rated for 200,000 pounds (90,718.5 kilograms) each. The fender system[397] elements designed to resist berthing loads are spaced at 20 foot (6.1 meter) centers and deliver a reaction to the dock structure of 190,000 pounds (86,182.6 kilograms). These lateral loads are unique to piers and thereby require them to have substantial lateral load force resisting systems. As a result of these loads, piers tend to be very robust in nature when compared to other structures.

[388] Piles are the vertical support members extending from the deck of the pier structure to the seabed.
[389] All elevations in this study are with respect to the North American Vertical Datum of 1988 (NAVD88).
[390] Mean low water is the average of all low tide elevations during the day over the current National Tidal Datum Epoch. The tidal epoch is the specific 19-year period over which NOAA uses to obtain observations that are used to develop tidal statistics.
[391] Mean sea level is the average of the water elevations recorded at each hour of the day over the current National Tidal Datum Epoch.
[392] Mean high water is the average of all high tide elevations during the day over the current National Tidal Datum Epoch.
[393] Mean higher high water is the average elevation of the highest daily high tide over the current National Tidal Datum Epoch.
[394] Deck uniform live loads refer to design loads of uniform force applied to a large area.
[395] Concentrated loads refer to a singular design load applied at a single location.
[396] Mooring bollards are fixtures mounted to ship berths for attaching the mooring lines and ropes used to hold ships against the berth.
[397] A fender system separates a ship from a dock structure and is used as an energy absorbing cushion.

Figure 91: Typical Section of Dock One



The mooring dolphin is cast-in-place concrete supported by precast prestressed concrete piles. The mooring dolphin has a top of deck elevation of 14 feet (4.3 meters) and a low chord elevation (the soffit / underside) of 11.2 feet (3.4 meters). The mooring dolphin access walkway is constructed of a concrete topping slab over hollow core prestressed deck slabs supported by cast-in-place reinforced concrete pile caps supported by precast prestressed concrete piles. The top of the access walkway matches the top of deck elevation of the pier at approximately 15.2 feet (4.6 meters) with a low chord elevation (bottom of cap beam) at approximately 13.2 feet (four meters).

The two-lane access bridge is a cast-in-place reinforced concrete deck slab supported by precast prestressed girders. The girders are supported by cast-in-place reinforced concrete pile caps which in turn are supported by precast prestressed concrete piles. The top of the deck elevation of the two-lane access bridge varies from 9.6 feet on the landside (2.9 meters) to 15.2 feet on the pier (4.6 meters) with a low chord elevation (bottom of cap beam) varying between 2.4 feet (0.7 meters) and eight feet (2.4 meters). At 9.6 feet (2.9 meters), the surrounding land is actually lower than the pier itself such that the access bridge ramps upward 5.6 feet (1.7 meters) to reach the deck of the pier.

The coal loading equipment located on Dock One, critical to its operation, are two ship loaders and a series of coal conveyor structures. The conveyors feed coal to the ship loaders which are run by an operator. The operator controls the loading of coal into the vessels berthed at Dock One.

It should be noted that this study focuses only on the performance of the Dock One pier structure itself: it does not examine the storm effects on the mooring dolphin, mooring dolphin access walkway, access bridge (refer to Section 4.4.6 of this document for a case study showing how to conduct an analysis of storm surge impacts on a bridge), and dock equipment and infrastructure (cranes, ship loaders, conveyors, buildings, etc.). While typically the dock structures at industrial facilities survive storm events, the same cannot always be said about the equipment and infrastructure. Additionally, the loadings caused by any possible equipment damage are also not factored into the assessment of the stresses placed on the pier. A full analysis of a port facility should consider all components of port operations including equipment, storage facilities, and access routes. This type of broad analysis, however, was beyond the scope of this case study.

When conducting climate change analyses of existing pier facilities, it is also recommended that a thorough understanding of the facility's condition be factored into the analysis. Information such as condition assessments, load ratings, and structural inspections should be consulted and, if not available, new inspections of the facility should be undertaken. Unfortunately, information on the existing condition of Dock One was not available for this case study and resources were not available to conduct new inspections. Thus, for the purposes of this study, Dock One is considered to be in "as new" condition and has been evaluated based on information found in construction drawings of the existing facility. Components that may have required repair or replacement are assumed to have been maintained in such a way that they perform as originally intended.

Step 3 – Identify Climate Stressors That May Impact Infrastructure Components

Wind, sea level rise, and storm surge are the most critical environmental variables relating to pier design that climate change might affect. This study focuses specifically on the storm surge component with wind and sea level considered to the extent they affect potential future surge elevations.

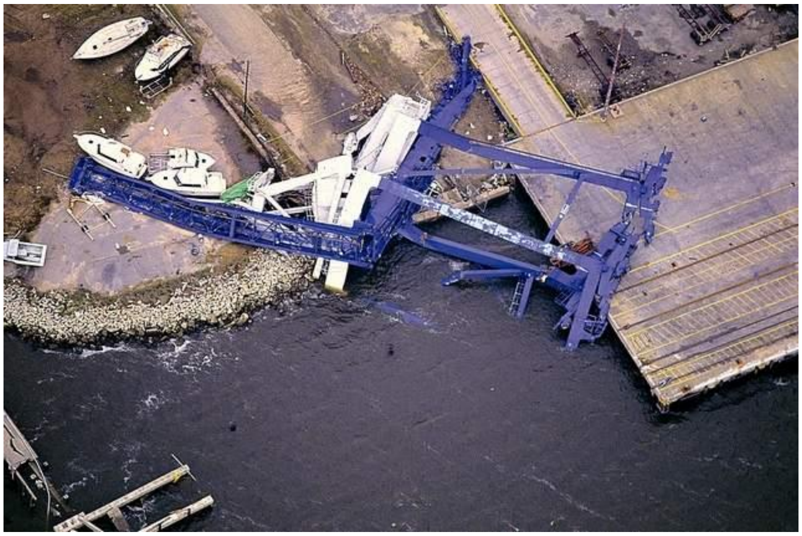
That said, it is important to note that with regards to pier design, "As a general rule, horizontal design loads on vessel berthing structures are governed by vessel berthing[398] and mooring loads,[399] or, sometimes, seismic[400]" loads.[401] "Wind, wave, and current forces acting directly on the structure usually can be neglected."[402] Near shore piers such as Dock One are first and foremost designed to perform the facility's primary function; the loading and unloading of ships. Storm surge is given consideration after the operational parameters of keeping the facility functioning through seasonal tide ranges are established. *There are currently no code requirements that impact the design of facilities such as Dock One with regards to storm surge*. The reasoning for this is that berthing and mooring loads are typically much greater than the loads caused by storm conditions and, historically, the survivability of these types of structures has been very high during storms. Such analysis should be revisited with an eye to future

[398] Berthing loads are those incurred when a vessel impacts a berth upon approach.
[399] Mooring loads are those loads applied from vessel tie lines to the mooring hardware located on the pier deck.
[400] Seismic loads occur during earthquakes.
[401] Gaythwaite, 2004
[402] Gaythwaite, 2004

changes in storm surge and sea levels. This analysis represents a first step in that direction. If it is determined that future events could make piers more vulnerable, then it would be important to consider climate effects in pier design.

Hurricane Hugo which struck Charleston, South Carolina in 1989 provides an example of the high survivability of pier structures in storms. The hurricane produced tremendous wind and storm surge damage; in fact, Hugo produced some of the highest storm tide heights ever recorded along the U.S. East Coast at the time. The damage to the Port of Charleston infrastructure was significant; however, the performance of the pier and wharf structures in the face of high wind and record surge was remarkable. Figure 92 below shows the after effects of a wind and storm surge combination strong enough to have toppled the container handling gantry crane. However, the dock structure itself and two-lane access ramp, shown at the top and right of the picture, survived intact.

Figure 92: Columbus Street Terminal in Charleston, SC, After Hurricane Hugo[403]



Closer to Mobile, The American Society of Engineers (ASCE) conducted post Hurricane Katrina assessments of ports, harbors, and marine facilities and published their findings in a book titled *Hurricane Katrina Damage Assessment: Louisiana, Alabama, and Mississippi Ports and Coasts*.[404] It contains numerous detailed descriptions of damage and overall performance throughout the region including that of piers and wharves similar to Dock One. Most all of these structures sustained very little structural damage; even those closer to where the storm made landfall. The McDuffie Island complex was investigated specifically and the minor structural damage noted in the investigation was due to several vessels being tied up alongside during the hurricane including a large bulk vessel laden with coal. Note that it is generally not an acceptable

[403] Spain, 1989
[404] Curtis, 2007

procedure to allow vessels to be moored alongside piers during storm events. It is the policy of the Alabama State Port Authority to request all ships to sail during hurricane events.[405]

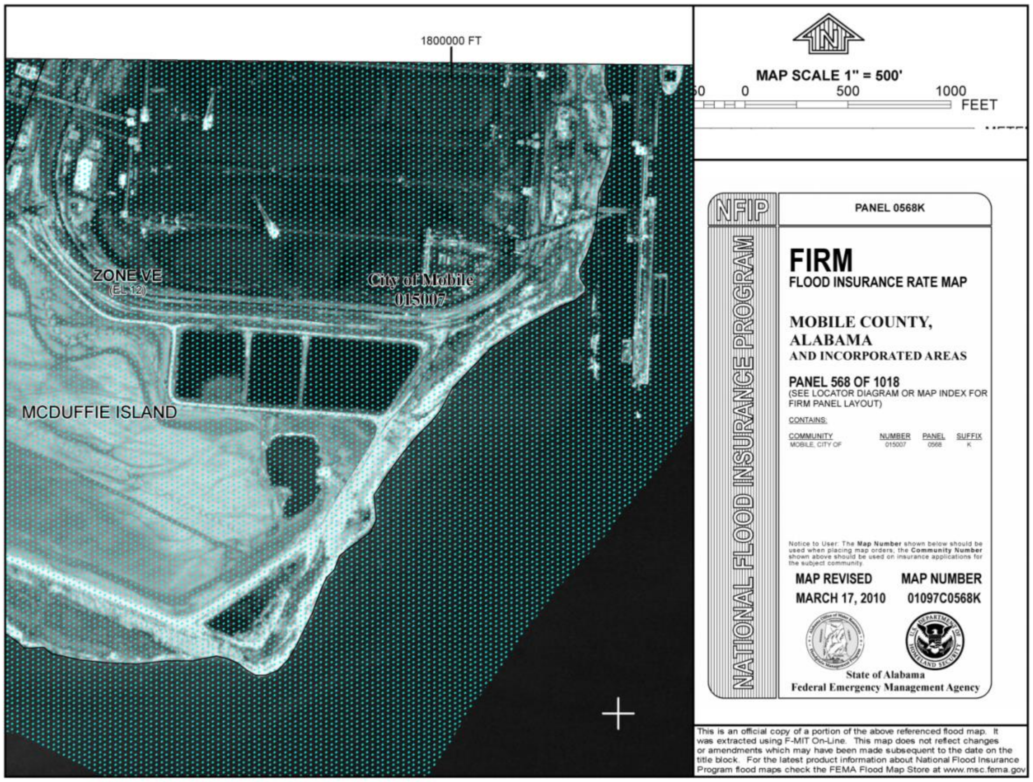
Step 4 – Decide on Climate Scenarios and Determine the Magnitude of Changes

As noted in Step 3, major industrial piers like Dock One are typically designed not for a particular return period storm but first and foremost for their functional purpose of berthing ships since such design loads tend to be controlling. Nonetheless, while it is not required by code, it is common practice for pier designers to check back to the 100-year coastal flood elevation (the one percent annual probability storm) to make sure it does not overtop the pier and jeopardize any equipment on the structure.

FEMA Flood Insurance Studies have historically been the authoritative source of this information for current conditions in coastal areas of the United States. As an example of these data sets, a portion of the Flood Insurance Rate Map (FIRM) that includes Dock One at McDuffie Terminal is shown in Figure 93. As one can see, the entire southern end of the McDuffie Terminal is expected to be inundated during a 100-year event. The base flood elevation[406] of this storm at Dock One is 12 feet (3.7 meters).[407] FEMA designates V and A zones within the coastal flood zone to delineate different hazard levels associated with the flood.[408] The V zone denotes "Coastal High Hazard Areas" with wave heights in excess of three feet whereas "Coastal A Zones" denote areas where wave heights are less than three feet. As shown in Figure 93, McDuffie Terminal lies within the coastal high hazard area (VE zone) with the "E" denoting that a detailed study was conducted and that base flood elevations and depths are available. FEMA (2014b) provides a listing and definition of all the possible zone designations that may occur within the V zone (e.g., V, VE and V1-30 zone designations) and A zones.

[405] Kichler, 2013
[406] The term "base flood" refers to the 100-year (one percent annual probability) storm. Thus, the base flood elevation is the elevation that the floodwaters are expected to reach during the 100-year storm.
[407] FEMA, 2010c
[408] See FEMA, 2014a for more information.

Figure 93: FEMA Flood Insurance Rate Map Showing the 100-Year Flood Elevation for McDuffie Terminal and Dock One[409]



In addition to FEMA, U.S. Army Corps of Engineers (2002) provides a wide range of data sources (including wind, waves, water levels and other information) useful for the design of particular facilities within ports (e.g., revetments, floating docks, and other assets that should not be inundated during floods).

As discussed in Step 4 of Section 4.4.4 , it is difficult to develop a future 100-year flood elevation that considers climate change impacts on sea levels and storm tracks, intensities, and frequencies. Given this, a scenarios approach was taken to the surge analysis whereby the track and intensity of historical storms in the region were changed along with sea levels to indicate possible future storm threats. The following three storm surge scenarios were considered for this adaptation assessment:

- **Hurricane Katrina Base Case Scenario**: This scenario represents the surge conditions that actually occurred in Mobile with Hurricane Katrina making landfall at the Louisiana-Mississippi border. The effects of Hurricane Katrina on the Mobile area were not as severe as they were at the Louisiana-Mississippi border.
- **Hurricane Katrina Shifted Scenario**: This scenario estimates the surge levels that would occur if Hurricane Katrina's path was shifted east to make landfall in Mobile.

[409] FEMA, 2010c. Note: The elevations shown are in NAVD88.

- **Hurricane Katrina Shifted + Intensified + Sea Level Rise (SLR) Scenario**: This scenario estimates the surge levels that would occur if Hurricane Katrina was shifted, intensified with stronger winds due to climate change, and came on top of 2.5 feet (0.8 meters) of sea level rise.

A more detailed description of each scenario and how it was developed can be found in Section 4.4.4 of this document (under Step 4) and in the *Climate Variability and Change in Mobile, Alabama* report.[410]

Table 51 shows the storm surge elevations under each of the three scenarios developed along with the current FEMA base flood elevation. The model results shown in Table 51 were obtained from the models at the south end of Dock One offshore in the navigation channel. The model domain resolution did not allow precise replication of the deep berths at the McDuffie Terminal and, as a result, the depths in the model near the dock are somewhat less than actual conditions. As such, the current was somewhat less so the model results were taken slightly to the east of the dock where the modeled depths were the same as actual conditions. Although the currents are relatively moderate, they do increase significantly to the north along McDuffie Terminal with the maximum modeled current, 5.9 knots (10.9 kilometers per hour), occurring at the north end of the McDuffie Terminal for the Hurricane Katrina Base Case Scenario.

Table 51: Storm Surge Analysis Results for Dock One at McDuffie Terminal[411]

Storm Surge Scenario	ADCIRC Hydrodynamic Model Results			STWAVE Wave Model Results		
	Water Surface Elevation Feet (m)	Sustained Wind Speed mph (kph)	Depth Averaged Current Knots (kph)	Wave Height[412] Feet (m)	Peak Wave Period (sec)	Wave Direction[413] (Compass Degrees)
Hurricane Katrina Base Case	12.4 (3.8)	75 (120.7)	2.9 (5.4)	5.6 (1.7)	7.1	356
Hurricane Katrina Shifted	19.7 (6.0)	106 (170.6)	2.9 (5.4)	6.2 (1.9)	7.7	354
Hurricane Katrina Shifted + Intensified + SLR	24.6 (7.5)	112 (180.2)	2.3 (4.3)	4.0 (1.2)	7.6	1

Step 5 – Assess Performance of the Existing Facility

As an interconnected and interdependent system, the performance of the existing pier is a sum total of the performance of its various components. This study focuses on assessing the

[410] USDOT, 2012
[411] The FEMA base flood elevation is 12 feet (3.7 meters) (FEMA, 2010c).
[412] This value represents the zeroth moment wave height as reported by the STWAVE Model and in deep water is equivalent to the more commonly used term “significant wave height” which is the average of the highest one third of waves in a random wave field
[413] Wave direction values refer to compass directions. For example, waves at zero degrees are traveling north and waves at 270 degrees are traveling west.

survivability and performance of the Dock One main pier under each of the three storm surge scenarios.

The main pier itself has significant mass and strength which aid in resisting both wave uplift and lateral loads. Additionally, the longitudinal (long axis) orientation of the pier is coincident with the fetch[414] and predicted wave direction. That is to say that the pier is not broadside to the prevailing wave front but rather the narrow end of the pier with the smallest profile faces the approaching waves. This minimizes both wave and current influence area on the structure since the surface area of pier elements exposed to wave load is minimal. For these reasons, it is expected that the pier will survive most storm events.

To confirm these expectations, validate the observations of the actual Katrina event, and provide an example of how one might quantify a pier's vulnerability to surge, an attempt was made to determine actual loads on the pier and compare them to its design capacity. This was done by conducting a strength analysis of the Dock One pier structure and comparing the results to surge-related loads derived from a European study by Cuomo et al. (2007) titled *Wave-in-Deck loads on Exposed Jetties*.

During a major storm event, three main forces should be considered for analysis of a pier:

- **The dead weight of the structure itself**
- **The buoyancy of the pier as a function of the water height:** As the wave rises, the buoyancy force on the structure is a combination of the water displaced by the structure as well as the water displaced by entrapped air beneath the structure
- **The hydraulic force of the wave / water colliding with the pier:** This includes the following three primary hydraulic loadings:
  - ***Impulsive*:** The impulsive loading is the initial impact of the wave on the pier in which the pier experiences the highest force over the shortest duration
  - ***Quasi-static*:** The quasi-static loading is longer and consists of the wave's pulsing action in addition to the buoyancy force of the sea water
  - ***Suction*:** After a wave passes, the water recedes creating a suction (downward) force on the deck

For the analysis of Dock One, the methodology in the report written by Cuomo et al (2007) was implemented to determine the quasi-static loads applied to the pier from the wave. Impulsive and suction loadings were not considered in this analysis.

With regards to impulsive loads, the report states that, "It must be stressed that impulsive loads measured during physical model tests have relatively short rise times…that might fall within the range of the natural periods of vibration of the prototype structures."[415] Similarly, it goes on to

[414] Fetch refers to the area over water where the wind is unobstructed with fairly uniform speed and direction.
[415] Cuomo, Tirindelli, and Allsop, 2007

state that, "when significant impulsive loads are expected to act on the suspended deck structure, the evaluation of the impulsive load to be used in design analysis must account for the dynamics of the prototype structure."[416]

It is estimated that the wave impact to the Dock One structure has a duration of 0.01 to 0.1 seconds for a storm surge of this magnitude. In order to assess the capability of a particular structure to resist high impact short duration loads, structural computer modeling and analysis is required to determine dynamic response of the structure and load distribution. Structural modeling could perhaps account for the inertia, dynamic response, and deflection / displacement of the dock when exposed to the energy of an impulsive load. This analysis is, however, well beyond the scope for this study. Additionally, the primary focus and main results of the paper are on the determination of the quasi-static loading. For these reasons, the impulsive load was not considered. The significance of not considering impulsive load on the pier is difficult to determine because it is not apparent how the load affects the structure. Impulsive loads are extremely large, two and three times that of static loads, however their short duration and limited contact area reduce their influence on required design strength.

With regards to the suction load, it is very small relative to the quasi-static load and acts in the same direction (downward) as the design loads. For these reasons, suction load is not considered in this analysis. A full assessment of an actual pier project should consider both impulsive and suction loads in the analysis.

The Cuomo et al. (2007) formulas for calculating the quasi-static wave force were derived from physical model tests on a 1:25 scaled model of a pier similar in construction to Dock One fitted with strain gauges[417] at different locations. Due to the empirical nature of the formula, it is assumed that it incorporates all the hydraulic forces (quasi-static, buoyancy, etc.) acting on the pier during a wave event. For this reason, additional buoyancy was not added to the upward force calculated from the Cuomo et al. (2007) equation used for determining load on the members of Dock One. Likewise, it could be assumed that the empirical data gathered during testing would have included the dead weight of the scaled model acting in the opposite direction of the quasi-static load. For this reason, a conservative approach was taken by not using the dead weight of the structure to resist wave uplift loads.

Hurricane Katrina Base Case Scenario

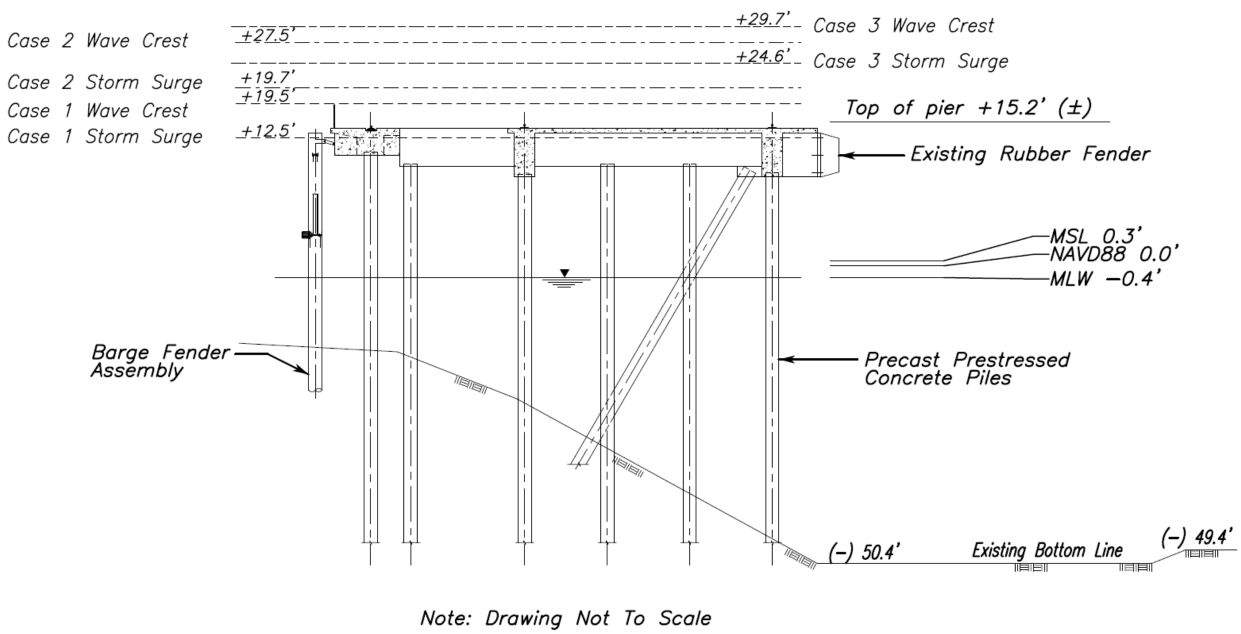
The Hurricane Katrina Base Case Scenario results in a storm surge elevation of 12.5 feet (3.8 meters) at Dock One; below the top of pier elevation of approximately 15.2 feet (4.6 meters) and above low chord elevation (bottom of cap beam) of approximately 11.2 feet (3.4 meters). The storm also entails an average significant wave height crest elevation of 19.5 feet (5.9 meters);

[416] Cuomo et al., 2007
[417] A strain gauge is a device used to measure strain on an object. The results are used to determine how that object is performing under applied loads.

high enough to overtop the pier, mooring dolphin, and mooring dolphin access walkway. Figure 94 illustrates the surge and wave crest elevations for this scenario and the other two surge scenarios.

Waves during the Hurricane Katrina Base Case Scenario break above all parts of the structure as well as creating uplift forces from the underside of the pier deck. This is the scenario that potentially causes the most damage to a structure of this type because the other two scenarios (the Hurricane Katrina Shifted and Hurricane Katrina Shifted + Intensified + SLR Scenarios) would cause the pier to be inundated by the surge, resulting in less force being applied to the structure from waves. Should a condition like this occur over a prolonged period of time, the cyclical nature of wave loading might lead to structural fatigue potentially having a damaging impact on key elements of the pier. Analysis of fatigue may be worth considering on an actual project, however, such an analysis was well beyond the scope of this study.

Figure 94: Typical Section at Dock One Showing Storm Surge and Wave Crest Elevations for the Surge Scenarios[418]



As noted in Step 2, the main section of Dock One at the McDuffie Coal Terminal is a cast-in-place reinforced concrete structure with three main longitudinal beams and corresponding transverse beams supported by precast prestressed concrete piles. The pile supported transverse beam is referred to as the cap beam. Two of the three longitudinal beams support the rails for the crane. These are referred to as the waterside and landside rail beams. The third longitudinal beam

[418] Case 1 refers to the Hurricane Katrina Base Case Scenario, Case 2 is the Hurricane Katrina Shifted Scenario, and Case 3 is the Hurricane Katrina Shifted + Intensified + SLR Scenario.

is the landside fascia beam.[419] The deck between the rail beams is referred to as the rail bay deck and has a thickness of 18 inches (45.7 centimeters). The exterior bay (between the rail and west fascia beam) is referred to as the landside bay deck and is 14 inches (35.6 centimeters) thick.

Each of these pier elements was analyzed individually using the Cuomo et al (2007) methodology in an attempt to determine its structural strength (shear[420] and moment capacity [421]) during the Hurricane Katrina Base Case Scenario.

For this analysis, quasi-static loads were applied to each element of the pier. A load factor[422] of 1.75[423] was applied to the quasi-static loads. Table 52 shows the factored quasi-static forces, the resulting factored moment and factored shear forces, and the moment and shear capacity of each element. Comparison of the results reveals that the factored moment and shear forces produced by the quasi-static load do not exceed the moment and shear capacities of any of the individual pier elements. Therefore, the pier is able to withstand the force from a wave for the Hurricane Katrina Base Case Scenario.

Hurricane Katrina Shifted Scenario

The Hurricane Katrina Shifted Scenario produces a storm surge elevation of 19.7 feet (six meters) at Dock One and an average significant wave height crest elevation of 27.5 feet (8.4 meters). With Dock One having a top deck elevation of about15.2 feet (4.6 meters), this scenario would put about 4.5 feet (1.4 meters) of storm surge over the top of the pier deck (see Figure 94).

The effects of this scenario before it overtakes the pier would be similar to the Hurricane Katrina Base Case Scenario before it overtakes the pier. Once submerged, the structure itself is protected from the environmental loads occurring above the storm surge elevation. The key is the ability of the structure to withstand the wave load on the superstructure through the transition from above water to below storm surge. The duration of this transition also has an effect. A surge that comes in quickly has much less impact than one that is prolonged thereby providing extended exposure to wave action. Quantifying the duration of exposure required to fail an element is a very subjective and dynamic problem that may need to be considered on some studies. However, for this analysis, given that the pier is likely to survive the longer duration wave exposure of the Hurricane Katrina Base Case Scenario, such an analysis is not necessary.

[419] A fascia beam is a beam at the face or perimeter of a structure.
[420] Shear capacity is the strength of a material or component against the type of yield or structural failure where the material or component fails through shearing.
[421] Moment Capacity or flexural strength represents the highest stress experienced within a material under bending at its moment of rupture.
[422] Load factors may be thought of as safety factors that are applied to the loads.
[423] A factor of 1.75 is recommended for wave loads in *Guide Specifications for Bridges Vulnerable to Coastal Storms* (AASHTO, 2008).

Table 52: Dock One Strength Analysis Results

Pier Element	Quasi-Static Load kips/ft (kN/m)	Moment		Shear	
		Factored Moment kips·ft (kN·m)	Moment Capacity kips·ft (kN·m)	Factored Shear kips (kilonewtons)	Shear Capacity kips (kilonewtons)
Landside Bay Deck	0.7 (10.2)	15.9 (21.5)	49.3 (66.8)	4.7 (20.9)	13.0 (57.9)
Rail Bay Deck	0.7 (10.2)	23.7 (32.1)	70.7 (95.9)	5.7 (25.6)	17.6 (78.5)
Waterside Rail Beam	1.8 (26.1)	4.9 (6.6)	959.6 (1,301.1)	4.2 (18.6)	254.7 (1,132.9)
Landside Rail Beam	1.8 (26.1)	4.9 (6.6)	959.6 (1,301.1)	4.2 (18.6)	254.7 (1,132.9)
Landside Fascia Beam	1.8 (26.1)	60.9 (82.6)	1,183.3 (1,604.4)	14.8 (65.7)	305.0 (1,356.9)
Cap Beam	1.8 (26.1)	40.8 (55.3)	1,934.3 (2,622.6)	12.1 (53.7)	305.0 (1,356.9)

Thus, given available data, the pier is likely to survive the Hurricane Katrina Shifted Scenario. Although it was not part of the analysis, it is likely that the equipment, machinery, buildings, etc. that are mounted to the pier deck will be extremely vulnerable to the storm surge in this scenario. Debris, saltwater, and wave impacts could possibly take these items off line.

Hurricane Katrina Shifted + Intensified + SLR Scenario

The third surge scenario, Hurricane Katrina Shifted + Intensified + SLR, produces a storm surge elevation of 24.6 feet (7.5 meters) and an average significant wave height crest elevation of 29.7 feet (9.1 meters). In terms of storm surge, with Dock One having a top deck elevation of about 15.2 feet (4.6 meters), this scenario would put over nine feet (2.7 meters) of surge over the top of the pier deck (see Figure 94).

As previously stated, the Hurricane Katrina Base Case Scenario is viewed as the worst case due to the fact that it has waves breaking on the pier. The effects of that scenario would be similar to the Hurricane Katrina Shifted Scenario before it overtakes the pier. Once submerged, the structure itself is protected from the environmental loads occurring above the storm surge elevation. Thus, given available data, the pier is expected to survive the Hurricane Katrina Shifted + Intensified + SLR Scenario. However, as with the previous scenario, additional consideration should be given to the vulnerability of the equipment on the pier deck to the surge.

Step 6 – Identify Adaptation Option(s)

As noted in Step 5, the various pier components studied are expected to adequately survive all three storm surge scenarios, including wave forces on the structure and uplift forces beneath the structure. Thus, no adaptive design options are required for the pier components analyzed. Although not studied here, additional consideration should be given to ways to best protect the vulnerable equipment investment on the pier so that after the storm passes there is minimal downtime required to get back online. The access bridge and mooring dolphin access walkway may reasonably be presumed to be the most vulnerable of the facility components. Consideration may be given to strengthening these elements or perhaps making them easily removable so that they may be properly stowed before the onset of a storm event.

Step 7 – Assess Performance of the Adaptation Option(s)

If adaptive actions were needed for the pier components analyzed, this step would entail assessing the performance of each adaptive action against each of the surge scenarios.

Step 8 – Conduct an Economic Analysis

If adaptation options were required for the pier components analyzed, an economic analysis should be conducted to determine each adaptation option's cost-effectiveness under each of the surge scenarios. See Section 4.4.1 for an example of how an economic analysis was applied to a culvert exposed to changes in precipitation due to climate change.

Step 9 – Evaluate Additional Decision-Making Considerations

As there are no adaptation actions required for the pier components analyzed, no additional decision-making considerations are applicable. If adaptation was required, this step might entail consideration of broader project sustainability, project feasibility, practicality, ongoing maintenance needs, funding availability, and, very importantly for the pier, stakeholders' tolerance for risk and service interruption. The latter is a key consideration when interpreting the results of any economic analysis conducted for Step 8. Also, as noted previously, the possibility of debris impacting the structure should be a consideration and, in cases where there is a high potential for large damaging debris, this might be a factor in choosing a stronger adaptation option.

Step 10 – Select a Course of Action

The recommended course of action is to take no adaptive actions to the pier at this time.

Step 11 – Plan and Conduct Ongoing Activities

For Dock One and similar facilities, owners should establish regular structural inspection intervals in order to maintain "as-new" condition. The ability of a structure to function fully as intended will go a long way in resisting the occasional overload, load reversal, or extreme environmental load. Intervals of one, three, or five years at most are standard throughout the industry with specific inspections occurring as needed, usually after a specific event. These regular inspections would also provide the opportunity to inspect the connections of access bridges and walkways and determine if they are capable of withstanding storm surge and wave loading.

Conclusions

Dock One at the McDuffie Coal Terminal was analyzed against current climate and three potential surge scenarios to determine the performance and survivability of the dock against environmental loads. Dock One had survived Hurricane Katrina with no damage and it was determined that the likelihood of this structure to perform well under all three climate scenarios is very good. The continued policy that no ships remained moored alongside the pier is critical to limit or eliminate damage to the pier during a storm event as well as limit or eliminate the potential of the ships themselves or pier appurtenances from becoming large damaging debris.

The key take away from this case study is that industrial piers like Dock One are designed for very large loads and tend to be very robust in nature when compared to other structures. Historically the survivability of these structures is very high and, given the general dismissal of storm environmental loads on the structure itself due to their relative insignificance compared to the operational design loads, in depth analysis has seldom been warranted in traditional practice. This might need to be reevaluated with the possibility of stronger storms associated with climate change in at least some circumstances. On the other hand, dock equipment and infrastructure (cranes, buildings, etc.) will become increasingly vulnerable to higher surge levels as climate changes: a pier structure that survives serves little purpose if the cranes necessitating its existence are damaged and off line. General sea level rise with climate change may also present a challenge to continued operations at these facilities. Therefore, port authorities should focus their resiliency attention on things like equipment and buildings that are not so heavily built and have been seen to suffer damage in storms.

Historical survivability of pier structures is also the primary reason that design guidance for storm surge loads has not been fully developed. While loads were extrapolated from the Cuomo et al. (2007) research document in order to develop some type of comparative analysis, the correctness and applicability to the Dock One pier can be challenged due to the pure empirical nature of the testing scenarios used in developing the load equations. Additional research and testing is necessary in order to establish a procedure for both load development and structural analysis that can be adopted universally for structures of this type with varying configurations. The culmination of this effort would result in a credible design guide that would be made available as a resource to pier designers. The limited guidance available should continue to be vetted by comparing the theoretical results with actual events.

4.4.9 Pavement Mix Design Exposure to Temperature Changes

Introduction

Roadway pavement, particularly newly installed pavement, can be sensitive to increased temperatures and loads causing distortions[424] that turn into "ruts" or crack when various forces combine. Pavement rutting[425] and cracks can slow traffic and freight movement, damage vehicles, and potentially affect vehicle control in some cases.

To date, relatively little research has been completed to investigate the potential impacts of climate change on pavement infrastructure in the United States. This is true despite the dependence of many states' economic and social activity on roadway infrastructure. A review of pavement engineering practices, models, and approaches to monitor, assess, and predict pavement performance reveals that climate, and therefore climate change, is an important consideration in at least two deterioration processes: rutting in Asphalt Concrete (AC)[426] pavements and cracking in Portland Cement Concrete (PCC) pavements.

> **Case Study Highlights**
>
> **Purpose:** Evaluate potential impacts to pavement due to projected increases in temperature.
>
> **Approach:** The asphalt concrete pavement mix currently used in Mobile was evaluated against the projected future temperatures by converting the ambient temperatures to pavement temperature.
>
> **Findings:** The current pavement binders used in Mobile are sufficient for the projected temperatures analyzed. However, the current pavement mix does come close to being vulnerable under the more severe projections analyzed.
>
> **Viable Adaptation Options (in other areas that could be vulnerable):**
>
> - For AC pavement: Use different or thicker pavement
> - For PCC pavement: Change the frequency or type of maintenance, or installation methods
>
> **Other Conclusions:** It may be beneficial to use either projected temperatures or updated historical temperatures when selecting pavement binders, rather than relying on historical temperature records.

As with other infrastructure, the fundamental concern related to climate change in pavement infrastructure is the potential for premature design failure. Current and past designs have generally assumed a static climate whose variability can be adequately determined from records of weather conditions that normally span less than 30 years and often less than 10 years. The possibility of climate change challenges this assumption and raises the prospects that the frequency, duration, or severity of both rutting and cracking may be altered which could lead to premature deterioration. This is the case because most AC pavements are designed for a 20 year design life; long enough to potentially be subjected to changing climate conditions. PCC pavements have an even longer lifespan, upwards of 40 years, which is long enough to be subjected to significant changing climate conditions. Given these concerns, this analysis considers how pavement mix designs will need to evolve over the course of the 21st century.

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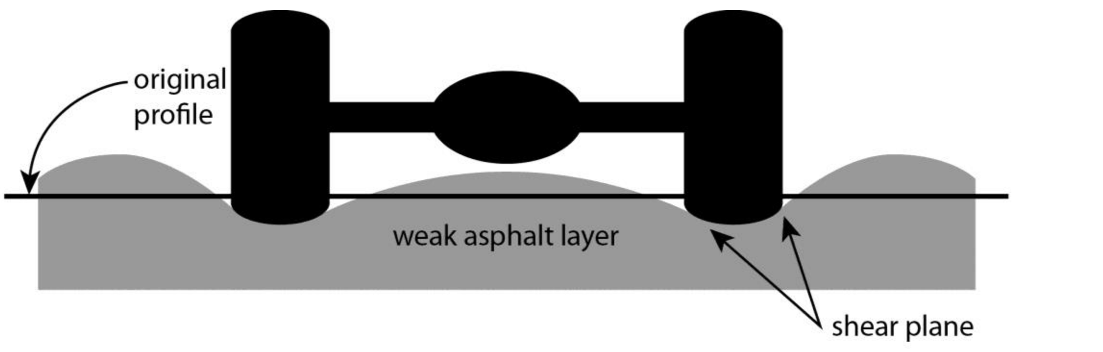
[424] Distortion is defined as that distress in the pavement caused by densification, consolidation, swelling, heave, creep, or slipping of the surface or foundation.
[425] Rutting is defined as longitudinal depressions in the wheel paths of asphalt pavements.
[426] Asphalt Concrete is the term given pavement comprised of a mixture of asphalt , aggregate, and other admixtures as may be required.

In this case study, the current practices and specifications of ALDOT were evaluated and an analysis of how climate change might affect mix designs[427] in the future was conducted. While climate change can affect pavement design in many ways,[428] the focus of this case study was limited to mix design and how that may need to change to prevent premature design failures for AC pavements from rutting and PCC pavement from cracking. It was found that no changes to mix design are required at this time in Mobile. The following sub-sections provide a brief overview of AC rutting and PCC cracking.

AC Rutting

Rutting is a distortion occurring in the wheel paths of an AC pavement (see Figure 95). It results from densification[429] and permanent deformation[430] under vehicle loads, combined with displacement of pavement materials, and affects the functional performance of a pavement. It is also a primary indicator of the structural performance of pavement. In deterioration models,[431] rutting is normally expressed as a depression depth relative to the plane of the pavement surface.

Figure 95: Diagram of Rutting in an AC Layer[432]



Rutting may be caused by several factors, including unstable AC mixes resulting from high temperatures, high asphalt content, or low binder viscosity.[433] Rutting is a common form of distress where heavy traffic loads such as heavily loaded trucks coincide with high in-service temperatures. As asphalt temperatures increase, the stiffness of the asphalt decreases, making it more prone to deformation under wheel loads.

[427] Mix design refers to the various components of a pavement. For asphalt, there are three primary components: (1) asphalt binder (a viscous petroleum-based product that essentially acts as the glue that holds the asphalt together), (2) mineral aggregate, and (3) air. Optionally, additional modifiers and additives can also be included. Of the three primary components, mineral aggregate makes up the vast majority of the mix with air and binder comprising the remainder. For concrete, the key mix components are Portland cement (acts as the binder), aggregate, water, and mineral and chemical admixtures (used to achieve higher quality concrete and / or better workability).
[428] Beyond mix design, temperature can also affect the construction and maintenance regimes for paving work. These impacts are not covered in this case study but are touched on in Section 4.4.1 . Climate change can also have an impact on pavements through changing moisture regimes. Moisture impacts on pavements and subgrades are not addressed in this case study although rapidly changing soil moisture conditions or periods of extended inundation can cause significant damage to pavements.

[429] Densification is compaction: an increase in the density of something.
[430] Permanent deformation occurs when a component does not return to its original shape after being strained.
[431] A deterioration model is a mathematical model used to predict pavement deterioration over time.
[432] Santucci, 2001
[433] Viscosity is the state of being thick, sticky, and semi-fluid in consistency, due to internal friction.

PCC Cracking

If PCC reaches too high a temperature during its placement, long-term PCC performance might be compromised. High temperatures increase the rate of hydration,[434] permeability[435] and thermal stresses,[436] and raise the chances of drying shrinkage cracking.[437] Cracking leads to decreased long-term PCC strength and durability. Most states, including Alabama, specify a maximum PCC temperature at the time of placement to mitigate the detrimental effects of hot weather. Mineral or chemical admixtures[438] can also be used in the PCC mix to help mitigate the impacts of high PCC temperatures.

It is important to understand the significance of the crack pattern in terms of the performance of PCC pavement with respect to the potential for distress development. Distress patterns can occur in one of two forms. One form is associated with wide transverse cracks that often occur with wide crack spacings or clustered crack patterns. The second form of distress is the loss of load transfer on adjacent transverse cracks leading to the development of a punchout.[439] The focus of identified failure modes of the punchout process is consequently closely aligned with the load transfer, crack width, and the effective slab bending stiffness of adjacent transverse cracks. Detailed field and laboratory study has clearly indicated that punchouts are initiated as a result of lost or reduced pavement support. Lost or reduced pavement support, though not directly related to PCC mix design, can be correlated with the formation of crack pattern development. Crack pattern development can be correlated to the temperature of the mix during placement which can be mitigated by adjustments to the mix design.

Organization of Case Study

The case study is organized around the 11 steps of the *General Process for Transportation Facility Adaptation Assessments* to illustrate how they can be applied to the topic of pavement mix design. The focus is on AC mix design but issues associated with PCC mix design are also discussed. The reason this case study focuses on AC mix design versus PCC mix design is that a majority of roadways / highways in Alabama are AC versus PCC.

Application of the General Process for Transportation Facility Adaptation Assessments

Step 1 – Describe the Site Context

There is no specific project location for this case study; instead the analysis is broadly applicable to any future highway paving project in the Mobile region where ALDOT practices for pavements would apply.

434 Hydration is the process whereby Portland cement (mixed with aggregates such as sand and gravel) reacts with water to produce concrete (and heat).
435 Permeability is the state or quality of a material or membrane that causes it to allow liquids or gases to pass through it.
436 Thermal stress is a decrease in the quality of a material that occurs due to excessive changes in temperature.
437 Drying shrinkage cracking occurs when the concrete shrinks due to evaporation of excess water but the subgrade (materials below the concrete) and internal reinforcement restrain the concrete, causing stresses and cracking in the concrete slab.
438 Admixtures are added to a concrete mix to improve quality and aid in the construction process.
439 Punchouts are localized areas where the slab is cracked and broken into several pieces.

Step 2 – Describe the Proposed Facility

The proposed new facility is a generic high truck volume highway in the Mobile region, but the analysis could also be applied to low volume roads in order to achieve better pavement performance. An example of this type of facility in the Mobile region for AC pavement would be I-10 near McDuffie Terminal. An example for PCC pavement would be North Broad Street in Mobile (US 98/Alternate US90). Higher truck volume highway facilities would be more at risk for climate change impacts based on the fact that rutting is a common form of distress where heavy traffic loads (such as occur with heavily loaded trucks as measured by the number of Equivalent Single Axle Loads[440]) coincide with high in-service temperatures.

Step 3 – Identify Climate Stressors That May Impact Infrastructure Components

The key environmental factor to affect pavement mix design, both in terms of AC rutting and PCC cracking, is temperature. The specific temperature variables relevant to pavement mix design include the:

- Maximum seven consecutive day average high air temperature (50 % reliability[441])
- Absolute minimum low air temperature on the coldest day (50 % reliability)

For AC mix design, the main materials in consideration are asphalt, binders and aggregates: the temperature variables listed above are used to assist in the selection of these materials. ALDOT uses a Superpave[442] system to help with the selection process.

For binders, the concept of Performance Grading (PG) is based on the idea that a hot mix AC binder's properties should be related to the conditions under which it is used. For AC binders, this involves expected climatic conditions as well as aging considerations. The PG system uses a common battery of tests and specifies that a particular AC binder must pass these tests at specific temperatures that are dependent upon the climatic conditions in the area of use. A binder used in the Sonoran Desert of California and Arizona, for instance, would have different properties than one used in the Alaskan tundra.

A suitable PG AC binder will minimize thermal cracking under cold temperatures (due to shrinkage of the material) while simultaneously minimizing traffic-induced rutting under hot temperatures. Pavement designs have multiple AC layers usually of varying asphalt grades. There is usually a surface, intermediate, and base course layer. Each layer would need to be reviewed for appropriate PG grade adjustment due to temperature within a given region: this assessment considers only the surface layer.

[440] ESAL is the loading equivalent of one, 18,000 pound (80 kilonewton) axle.
[441] Reliability refers to the probability that the given temperature value will be exceeded in any particular year.
[442] The U.S. Strategic Highway Research Program developed the Superpave system to specify optimal hot asphalt pavement mixes for given temperature and traffic conditions based on empirical research.

Grades are assigned in 10.8°F (6°C) increments for both minimum and maximum pavement temperatures as illustrated in Table 53. The naming of each binder specification corresponds with the metric pavement temperature ranges for which it is rated. For example, a PG 58-22 AC binder meets a seven-day maximum pavement temperature of 58°C (136.4ºF) and a minimum pavement temperature requirement of -22°C (-7.6°F). The maximum PG threshold refers to a temperature within the pavement, normally about 0.8 inches (20 millimeters) from the surface, while the minimum PG threshold refers to the actual surface pavement temperature. In practice, maximum temperature PG thresholds are adjusted upward one or more 10.8°F (6°C) increments to account for traffic and load considerations. Note that binder specifications are defined in terms of *pavement* temperature, not ambient temperature.

The conversion of ambient temperature to maximum pavement temperatures for use in selecting PG grade asphalts can be accomplished using the following formula: [443]

$$T_{20mm} = (T_{Air} - [0.00618][lat]^2 + [0.2289][lat] + 42.2°C)(0.9545) - 17.78°C$$

Where,
T_{20mm} = High pavement design temperature 0.8 inches (20 millimeters) below the surface
T_{Air} = Maximum seven consecutive day average high air temperature (°C)
lat = Geographical latitude of the site in degrees

Likewise, the conversion of ambient temperatures to minimum pavement temperatures can be done through the following formula:

$$T_{Min} = (0.859)(T_{Air}) + 1.7°C$$

Where,
T_{Min} = Minimum pavement design temperature
T_{Air} = Absolute minimum low air temperature on the coldest day

[443] Fwa, 2005 (pg. 7-27)

Table 53: Performance Grade AC Binder Specifications by Temperature[444]

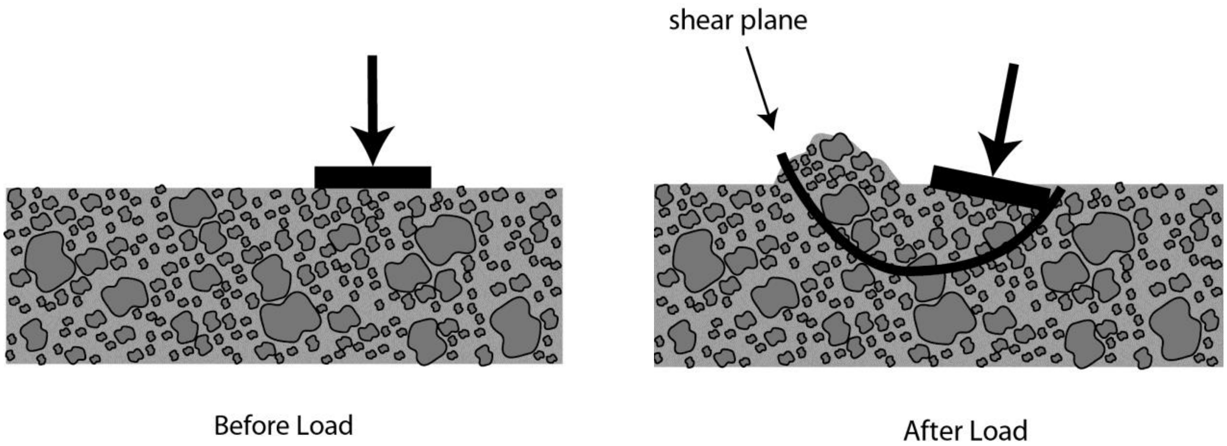
Extreme Minimum Pavement Temperature (°C)	Seven-Day Maximum Pavement Temperature (°C)					
	46 (114.8°F)	52 (125.6°F)	58 (136.4°F)	64 (147.2°F)	70 (158°F)	76 (168.8°F)
-40 (-40°F)	PG 46-40	PG 52-40	PG 58-40	PG 64-40	PG 70-40	PG 76-40
-34 (-29.2°F)	PG 46-34	PG 52-34	PG 58-34	PG 64-34	PG 70-34	PG 76-34
-28 (-18.4°F)	PG 46-28	PG 52-28	PG 58-28	PG 60-28	PG 70-28	PG 76-28
-22 (-7.6°F)	PG 46-22	PG 52-22	PG 58-22	PG 64-22	PG 70-22	PG 76-22
-16 (3.2°F)	PG 46-16	PG 52-16	PG 58-16	PG 64-16	PG 70-16	PG 76-16
-10 (14°F)	PG 46-10	PG 52-10	PG 58-10	PG 64-10	PG 70-10	PG 76-10

Another factor in mix design that affects rutting is the type of aggregate used. Aggregates refer to any granular material formed from a natural rock substance. These are materials extracted directly from the ground in quarries or pits. They can be either sand and gravel or hard rock. Aggregate properties and aggregate gradation[445] play major roles in the potential for rutting of an AC pavement. The rutting resistance of an AC mix depends on the shear resistance[446] of that mix.

Figure 96 illustrates the shear loading behavior of aggregate. If the shear stress created by repeated wheel load applications exceeds the shear strength of the mix, then permanent deformation or rutting will occur. Cubical, rough-textured aggregates are more resistant to the shearing action of traffic than rounded, smooth-textured aggregates. Cubical aggregates also tend to interlock better, resulting in a more shear resistant mass of material. In addition, increased compaction during construction or the use of higher percentages of coarse aggregate[447] fractions in the aggregate gradation provides more stone-to-stone contact in the AC mix which, in turn, helps reduce pavement rutting. Thus, as temperatures and / or vehicle loads rise, the specification of the aggregate mix should include more cubical and rough-textured materials.

[444] FHWA, 2002b
[445] Aggregate gradation is the distribution of aggregate particles among various sizes, usually expressed in terms of cumulative percentages larger or smaller than each of a series of sizes (sieve openings) or the percentages between certain ranges of sizes (sieve openings).
[446] Shear resistance is measured as the force required to pull the pressure-sensitive material parallel to the surface to which it was affixed, under specific conditions.
[447] Coarse aggregate is naturally occurring, processed or manufactured, inorganic particles in prescribed gradation or size range, the smallest size of which will be retained on the number four (0.2 inch [4.8 millimeter]) sieve.

Figure 96: Illustration of Aggregate Shear Behavior[448]



Step 4 – Decide on Climate Scenarios and Determine the Magnitude of Changes

The climate scenarios used for this case study were developed in Task 2 of the broader Gulf Coast Study Phase 2. Two specific temperature scenarios were developed for the 21st century; a "Warmer" narrative and a "Hotter" narrative. The "Warmer" narrative represents the 5th percentile (mean-1.6 SD[449]) of all the climate model outputs under the range of climate scenarios considered, whereas the "Hotter" narrative represents the 95th percentile outputs (mean+1.6 SD). Please refer to the *Climate Variability and Change in Mobile, Alabama*[450] and *Screening for Vulnerability* [451]for more details on how these narratives were developed.

Table 54 below provides a summary of projected changes to the pavement design-related temperature variables discussed in Step 3 under both the "Warmer" and "Hotter" narrative. The "Warmer" narrative projects a slight decrease in temperature of the coldest day in the near term and a slight increase in the maximum seven consecutive day average high temperature. On the other hand, the "Hotter" narrative projects a large increase in the temperature of the coldest day and the seven consecutive day average high temperature over the course of the 21st century.

Step 5 – Assess Performance of the Proposed Facility

ALDOT currently recommends the use of either PG 67-22 or PG 76-22 in the Mobile region, depending on expected traffic loads.[452] PG 67-22 is the most common application; PG 76-22 is specified for use only as a surface layer on high traffic load roads. To evaluate whether these binder specifications will need to change in the future due to climate change, the temperature

[448] Santucci, 2001
[449] Standard deviation
[450] USDOT, 2012
[451] USDOT, 2014
[452] ALDOT, 2012

projections in Table 54 were converted to pavement temperature values to enable selection of the appropriate PG rating from Table 53 using the formula shown in Step 3.

Table 54: Observed and Projected Pavement Design Related Temperature Variables in Mobile, Alabama[453]

	Observed (Model Baseline)[454] 1980-2009	"Warmer" Narrative			"Hotter" Narrative		
		2010-2039	2040-2069	2070-2099	2010-2039	2040-2069	2070-2099
Maximum Seven Consecutive Day Average High Temperature (°F) (50th Percentile)	94 (34.4°C)	94 (34.4°C)	95 (35°C)	96 (35.6°C)	97 (36.1°C)	99 (37.2°C)	103 (39.4°C)
Coldest day (°F) (50th Percentile)	20 (-6.7°C)	19 (-7.2°C)	20 (-6.7°C)	20 (-6.7°C)	23 (-5°C)	25 (-3.9°C)	28 (-2.2°C)

The analysis first verified that the PG 67-22 and PG 76-22 specifications are appropriate for Mobile's current climate. This was found to be the case. With regard to future climate, the analysis determined that no changes would be required to the minimum temperature rating (-22) under either climate narrative since, at most, the 50th percentile coldest day is expected to only get one degree Fahrenheit cooler than present. This translates to a minimum pavement temperature of 23.9°F (-4.5°C), well within the tolerance of the -22 specification. In terms of the maximum temperature rating with the lowest threshold (67), the analysis found that no change to this rating would be required under either of the narratives at any of the three future time periods tested. The highest projected ambient temperature across any of the scenarios evaluated, 103°F (39.4°C) under the "Hotter" narrative in the 2070-2099 time period, produces a maximum pavement temperature of 152.3°F (66.8°C), within the range of the 67 (and 76) maximum temperature ratings. Thus, the PG 67-22 and PG 76-22 specifications for AC binder are determined to be adequate for future projects in Mobile throughout the 21st century, despite the likely rise in projected temperatures. That said, the PG 67-22 specification comes close to being inadequate late this century if the "Hotter" narrative is realized.

Step 6 – Identify Adaptation Option(s)

As noted in Step 5, the current PG 67-22 and PG 76-22 specifications are expected to continue to be appropriate throughout the 21st century in Mobile. Thus, no adaptations to current AC mix design specifications are anticipated to be required in Mobile at this time. In other locations where projected temperature changes are greater, changes to binder specifications may need to be made and the appropriate adaptation in the mix can be determined by consulting Table 53.

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[453] Source: USDOT, 2014. Note: Figures shown represent an average across the five regional weather stations.
[454] The observed values represent calibrated statistical values derived from climate models as opposed to actual historical observations. Use of the model baseline allows for a more consistent comparison of past and projected future climate conditions.

In addition, as previously mentioned, aggregate type also plays a role in preventing rutting. Thus, an additional adaptation measure for locations expecting much warmer conditions would be to adjust the aggregate specifications of the mix. Specific options for doing this could include the following:

- Moving to a coarser aggregate that increases compaction during construction
- Using higher percentages of coarse aggregate fractions in the aggregate gradation thereby providing more stone-to-stone contact in the AC mix which, in turn, helps reduce pavement rutting
- Using Stone Matrix Asphalt (SMA) mixes. These mixes are designed to provide more direct stone-to-stone contact to help resist rutting. In an SMA mix, the stone skeleton is intended to carry the load and the fine aggregate particles are used to fill up the void space in the skeleton. In a dense graded mix, the fine aggregate is locked between larger aggregate particles and the load is transferred through the entire uniformly graded structure.

Other adaptation considerations, outside of mix design changes, that could be utilized to help minimize rutting in areas with projected hotter temperatures due to climate change could include:

- Use of thicker pavement sections at the time of initial design
- Consideration of PCC pavement versus AC pavement in certain applications
- Changing the frequency of maintenance

As with any complete pavement design process, these options should be subjected to a life-cycle cost comparison.

PCC

Modern specifications should account for the use of improved materials in order to ensure improved PCC performance under hotter placement conditions; conditions likely become more common in Mobile and throughout much of the country with climate change. To provide improved performance for sections paved under hot weather conditions, one adaptive option is that Continuously Reinforced Concrete Pavement (CRCP)[455] reinforcement standards be re-designed to provide steel quantities for specific use during hot weather conditions, and that an end result specification that limits the maximum in place PCC temperature during hydration be implemented[456]. The higher expense of this option especially warrants a life-cycle cost analysis. Hydration relates to the fact that when Portland cement[457] is mixed with water, heat is released. This heat is called the heat of hydration, the result of the chemical reaction between cement and water. The heat generated by the cement's hydration raises the temperature of PCC.

[455] Continuously reinforced concrete pavement is Portland cement concrete pavement with continuous longitudinal steel reinforcement and no intermediate transverse expansion or contraction joints.
[456] FHWA has developed concrete pavement design software, HIPERPAV III® that includes modules to address increased temperature and hydration procedures
[457] Portland cement is a product manufactured from limestone and clay that hardens under water and acts as the binder in a concrete mix.

One possible measure to minimize the potential problems associated with hot weather concreting can be to control the PCC mixture temperature. Under hot weather placement conditions, an effort should be made to keep the PCC temperature as low as economically feasible. By controlling the temperature of the ingredients, the temperature of the fresh PCC can be regulated.

Other possible measures could include (in the order of likely feasibility):

- The scheduling of placement activities during the times of the day or night when the weather conditions are favorable
- Minimizing the time to transport, place, consolidate, and finish the PCC
- The use of PCC materials and proportions with satisfactory performance in place under hot weather conditions
- The use of a PCC consistency that allows rapid placement and effective consolidation at high temperatures.
- Protecting the PCC from moisture loss at all times during placement and during its curing period such as by covering PCC with impervious paper or plastic sheets or applying membrane-forming curing compounds
- Use of cooled PCC, which can be achieved by using chilled mixing water, ice in the mixture, or rarely, the use of liquid nitrogen to cool the mixing water or the PCC mixture, or the cooling of the coarse aggregate

Step 7 – Assess Performance of the Adaptation Option(s)

As no adaptation option was required with respect to AC mix design, the performance of the adaptive design options was not formally assessed for this case study. If different climate scenarios for a given area required different binder specifications, this part of the analysis should entail a comparison documenting how the specification optimized for each scenario performs under all the other scenarios tested.

Step 8 – Conduct an Economic Analysis

An economic analysis of each adaptation option was not conducted for this case study; however, it is recommended that an analysis that includes a life-cycle cost comparison be conducted prior to determining whether and how to change pavement design standards if temperatures show signs of changing over time. Generally speaking, adjusting PG grades of mix designs based on temperature changes is a fairly low cost adjustment and a good choice economically over the long term relative to the chance of premature rutting in new AC pavement which could require costly maintenance and repairs.

Step 9 – Evaluate Additional Decision-Making Considerations

If an adaptation was required, before selecting an option one would need to consider whether there are any other specific factors relevant to their operations and include those into their decision-making. For pavement mix design, these factors might include:

- Broader project sustainability beyond just climate change adaptation (e.g., the relative sustainability factors of AC versus PCC in the project area)
- Maintenance funds availability
- Capital funds availability
- Stakeholders' expected quality or level of service

Step 10 – Select a Course of Action

Since climate projections do not show a change in temperature patterns great enough to require changes to pavement mix design in Mobile, the recommended course of action for new projects occurring at this time is not to undertake any adaptations to current practice. That said, if evidence emerges that temperatures are trending in line with the "Hotter" narrative, then starting mid-century it may make sense to re-evaluate changing the PG 67-22 standard to a higher specification that can handle heat better. The cost of the potential adaptations / changes should be weighed against potential benefits (avoided traffic delays and construction costs associated with repairing pavements if they fail early), and with an understanding of the leanness of budgets now and in the future. For PCC pavements, since the analysis is about practices on the day of installation, it probably doesn't make sense to change construction techniques until temperatures are too hot to support current practices.

Step 11 – Plan and Conduct Ongoing Activities

Agencies need to monitor temperature changes to assess whether conditions are trending in line with the climate scenarios tested. Once an upward trend of higher seven consecutive day temperatures is clearly established, the mix design could be adjusted to account for these changes. Current pavement design procedures and software include consideration of environmental factors. Agencies' pavement engineers can incorporate forecasted changes in the environmental factors into the pavement design process. As noted above, such proactive adaptations might start first on the most critical infrastructure, perhaps even prior to a PG specification threshold being crossed, and then proceed to less critical infrastructure once it's clear a new climate regime has arrived. It is important as well to monitor all new pavements to ascertain if the current and, in the future, revised mix designs or other measures incorporated into the new pavement design perform as expected.

Conclusions

This case study provided a high-level, non-site specific analysis of how projected changes in temperature in Mobile, Alabama might impact pavement mix design on AC and PCC roads. It was determined that under both the "Warmer" and "Hotter" narratives, temperature changes were not great enough to require any adaptations from current practice at this time although this conclusion may need to be re-evaluated later in the 21st century. The primary lesson learned from this case study is the need to monitor temperature changes, periodically update historical temperature records, and use climate projections where appropriate instead of simply using outdated 20th century numbers.

Moving from exploratory research that raises awareness of climate change to practical guidance aimed at reducing costs and safeguarding infrastructure will require additional efforts and collaboration. Pavement engineers, with assistance from government agencies and climate change experts, should be encouraged to develop a protocol or guide for considering potential climate change in the development and evaluation of future designs. The guide should extend beyond the narrow focus on pavement mix design in this case study to incorporate all elements of climate change impacts on pavement design (e.g., subbase, drainage, pavement texture). Researchers should explore if and how the AASHTOWare Pavement Mechanist Empirical (ME) Design software[458] can be adapted to incorporate climate change. The current software allows input of historic weather station data for climate models for a given project location. It then takes this data and projects weather conditions for the design life of the pavement. Amending the software to incorporate climate change projections would be a logical next step in its development.

[458] AASHTO, 2014

4.4.10 Continuous Welded Rail Exposure to Temperature Changes

Introduction

Temperature is a critical consideration in both the laying of rail tracks and in their continued reliable condition. If rail temperature is not properly considered in the laying of track, the track structure can become disturbed in periods of extreme heat or cold requiring maintenance expenditures, causing train delays, and leading to a heightened risk of derailments. The physics are rather simple. Steel rail will expand with heat, possibly resulting in a track perturbation commonly called a sun kink (see Figure 97). In extreme cold weather, the contracting rail can literally pull itself apart, more commonly, at the joints or the welds.

Case Study Highlights

Purpose: To understand whether continuous welded rail (CWR) in Mobile could be vulnerable to projected increases in temperature.

Approach: Considering a generic CWR in Mobile, the minimum desired rail neutral temperature was calculated using an equation from AREMA (2013). Then, an evaluation was made as to whether the neutral temperature would need to change under the projected future temperatures.

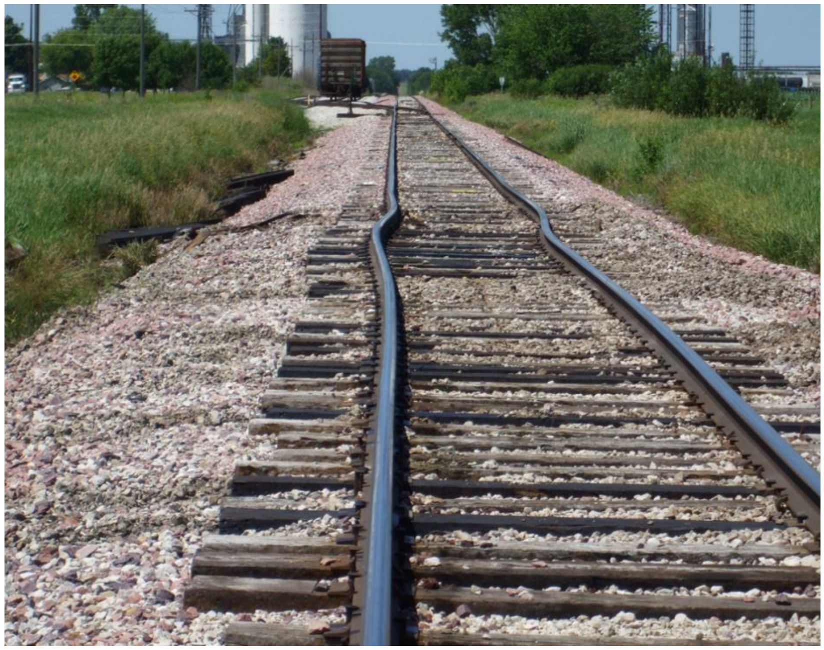
Findings: The rail in Mobile may not be vulnerable under less severe climate change conditions, but could be vulnerable under more severe ones.

Viable Adaptation Options:

- Increase rail neutral temperature
- Ensure that ballasted tracks have sufficiently wide shoulders to support the ties

Other Conclusions: Monitoring temperature trends, and keeping track of buckling or kinking incidents, may help alert track managers to the appropriate time to take proactive adaptation measures.

Figure 97: Example of a Hot Weather Sun Kink[459]



According to the Federal Railroad Administration (FRA) Office of Safety database, there were over 150 derailments nationwide between 2005 and 2009 related to track buckles or sun kinks, resulting in $43 million in damages.[460] Railroads work very hard to avoid and prevent derailments because of their cost, the resulting damages and claims, and the potential for personal injuries and fatalities. With ambient temperatures greater than 95^0 F (35°C) railroads issue slow orders whereby train speeds are lowered. This has the effect of decreasing stress on the rail and allows train crews a few more seconds to identify any track perturbations. As a practical matter, the speed restrictions bring passenger train speeds down to freight speeds. On a freight only line, the effect on operations is minimal until a track perturbation actually occurs or is discovered.

Cold weather pull-aparts create operational challenges but, likely, fewer derailments because if a pull-apart occurs along the track, the integrity of the track signal circuit[461] may be breached and the wayside signals would display a "stop," "stop and proceed," or "proceed at restricted speed" indication (more often than expected the pull-aparts do not drop the signal. This is caused by track components, like tie plates, bridging the connections). A train normally can negotiate a pull-apart rail gap of three inches (7.6 centimeters) or less at slow speed (10 miles per hour [16.1 kilometers per hour] or less). Where the broken rail gap does not exceed three inches (7.6

[459] Iowa DOT, 2013
[460] Zhang and Nizer, 2010
[461] A track signal circuit refers to the electrical current run through rails that is used to detect train locations and aid in train signaling.

centimeters), the train can be “walked” across the gap under the supervision of a qualified maintenance of way supervisor. Otherwise, the broken rails would need to be de-stressed and reconnected by means of a temporary joint bar until the rails can be re-welded.

Recognizing the operations and safety challenges temperatures can pose to rails, this section will investigate the impacts that projected temperature changes might have on new track laying and maintenance of way practices in the Mobile region over the 21st century using the *General Process for Transportation Facility Adaptation Assessments*. The focus will be on Continuously Welded Rail (CWR)[462] which is most prevalent on mainline Class 3 tracks[463] in the Mobile area and where the biggest impacts would be felt from delays and derailments owing to temperature-related problems. Light rail and subway rail lines, although also sensitive to temperature, have different characteristics from the rails studied here and are not included in this analysis. The assessment finds that track-laying practices may need to be adapted in the future under one of the climate scenarios tested. However, no adaptation actions are recommended at this time except to monitor conditions as adaptations can readily be made to the existing track if conditions warrant.

Application of the General Process for Transportation Facility Adaptation Assessments

Step 1 – Describe the Site Context

Three Class I[464] railroads own and maintain rail lines in the Mobile region; CSX, Norfolk Southern, and Canadian National. Although subject to the same regional environmental factors, each railroad has its own approach and practices for laying and maintaining rails. This case study is intended to apply to all of the railroads, although much of the analysis will focus on CSX practices specifically because of data availability.

Step 2 – Describe the Proposed Facility

No specific facility is examined in this case study. Instead, the analysis is applied to a generic segment of new CWR track on one of the CSX or Norfolk Southern rail lines found to be critical in the Mobile region in the *Assessing Infrastructure for Criticality in Mobile, Alabama* report.[465]

Step 3 – Identify Climate Stressors That May Impact Infrastructure Components

The key environmental variables in the performance of rail track are the absolute expected maximum and minimum air temperatures over the lifespan of the rail installation (typically less

[462] Rail lines consist of either CWR or jointed rail sections with CWR being most commonly employed on main line tracks and jointed rail on secondary tracks. Jointed rail was the earliest form of rail installation and consists of individual segments of rail each typically between 39 and 78 feet (11.9 and 23.8 meters) long. Jointed rails are mechanically joined by means of joint bars to firmly support the abutting rail ends and to allow longitudinal movement of the rails in the joint to accommodate expansion and contraction due to rail temperature variations. In CWR, commonly employed since the 1950s on main lines, individual rail segments are welded together into strings that can be miles long between joints allowing for a smoother ride, lower maintenance requirements, and higher speeds.
[463] FRA track classifications relate different track geometry to maximum authorized speed for freight and passenger trains. The maximum speeds for freight trains and passenger trains on Class 3 tracks are 40 and 60 miles per hour (64.4 and 96.6 kilometers per hour), respectively.
[464] Railroads are classified by the U.S. Surface Transportation Board based on their annual operating revenue over a three year period. Class I railroads are the highest revenue railroads with annual operating revenues of $250 million or more over each of the last three years (adjusted for inflation).
[465] USDOT, 2011

than twenty years, sometimes significantly less, for mainline rail[466]). Ambient air temperatures impact rail temperatures; excessively high rail temperatures can lead to rail expansion and sun kinks and excessively low rail temperatures can lead to rail shrinkage and pull-aparts.

The concept of "neutral temperature" is of paramount importance in laying CWR in order to reduce the risk of such hazards. Neutral temperature is defined as the rail temperature in the CWR rail section that would result in zero thermal stress[467] in the rail section. Thermal stress occurs when the rail temperature increases or decreases from the neutral temperature and the rail seeks to expand or contract but is limited in its ability to do so at the ends of the string or where it is anchored. During extreme high rail temperatures, the resulting force due to rail expansion can overcome the ability of the ties and ballast shoulders to hold the rail in place leading to sun kinks. During extremely low temperatures, a rail break can occur when the resulting stress exceeds the strength of the rail section, resulting in a rail pull-apart. The resulting pull-apart gap between the broken rails is limited by the rail anchors and clips on each tie. Some railroads elect to anchor the rail at specific locations (e.g., placing an anchor at every other tie, near switches-box anchor 200 ties before and after). A more uniform distribution of the anchors may help in preventing buckling derailments.

The desired neutral temperature is determined by the temperature of the rail (not the ambient temperature) at the time of its installation and fixing to the ties.[468] There is an optimal range at which to set the neutral temperature that is based on the expected temperature patterns at the installation site: installing a rail at too high a neutral temperature might result in a higher risk of pull-aparts in cold temperatures and installing it too low may result in greater risk of sun kinks during warm temperatures.

The acceptable range for the neutral temperature is determined by the following equation:[469]

$$\text{Minimum Desired Rail Neutral Temperature} = ((2H_T+L_T)/3) + 10$$

$$\text{Maximum Desired Rail Neutral Temperature} = [((2H_T+L_T)/3) + 25] \pm 5$$

Where,
H_T = Highest rail temperature projected over its design life (in Fahrenheit)
L_T = lowest rail temperature (in Fahrenheit)

Note that the formula makes use of the highest and lowest *rail* temperatures, not the highest and lowest ambient air temperature. This is because it is the temperature of the rail, not ambient

466 As main line rails become worn, they are typically taken up and re-used on lower speed branch lines. The ultimate lifespan of a rail can be upwards of sixty years although it might be re-laid multiple times during that period. The neutral temperature (see paragraph below this footnote) can be reset with each rail laying depending on the requirements of the new location.
467 Thermal stress is only one of the many stresses that affect rails: other stresses come from train loads and train motions.
468 The rail temperature at the time of installation is influenced by the ambient temperature but can be adjusted in the field as needed using specialized equipment so that the desired neutral temperature can be achieved regardless of weather conditions at the time of installation.
469 AREMA, 2013

temperature alone, that counts when determining thermal stresses.[470] Thus, to assess the impact of changing climate conditions on the setting of neutral temperatures, it is necessary to draw a relationship between the maximum and minimum ambient air temperature (as output from climate models) and actual rail temperatures. FRA guidance states that rail temperature shall be considered 30ºF higher than ambient temperatures in hot weather and equal to ambient air temperature in cold weather.[471]

The FRA has also developed, and Amtrak has tested, a more sophisticated model for relating ambient air temperatures to rail temperatures that constitutes a rail weather system to help predict track buckling risks in real time given actual weather conditions and known track attributes.[472] Typically, the railroad knows—or can easily find out—the ambient temperature along the line. It is the *rail* temperature, however, that causes the track to expand and possibly buckle the track. This model is based on the heat transfer process of a rail exposed to the sun. A rail weather station was established and used to calibrate the model. The station was composed of a portable weather station and a short segment of rail track with sensors installed on both rails. Ambient temperatures were taken by the weather station and the rail temperature by rail thermometers. Data from these instruments were sent to a control office for further action if required. Modeled results have been compared to actual conditions; the model predicts the maximum rail temperature within a few degrees and within 30 minutes of the actual time when the high temperature occurs during the day. While it would be ideal to assess the impact of projected temperature patterns using such a system, time and budget considerations dictate that a more basic analysis using FRA's guidance relating ambient to rail temperature will be used in this case study. Exploring the possibility of using a rail weather system with projected climate inputs is, however, a recommended area for future research.

Step 4 – Decide on Climate Scenarios and Determine the Magnitude of Changes

The climate scenarios used for this case study were developed under an earlier phase of the broader Gulf Coast Study. Two specific temperature scenarios were developed, a "Warmer" narrative and a "Hotter" narrative. These narratives were chosen to "bound" the range of model outputs. The "Warmer" narrative represents the 5th percentile of all the climate model outputs under the range of climate scenarios considered, whereas the "Hotter" narrative represents the 95th percentile outputs. Please refer to the *Climate Variability and Change in Mobile, Alabama*[473] and the *Screening for Vulnerability* report[474] for more details on how these scenarios were developed.

As shown in Table 55, each scenario entails different temperature patterns. Under the "Warmer" narrative, maximum temperatures are projected to remain largely unchanged while minimum

470 Due to solar radiation, the rail will be much hotter than the ambient air temperature during the day.
471 FRA, 2013
472 Zhang and Nizer, 2010
473 USDOT, 2012
474 Source: USDOT, 2014. Note: Figures shown represent an average across the five regional weather stations.

temperatures are projected to decrease slightly. With the "Hotter" narrative, both maximum and minimum temperatures are projected to increase.

Table 55: Projected Changes to Maximum and Minimum Temperatures in Mobile, Alabama[475]

	Observed (Model Baseline)[476] 1980-2009	"Warmer" Narrative			"Hotter" Narrative		
		2010-2039	2040-2069	2070-2099	2010-2039	2040-2069	2070-2099
Maximum Annual Highest Maximum Temperature (°F)	103	102	103	103	106	109	113
1st Percentile Coldest Day (°F)	4	-1	0	1	15	18	20

Step 5 – Assess Performance of the Proposed Facility

As discussed in Steps 1 and 2, the proposed facility is a new CWR installation anywhere within the Mobile region (either on a new rail line or a rail replacement on an existing line). Per the guidance for new facilities in the *General Process for Transportation Facility Adaptation Assessments*, the base case proposed facility should be based on a traditional design or practice that one would employ without consideration of climate change. Thus, in this case study, the base case is defined as the standard practice, the standard neutral temperature, one would employ when laying rail in the Mobile region today.

Although constrained by the formula discussed above, each railroad operating in the Mobile region employs its own practices when defining what specific neutral temperature will be used in each installation. FRA requires each railroad to develop its own plan, which then becomes a self-regulating proscriptive document. This analysis will focus on one of Mobile's Class I railroads which currently uses 100ºF (37.8°C) as the neutral temperature on all their CWR tracks in the Mobile region. Using the neutral temperature formula shown in Step 3 and the FRA guidance which states that rail temperatures can be assumed to be 30°F (16.7°C) higher than the ambient temperature in hot weather, it was determined that the acceptable neutral temperature range given historical conditions is between 100ºF (37.8°C) and 115ºF +/- 5ºF (46.1°C +/- 2.8°C).

Referring to the formula in Step 3, an example of the calculation for the minimum desired rail neutral temperature under existing conditions is as follows:

Observed Maximum + FRA Guidance = H_T

[475] USDOT, 2014
[476] The observed values represent calibrated statistical values derived from climate models as opposed to actual historical observations. Use of the model baseline allows for a more consistent comparison of past and projected future climate conditions.

$103 + 30 = 133 = H_T$

$((2H_T + L_T) / 3) + 10$ = Minimum Desired Rail Neutral Temperature

$(2x133 + 4) / 3 + 10$ = Minimum Desired Rail Neutral Temperature

100°F (37.8°C) = Minimum Desired Rail Neutral Temperature

Thus, the sample railroad's neutral temperature of 100ºF (37.8°C) is representative of the lower end of the acceptable range.

Next, an evaluation was made to determine if the neutral temperature of this railroad would need to change under the "Warmer" and "Hotter" Climate narratives. Table 56 shows the acceptable "Warmer" and "Hotter" narrative neutral temperature ranges for various future time periods. As shown in the table, the current example railroad's practice of setting a neutral temperature of 100ºF (37.8°C) would remain acceptable throughout the 21st century under the "Warmer" narrative. However, a 100ºF (37.8°C) neutral temperature would be inadvisable under the "Hotter" narrative at all future time periods; continuing to use this neutral temperature might increase the risk of sun kinks in the future.

Table 56: Acceptable Rail Neutral Temperature Ranges (ºF) in Mobile Considering Climate Projections

	Current Range	Projected 2010-2039	Projected 2040-2069	Projected 2070-2099
"Warmer" Narrative	110-115+/-5	98 to 113+/-5	99 to 114+/-5	99 to 114+/-5
"Hotter" Narrative		106 to 121+/-5	109 to 124+/-5	112 to 127+/-5

Step 6 – Identify Adaptation Option(s)

As noted above, no adaptive actions to this railroad's neutral temperature practices would be required under the "Warmer" narrative. The "Hotter" narrative, on the other hand, would require adaptive actions because the current neutral temperature used by this railroad would fall below the acceptable range given projected temperature increases. It should be noted that neutral temperature does not stay at the set values. Based on location, grade, traffic, and maintenance activities the neutral temperature will shift or drift typically in down direction. Some railroads bias to the highest neutral temperature in the acceptable range so when this drift occurs, it will stay in the acceptable range longer.

The primary adaptation action that could be taken on new CWR track would be to increase the rail neutral temperature to within the ranges shown in the bottom row of Table 56 when laying new track. If necessary, rail neutral temperature can be reset on existing track by removing, re-stressing, and reinstalling the rail. Activities such as this are frequently bundled into a comprehensive track maintenance program, rather than it being a program in itself. An additional adaptive action, and one that could be implemented on existing rail lines as well, would be to ensure that ballasted tracks are constructed and maintained with minimum one foot (0.3 meter) wide shoulders to provide lateral support to the ties. Frequently, the buckled track results from insufficient ballast section. A fully ballasted track section with maintained shoulders provides the resistance to tie displacement caused by lateral rail forces during extreme high rail temperatures. Ensuring ballast is fully deployed and properly maintained can help lessen the risk of track buckling incidents.

Step 7 – Assess Performance of the Adaptation Option(s)

The likelihood of either the "Warmer" or the "Hotter" narrative occurring cannot be determined due to uncertainties in climate modeling. Thus, it is prudent to consider how each of the two adaptation options for the "Hotter" narrative discussed in Step 6 would perform if they are implemented but there are no climate changes or if the "Warmer" narrative were to occur. With respect to increasing the neutral temperature in line with the ranges shown in the bottom row of Table 56, this adaptation option would perform acceptably under the "Warmer" narrative so long as the increase in the neutral temperature is kept below 115ºF (46.1 ºC). The second adaptation option, monitoring to insure that the ballasted track section with shoulders is maintained, would be beneficial under every climate scenario.

Step 8 – Conduct an Economic Analysis

An economic analysis was not included in this case study and is not recommended in the case of rail neutral temperature issues. Rail neutral temperature is very costly to measure on in-service tracks using current techniques. New less expensive methods are being developed and should be available in the near future. There is little to no added cost to setting different rail neutral temperatures and the maintenance of a fully ballasted track should already be a part of the railroads maintenance program.

Step 9 – Evaluate Additional Decision-Making Considerations

Each railroad will need to consider whether there are any specific "soft" non-economic factors relevant to their operations and factor that into their decision-making. Two such considerations might be safety and public relations issues related to derailments caused by sun kinks or pull-aparts. What is the risk tolerance of the railroad to such concerns? Note that, to some extent, this might tie back to the materials typically hauled on that rail segment (e.g., rail lines commonly used to haul hazardous materials may be more a priority for re-setting neutral temperatures than those that primarily handle coal and grain).

Step 10 – Select a Course of Action

The purpose of this engineering case study was to determine if new rails on a Mobile area railroad were vulnerable enough to projected temperature changes to consider some sort of adaptation in track laying procedures. The investigation showed that standard practices in place within Mobile today are acceptable if the "Warmer" narrative bears out and unacceptable if the "Hotter" narrative were to occur. However, there is uncertainty as to which scenario will actually happen. The prudent course of action is therefore to continue to monitor temperature trends and incident levels to see if they are trending along the lines of the “Hotter” narrative and, if so, at some point in the future consider changing rail neutral temperature practices. For existing rail lines, this may necessitate relaying the track and / or beefing up ballast shoulder maintenance.

Step 11 – Plan and Conduct Ongoing Activities

In general, sound maintenance and good inspections will continue to be the keys to future derailment prevention. When accidents do occur, railroads should continue to track the number of heat and cold-related incidents on their facilities and adjust practices accordingly based on climate changes. FRA’s R&D department is working on low solar absorption coatings for rail that will significantly reduce the heat absorbed by the rail and reduce overall peak rail temperature. These coatings could significantly reduce the risk of track buckling. Agencies with responsibility for rail facilities should stay apprised of the results of this research.

Conclusions

This case study determined that projected climate changes will have an impact on the laying of CWR rail tracks in Mobile if the “Hotter” narrative is realized. The rail neutral temperature will need to be raised under this climate scenario. The “Warmer” narrative will not require a change in current rail laying practices in Mobile.

A similar evaluation of rail neutral temperature practices should be considered more broadly for other portions of the country. The viability of using FRA’s rail weather system for this analysis should also receive consideration as a future research project.

4.4.11 Operations and Maintenance Activity Exposure to Climate Change and Extreme Weather Events

Introduction

Operating and maintaining transportation facilities and networks is critically important to the performance and longevity of transportation systems. Operations and maintenance (O&M) activities must address significant ongoing challenges such as aging infrastructure and increases in traffic volumes as well as the added threat of long-term climate change impacts. Operations discussed in this section primarily pertain to the management of traffic flow despite disruptions – in this case, as a result of climate change or extreme weather events. Examples of operations activities under this context include emergency response protocols, pre-deployment of emergency supplies and equipment, use of Intelligent Transportation Systems (ITS) to disseminate critical information to travelers, or established procedures for emergency closures of roads or bridges. Maintenance, on the other hand, refers to the process of maintaining or preserving transportation assets (e.g., bridges, pavement, embankments, signage). In this section, maintenance activities are characterized as either "planned" (including "preventive" and "routine" maintenance) or "on-demand" (also known as "reactive" or "corrective" maintenance).

Case Study Highlights

Purpose: Discuss how operations and maintenance (O&M) are affected by climate stressors.

Approach: An asset-specific engineering assessment was not conducted for this case study, since it was focused on O&M and not actual infrastructure design. Using Mobile as an example, this case study discusses how O&M can be disrupted generally, and how transportation organizations can adapt and prepare for these challenges.

Findings: Climate impacts on O&M activities can cause strains on budget and service disruptions.

Adaptation Measures: Careful planning and training can help minimize impacts on O&M. Mobile-specific examples are provided throughout this case study.

This section discusses how weather and climate may influence O&M activities and how current activities may be adapted to reduce the vulnerability of transportation systems to weather and climate-related risks. While this chapter primarily focuses on maintenance, discussion of operations, such as emergency management and ITS strategies from interviews in the Mobile Area are included as relevant to maintenance practitioners. Highways receive primary attention but examples from other modes are provided as well, where applicable. The section first provides an overview of how O&M activities are organized, planned, and performed on the highway system. Second, it discusses how these activities could be affected by climate change due to impacts to the maintenance crews. Finally, it recommends improvements to O&M procedures in the face of changing environmental conditions, in general, and specific changes in practice identified during the Gulf Coast Project.

An Overview of Transportation O&M

Daily O&M activities influence how users experience the transportation system more than any other function of transportation agencies.

Operations

Operations encompass management of the flow of traffic and coordinating responses to crashes and disruptions due to weather and other factors. Operations range from minute-to-minute reporting of and response to traffic conditions through active traffic management (e.g., traffic lights) and the fine tuning of the Intelligent Transportation System (ITS) components (e.g., Variable Message Signs (VMS), ramp meters, roadway and weather sensors) to the coordination of major responses to natural or manmade disruptions. Operations for a transportation system (particularly for a highway system) tend to be more centralized than maintenance due to the level of coordination needed to oversee traffic flow over larger geographic regions.

Efficient and effective system operations are an important component of overall transportation system performance. One estimate indicates that traffic incidents account for 25 percent of the nation's traffic congestion, with poor traffic signalization accounting for another five percent, and bad weather accounting for yet another 15 percent.[477] The exact percentages will vary based on location. As the severity of an incident increases, more agencies become involved with a corresponding need for coordination across state DOTs, emergency management agencies, emergency responders, enforcement agencies, public health officials, and humanitarian relief organizations.[478] This need for a coordinated response was demonstrated following recent weather-related natural disasters such as Hurricanes Katrina and Sandy and Tropical Storm Irene, tornadoes in Oklahoma, and landslides in Washington.

Maintenance

Maintenance activities are conducted by dedicated local maintenance crews. The activities they perform fall within two key categories:

- Planned activities (includes "preventive" and "routine") tend to involve activities that can be scheduled with some amount of certainty, such as routine maintenance paving or grass mowing.
- On-demand (also known as "reactive" or "corrective") activities involve issues that occur on an unscheduled basis, such as damage to a sign or the appearance of a pothole.

Maintenance activities are generally undertaken by agencies responsible for a particular jurisdiction, such as the state highways within a particular county or a collection of counties, typically referred to as a "division" or "district." or "residency"). While cooperation with adjacent districts is by no means atypical, their activities tend to be specific to a fixed geography.

[477] FHWA, 2005
[478] Lockwood, 2008; AASHTO, 2013

Each district is typically equipped with the materials, machinery, and training needed to conduct a wide range of these activities on both a planned and on-demand basis.

Overview of the O&M Planning and Budgeting Process

Unlike a capital project that is part of a larger investment program, O&M activities are line items in an agency's budget and stand alone in terms of program accountability. The budget is tied to current and projected revenue and is fixed (no borrowing). Allocations are typically broken down by major program areas (e.g., pavements, bridges, signs, drainage, signal maintenance, emergency response, and contingencies) and activities are planned and implemented throughout the year with adjustments as needed within total funding availability. O&M expenditures are generally considered non-capital (i.e., "cash" expenditures), and are typically strained due to the general shortage of public agency "cash."

The fixed nature of O&M budgets has implications for planned versus on-demand expenses. On-demand activities may increase as the system ages, traffic levels increase, and increased urbanization causes greater runoff. As changes in climate add additional stress onto the system, on-demand activities will require even more resources. Due to the fixed nature of O&M budgets coupled with the dynamic maintenance needs in response to on-the-ground changes in condition, when funds are directed to on-demand activities, resources for planned activities become increasingly constrained, and the overall resource needs increase. Because many of the "planned activities" include preventive and routine maintenance designed to keep the system in a state-of-good-repair, shifting resources away from these activities may undermine the lifecycle management of the asset(s), leaving the system *more* vulnerable to extreme weather events over the long term. Through improved transportation asset management, however, a whole-life view of all assets can be provided to allow monitoring, tracking, and analysis of how funding strategies affect asset condition, and can allow an agency to make policy and strategic decisions regarding funding (such as cross-asset decision making or investment decisions).

Weather and Climate Impacts on O&M

Virtually all of the activities performed by maintenance crews are weather-dependent to a degree. In some cases, the work cannot be completed due to the weather events effect on the O&M activity (e.g., painting in the rain) and at other times it cannot be completed due to worker safety concerns (e.g., extreme heat days). Table 57 illustrates highway maintenance that cannot be completed during certain weather events. Heavy precipitation, lightning, and strong wind affect almost all maintenance activities. Severe storms will also disrupt most activities as listed in Table 57 and even a light rain or moderate wind can be enough to delay activities like painting or sign replacement. Increased temperatures are more likely to affect the maintenance crews' ability to work rather than affecting the work product.

Table 57: Maintenance Activities Impacted by Various Climate Stressors[479]

Maintenance Activity	Heavy Precipitation	Drought	Strong Wind	Lightning	Low Temperature	High Temperature
Replace Signage	•		•	•		
Maintain or Rehabilitate Concrete	•		•	•	•	•
Schedule Crews	•		•	•		•
Clear Drainage	•	•		•		
Repair Embankments	•	•	•	•	•	
Prevent Erosion and Sedimentation	•		•	•		
Excavation	•			•	•	
Fencing	•		•	•	•	
Painting	•		•	•	•	•
Paving	•				•	
Bridge Work	•		•	•		
Maintain Vegetation	•	•	•		•	•

Climate Change Impacts that Affect O&M Activities

The U.S. Department of Labor's Occupational Safety & Health Administration (OSHA) has devoted considerable effort to the avoidance of heat-related worker illness, and most maintenance organizations have dedicated safety professionals and continuous training to guard against heat illness and other threats to worker safety.[480] NOAA heat indices are used to monitor heat exposure and OSHA guidelines are used to regulate activities (see Table 58). As the heat index increases, more protective measures are necessary. For example, low levels of heat index do not necessarily lead to modified work schedules, whereas higher levels might lead to a reduction in the number of working hours on construction projects on a given day.

[479] Source: Meyer et al, 2014 (as modified)
[480] OSHA, 2013a; OSHA, 2013b; and OSHA, 2013c

Table 58: OSHA Guidance for Worksite Modifications According to Heat Indices[481]

Heat Index	Risk Level	Level of Protective Measures	Example Measures
Less than 91°F	Lower (Caution)	Basic heat safety and planning	• Provide drinking water • Ensure that adequate medical services are available • Plan ahead for times when heat index is higher, including worker heat safety training • Encourage workers to wear sunscreen
In addition to basic heat safety and planning, the following is also recommended for higher heat levels:			
91°F to 103°F	Moderate	Implement precautions and heighten awareness	• Alert workers of risk conditions • Remind workers to drink water often (about 4 cups / hour) • Review heat-related illness topics with workers: how to recognize heat-related illness, how to prevent it, and what to do if someone gets sick • Respond to heat-related illness and medical emergencies without delay • Schedule frequent breaks in cool, shaded area • Acclimatize workers • Set up buddy system / instruct supervisors to watch workers for signs of heat-related illness
103°F to 115°F	High	Additional precautions to protect workers	In addition to above: • Ensure adequate medical services are available • Have a knowledgeable person onsite • Establish and enforce work / rest schedules • Adjust work activities to help reduce worker risk • Use cooling techniques • Watch / communicate with workers at all times
Greater than 115°F	Very High to Extreme	Triggers even more aggressive protective measures	In addition to above: • Reschedule non-essential activity for days with a reduced heat index • Move essential work tasks to the coolest part of the work shift; consider earlier start times, split shifts, or evening shifts • Stop work if essential control methods are inadequate or unavailable

Table 59 provides examples of weather-related effects on infrastructure and attendant O&M activities. Relevant impacts to the Gulf Coast (Mobile Area in particular) are noted with an asterisk (*). All of the identified weather-related effects are already impacting locations in the U.S.; however, with shifting geographic climates, new areas are becoming exposed to each of the climate stressors. These areas will have to redirect resources – both financial and personnel – to be prepared to meet these new challenges. In addition to the direct impacts shown in Table 59,

[481]Source: OSHA, 2013b (as modified)

indirect and synergistic climate effects can also be of concern. For example, drought and wildfire conditions associated with climate change can increase sediment loading and cause trees to weaken and contribute to more dead wood in stream valleys. When combined with higher peak flows due to urbanization and increases in heavy precipitation events, the increased amount of dead wood may increase the probability that culverts become plugged. Combined, these stressors increase the likelihood of culverts failing catastrophically during flood events. Similarly, slope slides and rockfalls could increase and tree branches could cause power outages at rates that would otherwise be unexpected under current conditions.

Table 59: Summary of Climate Change Impacts on Maintenance of the Highway System[482]

	Climatic / Weather Change	Highway System Impact Requiring Maintenance	Impacts on Maintenance Work
Temperature	Change in extreme maximum temperature	• Premature deterioration of infrastructure* • Damage to roads from buckling and rutting* • Bridges subject to extra stresses through thermal expansion and increased movement • Closure of roads because of increased wildfires	• Safety concerns for highway workers from heat stress* • Increased attention to pavement failures*
	Change in range of maximum and minimum temperature	• Shorter snow and ice season • Reduced frost heave and road damage • Structures will freeze later and thaw earlier with shorter freeze season lengths* • Increased freeze-thaw conditions in selected locations creating frost heaves and potholes on road and bridge surfaces	• Decrease in frozen precipitation would improve mobility and safety of travel through reduced winter hazards, reduce snow and ice removal costs, decrease need for winter road maintenance, result in less pollution from road salt, and decrease corrosion of infrastructure and vehicles • Longer paving season in colder locations • Increased pothole work* • Vehicle load restrictions in place on roads to minimize structural damage due to subsidence and the loss of bearing capacity during spring thaw period (restrictions likely to expand in areas with shorter winters but longer thaw seasons)

[482] Source: Meyer et al., 2013 (as modified)

	Climatic / Weather Change	Highway System Impact Requiring Maintenance	Impacts on Maintenance Work
Precipitation	Wider Range of Precipitation Variability	• If more precipitation falls as rain rather than snow in winter and spring, there will be an increased risk of landslides, slope failures, and floods from the runoff, causing road washouts and closures as well as the need for road repair and reconstruction • Regions with more precipitation could see increased weather-related accidents, delays, and traffic disruptions (loss of life and property, increased safety risks, increased risks of hazardous cargo accidents)* • Closure of roadways and underground tunnels due to flooding and mudslides in areas deforested by wildfires • Increased wildfires during droughts could threaten roads directly, or cause road closures due to fire threat or reduced visibility • Clay subsurfaces for pavement could expand or contract in prolonged precipitation or drought causing pavement heave or cracking • Increasing precipitation could lead to soil moisture levels becoming too high (structural integrity of roads, bridges, and tunnels could be compromised leading to accelerated deterioration)* • Less rain available to dilute surface salt may cause steel reinforcing in concrete structures to corrode • Road embankments at risk of subsidence / heave • Drought-caused shrinkage of subsurface soils	• Increase in blocked culverts* • Removal of debris and other material from roads and roadsides • Need to re-open roads from landslides • Increased erosion from increased rainfall and from burned areas no longer protected from vegetation • Consideration of use of salt and other de-icing materials for varying levels of ice and snow • Possible impact on roadside mowing and handling of vegetation

	Climatic / Weather Change	Highway System Impact Requiring Maintenance	Impacts on Maintenance Work
Precipitation	Increased frequency of intense precipitation, other change in storm intensity (except tropical storms)	• Heavy winter rain with accompanying mudslides can damage roads (washouts and undercutting) which could lead to permanent road closures* • Heavy precipitation and increased runoff can cause damage to tunnels, culverts, roads in or near flood zones, and coastal highways* • Increase in weather-related highway accidents, delays, and traffic disruptions* • Increase in landslides, closures or major disruptions of roads, emergency evacuations and travel delays • Lightning/electrical disturbance could disrupt transportation electronic infrastructure and signalling	• Increase in on-demand maintenance* • Increase in bridge scour protection and response* • Increased attention to road drainage capacity and condition, road evacuation* • Increase in response to electrical outages* • Increase in operations monitoring and response need* • Disruption of planned maintenance work* • Lightning/electrical disturbance could pose risk to personnel, and delay maintenance activity
Sea level rise	Sea level rise	• Higher sea levels and storm surges increase corrosion risk to bridge resulting from decreased freeboard* • Temporary and permanent flooding of roads, underground tunnels, and other low-lying infrastructure due to rising sea levels* • Encroachment of saltwater leading to accelerated degradation of tunnels (reduced life expectancy, increased maintenance costs and potential for structural failure during extreme events)* • Loss of coastal wetlands and barrier islands will lead to further coastal erosion due to the loss of natural protection from wave action*	• Increase in on-demand maintenance* • Increased attention to drainage structure condition and capacity* • Increased attention to slope stability, erosion in right-of-way, saltwater intrusion to potable water sources* • Increased need for pumping of flooded facilities* • Increase in operations monitoring and response need*

	Climatic / Weather Change	Highway System Impact Requiring Maintenance	Impacts on Maintenance Work
Hurricanes	Increased tropical storm intensity (includes NorEasters and Hurricanes)	• Increased infrastructure damage and failure (highway and bridge decks being displaced)* • More frequent or widespread flooding of coastal roads* • More significant transportation interruptions (storm debris on roads can damage infrastructure and interrupt travel and shipments of goods) • Bridges are more prone to extreme wind events and scouring from higher stream runoff* • Bridges, signs, overhead cables, tall structures at risk from increased wind speeds* • Increased wind speeds could result in loss of visibility from drifting snow, loss of vehicle stability/manoeuvrability, lane obstruction (debris), and treatment chemical dispersion	• Increase in on-demand maintenance* • Increased attention to road flooding, roadside erosion, evacuations* • Increase in operations monitoring and response need* • Increased need for debris removal*

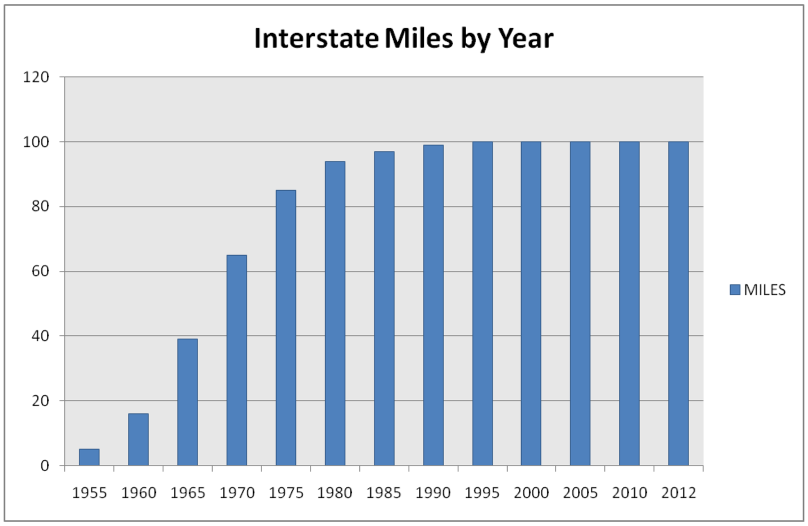
How O&M Activities Can be Adapted to Address Climate Change and Extreme Weather Events

Most of the mid-21st century highway network already exists and is built out.[483] Figure 98, for example, shows the slow pace of interstate mileage added per year since the early 1990s. The highways were constructed with long design lives so the current built out network is anticipated to serve travel demand needs for many years to come. Thus, the focus has shifted from new construction to O&M activities. These activates will account for much of the planning, preparation, monitoring, and response efforts needed to keep the system safe and efficient in the face of climate change. Part of this will include monitoring and forecasting changes in road use demands. Goods movement is likely to increase faster than passenger travel due to expectations of "just-in-time" delivery of goods, causing additional wear and tear to the road network. As mentioned previously, the annual planning and budgeting for O&M activities and the already-routine consideration of weather impacts in decision-making makes it easier to adapt activities, as needed, to prepare for and respond to a changing climate.

O&M activities have always contended with weather and adapted, by necessity, to changes in infrastructure condition, traffic levels, regulatory structures, and (knowingly or not) climate. The ability of organizations to successfully cope with a changing climate (manifested through new and increasingly severe weather patterns) may be limited by more immediate day-to-day concerns. Determining the appropriate role for O&M activities in addressing climate change involves considering what can be done within the confines of budgets, staffing, technology, and available information.

[483] The National Highway System is growing at a rate of only about one quarter of one percent per year, according to USDOT statistics (FHWA, 2010).

Figure 98: Total Interstate Mileage in Existence by Year[484]



Adaptation Actions at the Maintenance Department Level

Despite the uncertainty that remains (and will likely remain for the foreseeable future) with respect to future trends in some climate variables, it would seem prudent for O&M departments to have an understanding of projected climate trends and improve their situational awareness so that they can effectively respond.

General Maintenance Actions

Maintenance organizations can capitalize on experience gained in addressing weather-related infrastructure issues in one or several locations when seeking to proactively increase system resilience to climate change. Maintenance workers often work out of a single residency for the duration of their careers in service to a local constituency. This allows them to have an intimate knowledge of the facilities "owned" by the residency crews, their maintenance history, relative importance, and their resilience to weather events. Depending upon the structure of their work program, this knowledge can be greatly leveraged by integrating it into a transportation asset management system, where asset inventories (includes information about asset type, age, geographic location, etc.) and condition assessments are mapped to facilitate monitoring, performance assessments, risk analysis, and therefore provide necessary information to inform capital and O&M budgeting decisions. The residency organization would, thus, typically be able to provide ready and accurate answers to questions such as, "Which culverts are most likely to fail during a major storm event and which of these would cause the most system disruption?" If their work order system is tied to GIS, they can also usually provide a record of historic failures, repairs, and inspections to better predict the relative risks.

[484] McVoy, Venner, and Sengenberger, 2012

Analysis of work orders in response to weather events, monitoring of culvert conditions, and tabulation of slope failures can inform future maintenance decisions and be used for budget justifications. Likewise, managers can look for patterns of impacts resulting from climate changes and provide guidance regarding appropriate responses and promote information transfer throughout their organizations. In addition to their work with local residencies, central staff can help ensure that executive agency management is aware of and in support of efforts such as interagency communications and permitting needs that may be required to improve system resilience.

Specific Maintenance Actions

Maintenance forces should consider the following types of actions:

- Consult with designers about more durable materials and designs (e.g., paints, paving materials, drainage features) with consideration for likely future conditions (e.g., higher temperatures, increased rainfall intensities).
- Changing equipment needs due to expected increases in emergency response. It may be increasingly necessary to secure the type of equipment commonly needed in emergencies such as loaders and excavators with "thumbs" for handling woody debris and mobile stock piles of traffic control devices (e.g., cones, signals, signs).
- Stand-by contracts to increase response capacity and shorten reaction times. These contracts may take the form of dedicated response contracts or the addition of "where and when" provisions in all standard and specialized contracts let in connection with the agency's capital program. For example, in preparation for severe weather, ALDOT has local contracts on hand in the case that immediate assistance is necessary.[485]
- Increased identification and monitoring of erosion and sedimentation issues as rainfall intensities increase and climatic conditions change, putting additional stress on ecosystems that evolved under a different climate regime. Consult with designers about need to strengthen both temporary and permanent erosion control best management practices and stream bank protection and scour protection designs.
- Improved weather information systems (sometimes technically known as Road Weather Information Systems [RWIS]), typically employed in snow-belt states, may be applied for year-round use to monitor precipitation and flooding.
- Greater cross-training of staff, perhaps from across the agency, so that the ability to adapt and mobilize for emergency situations is enhanced.
- Stockpiling of materials (e.g., culvert pipe, temporary bridge components, fuel, stone armour) and equipment (e.g., generators, chain saws, traffic control devices). ALDOT stages materials and supplies in different locations in the greater Mobile area, some of which are outside of the storm surge inundation zone, to allow for access even in extreme events.[486] One pipelines operator noted that after a particularly damaging event, all pipelines companies

[485] Powell and Reach, 2012
[486] Powell and Reach, 2012

may be affected, resulting in a high demand for parts. Therefore, some pipeline operators in the region stockpile enough parts to at least temporarily maintain operations while they wait for more permanent solutions.[487]

Permissions, permits, approvals, and contracts as may be needed for debris disposal in the wake of a major storm event.

General Operations Actions

Being responsible for safe and efficient traffic flow, operations has to be able to detect problems through situational awareness, communicate with travelers, and direct other system responses as needed. To accomplish this reliably the operations system must be hardened sufficiently so that it will function during extreme events. In addition to this, the data management capacity of operations can provide valuable information (e.g., flooding locations, tree damage, fog occurrence) that maintenance forces can use as input for the development of their planned activities.

Specific Operations Actions

Some specific recommended operations actions based on current practice include:

- Develop and test a “play book” for emergency operations and, in particular, evacuation protocols.
- Include key stakeholders (e.g., the state emergency operations agency, police, fire, schools, hospitals, government personnel agencies) in routine information dissemination so that all will be in sync during an emergency.
- Cross-train operations staff with maintenance staff to ensure a smooth working relationship has been established before emergency events.
- Harden communications and power systems for emergency use.
- Supplement and adapt ITS resources for disaster monitoring and response.
- Provide detours and signage as may be needed for evacuation.

O&M Adaptation Actions at the Transportation Agency Management Level

O&M activities are carried out in accordance with agency management level policy as specified in budgets, support, and direction. Projects funded by the capital program can help improve climate resilience and decrease system vulnerability, while invariably competing with maintenance funding that can also improve resilience. Funding sources vary, but as a rule both state and federal funding can be used for either purpose. The analysis of capital versus O&M trade-offs (e.g., replacement of a few culverts with capital funds verses the cleaning of many

[487] Powell and Reach, 2012

culverts under O&M at the same cost) is best done using data driven asset management / risk assessment methods.

While day-to-day O&M activities typically attract little attention, O&M often becomes the center of attention during extreme weather events. Some specific O&M activities that can be undertaken to prepare in advance of an extreme weather event include:

- Conduct planning, design, and construction in accordance with the future demands of O&M. A simple and expedient way to insure this is to require O&M signature approval on contracts and plans that affect a particular district.
- Promote cross training and integrated emergency response both throughout the agency as well as multi-agency training, including ICS and NIMS training. Emergencies quickly become an "agency problem" and the response to them influences public perceptions of the entire agency. If the rest of the agency is not properly trained and equipped, response capacity is effectively limited to O&M staff and the resources they have ready to go. Other departments within an agency can play a role during emergencies relieving some of the burden on the maintenance staff. For example, engineering departments are typically well equipped to conduct activities such as damage assessment and perform best when clearly assigned this responsibility as part of the agency's overall mission.
- Foster integrated interagency relationships with state-wide emergency operations staff, other transportation organizations, first responders, etc. Note that this may be most effective when done at an executive level.
- Design GIS and other information systems to improve the agency's awareness of and response to a changing climate.
- Require tabletop exercises, drills, and scenario development for extreme events.
- Require after-action reports with recommendations for improvement to preparation and response efforts. As applicable, the after-action report should be a coordinated, integrated, multi-agency response in order to capture recommendations from a wide range of skills and capabilities.
- Utilize the knowledge and perspective of the residencies and operations offices in formulating agency-wide climate adaptation action plans.
- Work with local colleges and universities to incorporate maintenance engineering courses into the curriculum.
- Fund, support and equip O&M to handle an increasingly difficult role in adaptation to extreme weather while the competing demands of infrastructure aging and increasing traffic volumes continue to grow.

Design asset management systems and use capital funds to foster improvements in system resilience as described in Table 60.

Table 60: Climate Change Monitoring Techniques and Adaptation Strategies for Transportation Asset Management (TAM) System Components[488]

TAM System Component	Monitoring Technique(s) / Adaptation Strategy(s)
Goals and Policies	• Incorporate climate change considerations into asset management goals and policies, either through general statements concerning adequate attention of potential issues or targeted statements at specific types of vulnerabilities (e.g., sea level rise)
Asset Inventory	• Mapping infrastructure assets in vulnerable areas and responses by maintenance work order (potentially using GIS) • Inventory critical assets that are susceptible to climate change impacts to determine which may be in need of changes in operations or more proactive or frequent maintenance
Condition Assessment and Performance Modeling	• Monitor asset condition in conjunction with environmental conditions (e.g., temperature, precipitation, winds) to determine if climate change affects performance, incorporating risk appraisal into performance modeling and assessment – it is commonly the low intensity, high frequency weather events that contribute to failure rather than extreme events causing catastrophic failure • Identification of high risk areas and highly vulnerable assets through engagement of maintenance forces • Use of “smart” technologies to monitor the condition of infrastructure assets
Alternatives Evaluation and Program Optimization	• Include alternatives that use probabilistic design procedures to account for the uncertainties of climate change • Possible application of climate change-related evaluation criteria, smart materials, mitigation strategies, and hazard avoidance approaches
Short- and Long-Range Plans	• Incorporate climate change considerations into activities outlines in short-and long-range plans • Incorporate climate change into design guidelines • Establish appropriate mitigation strategies and agency responsibilities
Program Implementation	• Incorporate appropriate O&M-related climate change strategies into program implementation • Determine if agency is achieving its climate change adaptation / monitoring goals
Performance Monitoring	• Monitor asset management system to ensure that it is effectively responding to climate change • Consider use of climate change-related performance measures • Establish “triggering” measures to identify when an asset or asset category has reached some critical impacted level that requires O&M

488 Meyer, Amekudzi, and O’Har, 2010

Utilizing Federal Recovery Funding

When extreme weather events extensively damage the highway systems (as a rule, causing damage totalling more than $700,000), federal aid can be made available to assist with eligible expenses. Recovery funding is typically available for system recovery and restoration through the Federal Highway Administration's Emergency Relief (ER) Program[489] or the Office of Homeland Security's FEMA.[490] Funds are typically initially advanced from other local / state funding sources as needed and later reimbursed through these federal recovery programs. By necessity, both of these programs must cover a wide range of circumstances and require specific documentation and processes for qualification as set forth in FHWA's *Emergency Relief Manual*[491] and FEMA's *Debris Management Guide*.[492]

Federal reimbursement is an integral part of most emergency response plans used by O&M organizations and is best planned in cooperation with the cognizant agencies in advance of any disaster. Note that O&M activities are not always performed immediately following events, but sometimes need to wait for funding or response capacity to become available. For example, port operators in Mobile noted that heavy precipitation can increase runoff and erosion that in turn increase dredging requirements. However, while dredging needs increase with increased precipitation, the frequency and timing of dredging depends on other factors, including budget availability.

Adapting O&M Activities in Mobile

Interviews conducted with state and local officials in Mobile provided a better understanding of the potential impacts of climate change on agency operations in the project study area.

Overview of O&M Departments in Mobile

The City of Mobile has two primary departments related to transportation, Engineering and Public Works, that support O&M activities (including demand activities and extreme weather events). The City Engineering Department consists of about 25 staff members with a primary role in supporting design and development of capital projects and providing technical support to public works. Public Works takes the lead in addressing short orders (e.g., cleaning inlets, fixing potholes). With the help of the City's GIS Department, the Engineering Department is beginning to build asset information on what was funded and fixed. Service requests, for example, can be plotted in GIS and tied to an address.

Mobile County Department of Public Works maintains all County-owned facilities, including an airport on Dauphin Island, 1,379 miles (2,219.3 kilometres) of roads, drainage facilities

[489] FHWA, 2012a
[490] FEMA, 2008
[491] FHWA, 2013
[492] FEMA, 2007a

associated with these roads, bridges, and traffic control devices.[493] In general, the County addresses different O&M needs based on different characteristics of transportation infrastructure (e.g., residential or urban classification of road).

The local ALDOT Maintenance Bureau has responsibility for four southern counties and a fleet of about 140 trucks in the Mobile area. ALDOT is building a new asset management system that will have information on condition (A to F) and other characteristics of each asset. The new system is also expected to be available to cities and counties. ALDOT work orders (assigned work) are generally not required with the exception of any bridge maintenance activities, which are monitored in the Alabama Bridge Information Management System (ABIMS). Work reports (completed work), however, are required for all maintenance activities and can be queried in ALDOT's asset management system.[494]

The operations and engineering department participates in all levels of project review (i.e. 30% design completion; 60% design completion; and plan, specification, and estimate review at 90-95% design completion) and has different standards for culvert designs depending on whether it is an interstate or non-interstate road. Equipment replacement at ALDOT is well-funded and equipment gets replaced on a periodic cycle (defined by years or miles driven). Equipment is viewed as a primary asset and regular replacement has helped reduce maintenance costs.[495]

O&M Adaptation Approaches in the Mobile Area

The Mobile area has the potential to be affected by longer and more intense heat waves, increased intense precipitation events and flooding, higher storm surges, and greater peak wind speeds during the 21st century.

As in most locations, today's maintenance decisions in the Mobile area are often based on month-to-month or day-to-day changes in weather and storm tracks as opposed to anticipated long-term changes. Many O&M actions taken to prepare for extreme events today, however, will yield lessons applicable to future more frequent and / or severe events. For example, sound emergency operations and training have become vital components of O&M programs in this region. Emergency response and operations training occurs annually and is embedded in the work culture. In the event of an emergency, response operations become a collaborative effort among the city, county, and state departments of transportation. Activities can cross jurisdictional boundaries and are National Incident Management System (NIMS)[496] compliant. A variety of O&M extreme weather preparedness actions are underway across ALDOT, Mobile County, and the City of Mobile; these actions will also help prepare for climate change.

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[493] Mobile County Public Works, 2013
[494] Powell and Reach, 2012
[495] Powell and Reach, 2012
[496] NIMS, the National Incident Management System, is the organizational system used during emergency events to facilitate coordinated response, communications, and command. See FEMA (2013b) for more information.

The Alabama Department of Transportation

ALDOT is divided into nine different divisions. The Mobile Area is part of the "Southwest Division" which covers the following three districts: District I – Mobile County, District II – Baldwin County, and District III – Conecuh & Escambia Counties. At ALDOT, emergency management has recently become a full time staff position within O&M. As a result, ALDOT has improved and strengthened its relationship with the Alabama Emergency Management Agency and conducts recurring training (e.g., hurricane evacuation exercises) in its divisions and districts. Communication plays a key role in emergency operations at ALDOT. The focus has been on specialized communication equipment that can function independent of cell service in the event cell towers are down to maintain coordination across and between divisions during an event.

Technologies used or considered for use have included:

- Portable Highway Advisory Radios (HARs)
- Satellite phones (under consideration)
- Cameras
- Detection devices
- Dynamic message signs
- Web pages dedicated to road conditions to alert communities (one staff member is dedicated to this)
- A 511 traveler information service that is under development and anticipated to be implemented in January 2014
- Social media (has been considered but not generally used yet due to challenges in QA / QC)

ALDOT also stations equipment and supplies at different locations around Mobile to help speed up how quickly equipment can be deployed and be prepared if one location is inaccessible.[497]

Mobile County and the City of Mobile

Mobile County and the City of Mobile face the same climate challenges faced by ALDOT, but the context and scale of their response differs. Critical infrastructure (e.g., drainage structures) is generally older (often decades older) than that found in the state highway system.

The county and city also must both operate under restricted budgets. The County is constrained by limited funding for equipment repair and replacement, so it is difficult to keep infrastructure in a state of good repair. County crews work four day / 10 hour weeks (from 6:30 am to 4:30 pm

[497] Powell and Reach, 2012

daily), making some of the existing flexible operations options to address extreme heat events difficult (e.g., starting or ending work crew days earlier, providing breaks) without change.[498]

The City of Mobile faces additional challenges associated with working in crowded neighbourhoods and uniquely municipal problems such as litter or debris removal, mature tree preservation, power outages, access, aesthetics, density of infrastructure, and community sensitivity. Further, the economies of scale present in a statewide system in the development of asset management systems, traveler information, worker training, purchasing, equipment, engineering, etc. are simply not available at the municipal level in Mobile and elsewhere.

The following broad-based adaptation planning, preparedness, and recovery strategies were observed in the Mobile area and have relevance to other jurisdictions:

- Run operations like a business. With limited funding, it is important to know which assets or projects are most critical to the safety, reliability, and performance of the system. When possible, develop regular replacement cycles to reduce maintenance costs across all assets.
- Pre-position contracts. This includes contracts for reimbursements from the FEMA and for other contractor work. ALDOT has these vehicles in place for concrete, erosion control, traffic control, striping, etc. where the contractor charges a set rate based on what they bid on plus materials and equipment.
- Position emergency equipment in different locations. Prior to a hurricane or extreme weather event, position equipment like backhoes and chainsaws in different areas, away from locations vulnerable to storm surge for quick deployment. If one location becomes inaccessible, defer to remaining locations to access emergency equipment.
- Maintain good organizational relationships. Communicate well and communicate often. Coordination within the agency, between and across agencies, departments, jurisdictions (e.g., municipal, county, metropolitan planning organization, state), and levels of leadership is necessary. Develop coordination and communication plans and ensure that reliable communication devices are available in the event of an emergency or extreme weather event.
- Drill and train thoroughly and often to insure performance when it’s most needed.

Table 61 highlights specific adaptation strategies that are underway or under consideration by ALDOT, Mobile County, the City of Mobile, and others to adapt to projected changes in climate in the Mobile region.

Conclusions

With the National Highway System essentially “built out,” O&M activities will play a critical role in efforts to adapt to a changing climate. O&M organizations have always contended with weather-related impacts and adapted to changes in infrastructure conditions, traffic volumes, and regulatory landscapes. Their mission to keep the system in a state of good repair and respond to incidents and emergencies, combined with annual planning and budgeting for O&M activities,

[498] Mitchell and Sanchez, 2013

and everyday consideration of weather impacts in decision making will put O&M staff on the front lines in ensuring the climate resilience of transportation systems. The reality of fixed and finite budgets and the press of other demands besides climate change can make it exceedingly difficult for O&M organizations to be as proactive as they should or would like to be. Advanced asset management systems that integrate climate change monitoring help prioritize the integration of adaptation into O&M activities. O&M personnel in the Gulf Coast region have effectively coped with unique and continuing challenges from extreme weather and climate change and have noted the importance of cooperation and preparation that can help other locations in addressing climate change.

Table 61: Summary of Climate Stressors, Associated O&M Impacts, and Adaptation Strategies Applied or Under Consideration in the Mobile Area

Climate Stressor	Potential O&M Impacts	O&M Adaptation Strategies Applied or Under Consideration
Precipitation Induced Flooding	• Temporary travel disruptions and facility damage	• Stand by and dedicated response contracts to increase capacity • Prepositioning of pumps, supplies, and materials • Regular clearing of drainage ways • Floodway management • Planned detours and response per NIMS
Extreme Heat	• Worker exposure to high heat and possible dehydration • Direct infrastructure impacts - pavements, structures • Stress on equipment	• Have maintenance crews switch to an earlier start time in the summer months • Schedule worker activities to limit prolonged periods of time outdoors • Provide more frequent breaks (e.g., 10-15 minute breaks every hour) • Provision of electrolytes in addition to water • Keep equipment in good repair
Hurricanes / Storm surges	• Direct storm surge inundation of roadways , bridges, tunnels causing temporary travel disruptions, and facility damage • Bridge scour leading to closure of bridge • Loss of utilities including natural gas (affecting pumps and lift bridge) and electric power (causing loss of communications, signal systems, and pumping capacity) with associated travel disruptions.	• Permissions, approvals, procedures and contracts as may be needed for debris disposal and reimbursements • Preparation per NIMS guidance • Position materials and equipment, do staging outside damage area • Plan for FEMA reimbursement • Keep pumps in good repair • Plan for storm proofing tunnels • Harden communications • Have a disaster debris management plan
Wind	• Downing of trees and power lines causing safety hazards and travel disruptions • Breaking of limbs that clog drainage ways and lead to flooding • Breaking of traffic signal wires leading to travel delays • Breaking signs leading to safety hazards and navigation difficulties	• Keep trees trimmed as practicable • Be ready to respond to debris clearance needs • Prepare backup generation and make sure it is ready to go for offices and signals • Put traffic signals on mast arms. • Strengthen sign hardware as it breaks • Supplement and adapt ITS resources to assist with disaster monitoring and response

5. Lessons Learned

5.1 Introduction

The case studies in this report examined the potential impacts of climate change-related stressors on different types of transportation assets in Mobile, Alabama. The case studies hone in on components that are common to transportation facilities across the country (e.g., culverts, abutments, embankments, pavement surfaces, and bridge structures) and across transportation modes. The primary intent of this effort was not to conduct exhaustive technical analysis; rather these case studies aim to establish and demonstrate engineering processes that allow for incorporation of climate change and extreme weather risks into asset-level decision making.

This section summarizes the lessons learned from the case studies presented in Section 4. These "lessons" range from general observations of the process used to determine design values for key input variables (such as expected values of rainfall intensity or storm surge heights), to more specific conclusions relating to the actual design process for particular assets. The lessons learned are based on analysis of specific transportation facilities and projected environmental conditions in Mobile; thus, the engineering design recommendations raised are specific to the site where the facility is located. That said, the lessons learned in this study suggest that strategies are available to help overcome the challenges associated with uncertainties of future climatic conditions; such strategies are transferrable to other projects, locales, and risk management contexts.

The remainder of this section describes:

- Lessons learned with respect to the *General Process for Transportation Facility Adaptation Assessments* (the *Process*), which was the overall approach adopted for the consideration of adaptation strategies on existing assets. The *Process* was also the organizing structure for the case studies.
- Lessons learned on the process for determining the values of key variables used in engineering design that will likely be affected by changing climatic conditions.
- Summaries and findings for each of the facility-specific case studies conducted as part of this assessment.

5.2 Applicability of a General Process for Transportation Facility Adaptation Assessments

Two key lessons emerged as engineers and experts in the kinds of transportation facilities attempted to conduct detailed assessments of facility specific vulnerabilities and adaptation options. The first was the need for some systematic approach or process for incorporating climate and weather risks into standard engineering assessment methodologies. The second was the need for some guidance or examples that practitioners could use to understand and implement this new process.

In recognition of these lessons, the assessments conducted for this study followed a new approach (the *Process*); this served as the organizing framework for each case study assessment and ensured consistency across disciplines and facilities. As described in Section 3, the *Process* incorporates the uncertainties associated with future climatic conditions into an 11-step framework. The *Process* identifies likely vulnerabilities of individual transportation facility assets to climate change and extreme weather variables, examines different adaptation options in light of expected future conditions, and proposes planning and engineering solutions. Importantly, the *Process* was developed as a generic approach to the engineering design of different types of assets under a range of climate change-related variables. The types of design-related questions the *Process* was developed to consider:

- How might environmental conditions change during an asset's design life?
- Will the changes be significant enough to adversely affect the asset?
- What type of adaptation options are available and are they effective?
- If effective, are they cost-effective given the adverse impacts?
- At what rate will changes in climate occur and how may the changes influence the timing of a response?
- How can alternatives be evaluated and/or pursued given the large uncertainties involved in projections of future climate?

Critically important from the standpoint of transferrable lessons, the *Process* does not change the basic variables and design input relationships and procedures that are common to engineering design throughout the United States. The *Process*, as implemented in each case study, does customize the following:

- The values of the climate inputs used in the design methodology;
- The recommended number and type of design options one develops;
- The thought process used to select the final option, which balances cost-effective and resilient transportation services with potential risks.

In particular, three of the steps in the *Process* are relatively new additions to the typical approach toward engineering design:

- Step 4 – Decide on Climate Scenarios and Determine the Magnitude of Changes
- Step 6 – Identify Adaptation Options
- Step 7 – Assess Performance of the Adaptive Design Option(s)

Step 8 – Conduct an Economic Analysis, while often performed on major projects today, takes on great significance in the *Process* by providing a tool for aiding decision-making in the context of uncertain future conditions. The case studies illustrate how these steps can facilitate incorporation of future climate risks in engineering design.

Judgment will be required by the decision makers as to what assets warrant the use of all or any of the steps in the *Process* when considering improvements to a specific asset or facility. The decision should consider the criticality, the vulnerability, the consequences of failure, and the remaining design life of the asset or facility when determining the design approach.

5.2.1 Lack of Engineering Guidance or Reference Materials for Considering Climate Change

One of the important lessons learned from the engineering case studies is that the 11-step Process can be successfully applied across different types of assets and for a range of climate-change stressors. In fact, the Process was specifically developed to generate consistency among various engineering disciplines working on this project. The Process can therefore serve as an organizing framework for how engineering design can be undertaken considering the uncertainties associated with possible future environmental conditions.

There is an important need for additional guidance on how engineering design can be undertaken given uncertainty about future conditions. Very little guidance and few reference materials are available to provide suggested approaches for considering climate change-related uncertainty in the engineering design process. In developing many of the engineering case studies, considerable discussion and debate occurred among the designers representing different engineering sub-disciplines (including structural, hydraulic, geotechnical and pavement) on the most appropriate approach for analyzing a particular asset given expected future loads and stresses. Engineering practice and indeed engineering culture is focused on research and statistical analyses of historical events (rainfall, extreme heat, etc.); these data provide the required input variables used in decision-making. The uncertainty associated with future input variables that are derived from climate model projections drives the need for a new approaches to develop those input variables and consider their effects when making planning, design, and operations/maintenance decisions.

In recognition of this need, the *Process* was developed to be general enough to be applied to multiple transportation modes and asset types. It can also be used both for *existing* facilities, where adaptive retrofits might be considered, and for proposed *new* facilities where adaptation measures can be incorporated into the design. A rather informal, but important lesson from this study is the cultural difficulty associated with asking engineers to embark on an analysis in the absence of these accepted practices and guidelines. The engineering profession relies on official guidance or design guidebooks for the design process – in part to set a standard of care, facilitate design process consistency and limit potential professional liability based on engineering-supported decisions. In cases where an engineer recommends a design exception based on strong evidence of potential changes in climatic conditions, the exception would require careful consideration and strong justification. Currently, accepted procedures to back up such exceptions are lacking.

5.3 Developing General Input Values for Engineering Designs

A design process that reflects projected changes in climatic conditions has to account for possible changes in the input values of the design variables beyond simply relying on historical data. This is a significant shift from standard engineering design practice.

Engineering designs for transportation facilities rely on a determination of the stresses and loads that facility components will likely face. Identification and determination of the stresses and loads is thus critical to selecting designs that will provide durable and stable asset performance. The engineering profession has developed design procedures and methods that are based on years of experience and documentation of the relationships between load/stress input variables and the resulting design characteristics. For example, one of the most used formulas for determining flow rates for the design of facilities handling rainfall runoff is: $Q = (C)(i)(A)$, also known as the Rational Formula. In this formula, Q is the flow rate, C is a runoff coefficient representing the degree to which ground cover is impervious to water seepage, i is the intensity of rainfall, and A is the drainage area. Thus, the design of a culvert, which is based on the amount of water that is expected to pass through (Q), will depend on the rainfall intensity (i) falling in the area draining into the culvert (A), reflecting the ability of the water to be absorbed by the ground between where it falls and its arrival at the culvert (C).

Traditional engineering design would use historical data for the values of i. In the context of future climatic conditions, however, one might expect that rainfall intensity would change in ways that are not simply an extension of past trends, but may occur at higher/lower rates of intensity more often than has been noted historically. The ground cover runoff coefficient factor might change as well, depending on the location, due to drought, pests, fire, invasive plants due to higher temperatures and/or erosion caused by increased runoff. Thus, a design process that reflects projected changes in climatic conditions in the future would have to account for possible changes in the input values of the design variables beyond simply relying on historical data; this is not unlike designing structures to withstand seismic risks.

5.3.1 Utilizing Climate Data in Engineering Assessments

The engineering case studies illustrated the need to provide input data at a scale necessary for design purposes. This study developed data at the temporal and spatial scale needed to conduct engineering design at the project level but many input variables remain to be translated to useful metrics.

When considering future climate, it is necessary to consider the types of projections that are available and the drivers underlying those projections. In this and most similar studies, the analysis of future climate conditions was predicated on the emission scenarios offered by the IPCC. The IPCC offers a range of emission scenarios that are then used as inputs to multiple global climate models. Importantly, the IPCC states that each of the emissions scenarios are equally likely to occur;[499] thus, while not providing a different conclusion on any one trajectory of future climate, the climate projections derived from those scenarios are useful in providing a range of outputs that can be considered as a sensitivity test in the planning and design of transportation facilities. A range of variables has been developed that can be used and parameterized in design decisions; some climate data may be found to have no bearing on design while other data may have wide-ranging consequences. The case studies demonstrate that the values of the design variable inputs can have strong influences on expected stresses and loads on transportation assets, as well as on appropriate adaptation response.

The engineering case studies illustrated the need to provide input data at a scale necessary for design purposes. This has been a challenge noted for many years and an identified gap in the application of climate scenario data in engineering design. This study developed data at the temporal and spatial scale needed to conduct engineering design at the project level. Such data were derived from the best climate modeling results available for the region, as well as from assumptions on the best approaches for providing that data that could be used in engineering design.

Earlier tasks of this study included development of climate information (see the Task 2 reports) from which climate narratives were developed (see the Task 3.1 report).[500] “Warmer” and “Hotter” narratives were developed to describe ranges of temperature values, and “Wetter” and Drier” narratives of precipitation projections were developed for the Mobile study region for use in the engineering case studies.

The scenario approach produces a range of values for the input design variables. Depending on the environmental stressors being considered, many of the engineering case studies showed that the scenarios defined by the lower ranges of design input values had either little or no impact on the current design of the asset, or that the impacts could lead to some corrective design action. For those with no impact, the original design of the asset provided enough strength and durability to withstand the forces that were likely to be placed on the asset assuming climate change-induced design values. In other cases, and this was true especially for storm surge, the assets were found to be vulnerable for all scenarios. In fact, with storm surge, in some cases the lower scenario was actually found to be *more* impactful than the higher scenario. This lends further credence to the importance of including lower end scenarios in an adaptation assessment.

5.3.2 Addressing the Design Storms vs. Modeled Future Storms

Rooting future scenarios in the experience of a single historical weather event and then altering characteristics to reflect possible future permutations, has the benefit of providing very relatable results to local stakeholders, especially if a severe storm event occurred recently. However, this

[499] IPCC, 2007
[500] USDOT, 2012; USDOT; 2014

does not allow for the calculation of a return period which presents a challenge when comparing future asset performance against a design standard rooted in return periods (e.g., no overtopping is allowed up to the 100-year storm).

The sea level rise and storm surge scenarios we developed and applied in this study were compiled after coordination with TAC members as well as local stakeholders, and reflect an interest in grounding consideration of future storms in the first-hand experience and lessons learned from Hurricanes Katrina and Georges. Thus, the scenarios considered a number of potential scenarios for storms, including:

- Hurricane Katrina historic storm on its observed path
- Hurricane Georges historic storm on its observed path
- Hurricane Katrina with a shifted path, with its eye making landfall west of Mobile
- Hurricane Katrina with a shifted path, and higher sustained winds

Selected surge and wave models were altered to reflect sea level rise scenarios associated with climate change. These combinations of storms and sea level rise were a subset of the available information generated in earlier tasks, and represented a wide range of outcomes for consideration in the engineering phase of this work. The engineering assessment aimed to understand potential surge impacts on local transportation facilities in the future if a similar storm were to strike again with climate change. Rooting future scenarios in the experience of a single historical weather event and then altering characteristics to reflect possible future permutations, has the benefit of providing very relatable results to local stakeholders, especially if a severe storm event occurred recently.

Engineering practice is based on the premise of "acceptable risk"; acceptable risk is addressed in design standards that hinge on the recurrence probability of storm events (e.g., a bridge designed to pass the 100-year storm with a 1% annual chance of occurrence). Design storms or return period storms are thus a surrogate for an agency's risk tolerance and reflect a trade-off between the costs of providing additional protection to a facility and the likelihood that the event being protected against will actually occur.

As the engineering analysis progressed, the dialogue on surge efforts focused on whether the existing methodology of using the manipulation of a single storm event would be preferable, given that typical derivations of the design storm include a fuller spectrum of possible storm events. It was concluded that the applied method did not provide for the generation of return period probabilities necessary for a probabilistic analysis, but presented options for assessing impacts for some selected and relatable storms. This presents a challenge when applying the surge projections to some engineering assessments because comparing future asset performance against a design standard rooted in return periods (e.g., no overtopping is allowed up to the 100-year storm) is not possible. Understanding performance relative to a return period storm is important to established engineering practice. It is also key to fostering a dialogue on risk because without some projection of how event probabilities may change over time no formal assessment of benefits and costs can be made in the manner that was done for the culvert case study.

Defining the storm surge scenarios based on different greenhouse gas emissions scenarios and determining changes to surge elevations of the design storm of interest (e.g. the 100-year storm) under each scenario would address this challenge. This was the same general approach employed on the culvert study for precipitation depths whereby a future 100-year storm precipitation depth, for example, was generated and compared to today's value. That said, as noted in Step 4 of Section 4.4.4, generating accurate future surge return periods could be a very expensive undertaking that will require considerable modeling efforts and statistical analysis on par with those employed by FEMA to develop the current return period values. Such analyses are likely to be well beyond the resources available for many individual transportation projects and will likely need to be done as part of nationwide, state, or regional undertakings by FEMA, USACE, NOAA, or other pertinent agencies. As such this information is not likely to be available for the foreseeable future for transportation project design, and a simpler alternative approach will likely have to suffice in the interim.

One simpler approach could involve adding sea level rise to the existing return period surge elevation of interest and deriving a new return period event in that manner. However, this "bath tub" approach would need to acknowledge that it only considers the sea level rise component of climate change impacts on surge (changes to storm intensity and frequency would not be accounted for, as there is less understanding of climate's impacts on these factors) and it does not account for the non-linearities that changes in water depth engenders on surge elevations and wave heights. A somewhat more sophisticated approach that would address the aforementioned non-linearity issue would involve identifying the elevation of the current return period event of interest and then conducting localized surge and wave modeling of that event coupled with sea level rise. The process used in this study – based on specific historic events combined with alterations to reflect projected future conditions – was the best option available, but additional research and dialogue to identify and refine future surge scenarios for use by engineers in design is needed.

5.3.3 Downscaling Climate Scenario Data for Engineering Design

The engineering case studies illustrated the need to provide input data at a scale necessary for design purposes. This study developed the best available projections at the temporal and spatial scale needed to conduct engineering design at the project level, subject to the limitations inherent in developing projections of future temperature and precipitation values relevant to engineering design.

Obtaining data at the appropriate scale has been a challenge noted for many years and an identified gap in the application of climate scenario data in engineering design. In some cases, the results of climate models are appropriate to do engineering design. For example, the design of large culverts can use 24-hour precipitation values that are generated from climate models. In other cases (e.g., the design of small culverts) engineers need precipitation data that occurs in shorter time periods, which has traditionally been represented with intensity-duration-frequency (i-d-f) distributions with less than 24-hour values. Global climate models and regionally downscaled projections can provide hourly values, but often this data is summarized and made available at the daily, weekly, monthly or seasonal level of aggregation to provide a greater level of robustness.

Further discussion of the processes identified here and other methodologies for applying derived climate data will be of great value in developing future values for precipitation and temperature at the level where engineers can apply them at the project level.

5.3.4 Considering the Full Range of Climate Change Scenarios

It is important for a robust design process that a range of climate change-related variables be considered, simply to make sure that even the lower estimates do not require corrective design action, and that a reference alternative is presented for the scenario analyses of the higher stresses on the assets. Additionally, in some cases the lower scenario was actually found to be more damaging than the higher scenario.

The scenario approach produces a range of values for the input design variables. Thus, for example, the "Warmer" narrative produced a lower temperature average and a different range in high/low temperatures than the "Hotter" narrative. Similarly, the scenarios relating to storm surge resulted in different surge heights.

Depending on the stressors being considered, many of the engineering case studies showed that the scenarios defined by the lower ranges of design input values had either little or no impact on the current design of the asset, or that the impacts could be addressed through corrective design action. For those with no impact, the original design of the asset provided enough strength and durability to withstand the forces that were likely to be placed on the asset assuming climate change-induced design values. For example, the pavement design case study determined that under the "Warmer" narrative, temperature changes were not great enough to require any adaptation from current practice through 2100.

In other cases, and this was true especially for storm surge, the assets were found to be vulnerable for all scenarios. In fact, with surge, in many cases the lower scenario was actually found to be *more* damaging than the higher scenario. In many cases it is the wave action that is likely to cause the most damage to a facility and once a facility is submerged (which is more likely in a higher surge scenario), the wave impacts are actually reduced. This was the case, for example, in the studies on the ramp between the Causeway and the Bayway Bridge and on Dock One at McDuffie Terminal. This lends further credence to the importance of including lower end scenarios in an adaptation assessment.

5.4 Facility-Specific Lessons Learned

Engineering design practices are very specific to different asset types, and the considerations of environmental factors in engineering design will reflect the site-specific context. In this study, we have focused on example facility-impact relationships in order to understand those specific relationships and adaptation options. However, in analyzing adaptation options for a specific facility, it is important to consider all impacts a facility might face, not just for example SLR or surge or precipitation-induced flooding. The following sections describe the lessons learned for the different assets.

5.4.1 Culvert Exposure to Changing Precipitation Patterns

This case study of the Airport Boulevard culvert over Montlimar Creek demonstrated how a large culvert can be analyzed for a projected increase in precipitation. Adaptation options were identified and tested using a benefit-cost framework.

The analysis found that the culvert would not meet ALDOT standards on culvert design[501] under the future precipitation levels assumed for this analysis. That is, under feasible future climate conditions, the roadway could be overtopped during the 25-year storm events. To address this problem, the analysis considered two options for increasing the capacity of the culvert: adding one cell on each side of the existing crossing, and replacing the existing crossing with a larger one. Taking into account both the expected performance and the cost-benefit of each option, the study indicates that adding one cell on each side of the existing crossing (Option 1) is the preferred course of action, as it is effective, lower cost, and most likely to be cost-effective. The case study also showed that there are likely to be substantial costs incurred if no adaptation actions are taken to address expected flooding at the culvert.

This case study yielded some important lessons learned that are more broadly applicable to similar analyses. One lesson is that the use of 24-hour duration precipitation projections that are available from standard climate models is an appropriate approach for designing large culverts. The resulting adaptation design for the case study culvert was based on accepted engineering practice for determining runoff rates and resulting headwater elevations. For smaller culverts, however, where 24-hour projections *may* not be applicable, one is more likely to use the Rational Formula (described earlier) with input from intensity-duration-frequency (i-d-f) curves for which downscaled climate model data is not readily available.

This case study also demonstrated how a Monte Carlo analysis can be an effective way to deal with the environmental uncertainties influencing major projects. The analysis simulated thousands of different combinations of precipitation events under five climate/land-use scenarios

[501] ALDOT standards require that the 25-year flood be used for design of waterway crossings on secondary routes such as Airport Boulevard. The standards also require a minimum of two feet (0.6 meters) of freeboard[501] above the design flood stage (relative to the edge of pavement) to keep the water surface an adequate depth below the pavement base. See page 54-55 for more information. (ALDOT, 2008)

and then estimated the resultant flooding costs over a 30-year appraisal period. This approach was very useful in considering future climatic conditions that could affect the performance of the culvert and therefore of the benefits associated with adaptation options.

Another lesson learned is that benefit-cost analyses of adaptation options are greatly influenced by what is included within the bounds of the analysis. When considering only travel time costs associated with road crossings, the benefit-cost analysis supported selection of the less expensive Option 1. However, when one also considers the potential damage to buildings and businesses that could be damaged under a "Wetter" narrative, the benefit-cost analysis supports selection of Option 2, (replacing the culvert) because the additional damage costs more than offset the additional costs associated with building the second option. It is up to the analyst to determine which benefits will be included in the analysis; however, the case study does suggest a need to consider benefits beyond the road right-of-way.

Finally, the case study did not examine the impact of downstream increases to flow and attendant land use impacts associated with increasing the culvert capacity. When such analyses are conducted for a project, additional detailed hydrologic analysis would be required.

5.4.2 Bridge Over Navigable Waterway Exposure to Sea Level Rise

This case study of the Cochrane-Africatown USA Bridge examined whether a coastal bridge could limit navigation on the tidal Mobile River as a result of projected sea level rise. The U.S. Coast Guard requires a minimum vertical clearance of 140 feet. This analysis considered whether this minimum vertical clearance would still be met under the following sea level rise scenarios:

- One foot (0.3 meters) of global sea level rise by 2050
- 2.5 feet (0.8 meters) of global sea level rise by 2100
- 6.6 feet (two meters) of global sea level rise by 2100

The analysis found that the navigation requirements for the bridge would not be violated under the first two sea level rise scenarios. However, a sea level rise of 6.6 feet (2 meters) by 2100 would, over time, reduce vertical clearances such that navigation is impeded. In addition, bridge structural corrosion might be accelerated, and in some cases, the bridge itself (or its approaches) may become permanently inundated. On a broader scale, the case study showed that there are many options that could be considered to adapt to rising sea levels. Some options focus on structural changes to the bridge, such as raising the bridge deck, redesigning for a thinner deck and shorter spans, or retrofitting the bridge with moveable spans. For bridges with navigable spans that might cause significant navigational impacts due to sea level rise during their design lives, future bridge rehabilitation analyses should consider options to reduce navigation clearance-related impacts such as elevating the deck. Investigation of how substructures can be extended, modified, or completely rebuilt will need to be done at the time of rehabilitation on a case-by-case basis. Future modeling should be considered to ensure that whatever option is chosen takes into account the latest sea level rise trends. In cases where the design life of a bridge is short enough that expected sea level rise would not affect it during its design life, it might very well be the case that an eventual full replacement of the existing structure with a design that accounts for anticipated sea level changes will be a more cost-effective solution than retrofitting the existing structure now.

Other options relate to the broader use of the river, and the infrastructure around it. For example, it could be accepted that vertical clearance would be gradually reduced and larger vessels would not be able to navigate past the bridge in the future; ports requiring access to larger vessels could eventually be relocated downstream, or routes with lower bridges could eventually be located upstream of ports.

As sea level rise is a relatively gradual phenomenon (even considering its projected acceleration after mid-century), time will allow for continual evaluation of changes in actual sea levels. Monitoring of sea level rise trends compared to design life depletion rates should be conducted to help determine the optimal scheduling of an adaptation solution. As 2100 approaches, it might be that the actual sea level rise is not near the assumed analysis value and that no adaptations will be required. In the case of the Cochrane-Africatown USA Bridge, the likely trend in sea level rise might become apparent by mid-century and a decision could be made then on how to proceed.

Given the wide range of feasible adaptation options, port and transportation planners should begin monitoring sea level rise as its potential constraints on navigation as soon as possible. It may be decided that immediate action is not needed, but understanding future constraints could be factored into decisions related to siting of port facilities and upcoming bridge rehabilitation processes. Continued monitoring will allow decision makers to reassess their selected course of action as better information on future sea levels becomes available.

Finally, the case study did illustrate that the *Process* is broadly applicable to bridges across the country where sea level rise has an influence. Bridges over navigable channels would need to be investigated for clearance reductions due to sea level rise and determine if any remedial action would need to be implemented. Ultimately, this effort is best handled at a planning level in a coordinated manner amongst all bridges along a shipping channel. Adapting one bridge to accommodate sea level rise without consideration of adapting other bridges along the channel may impede access.

5.4.3 Bridge Approach Embankment Exposure to Sea Level Rise

This case study of the US 90/98 Tensaw River Bridge western approach showed how sea level rise effects on wind-generated wave heights and wave impacts could be analyzed for an approach roadway embankment. In particular, sea level rise can contribute to increased wave heights and energies impacting embankments, which in turn can cause increased scouring and erosion, potentially affecting the stability of an embankment.

This case study showed that embankments assessed are susceptible to sea level rise and wind-generated waves. The progression of sea level rise and its impact on sustaining taller waves could present challenges for maintaining the functionality of a roadway embankment. Under each of the climate scenario narratives in this case study, protection and risk reduction measures would be needed for the embankments. Specifically, the roadway and abutments would need to be raised to a height that would significantly reduce the risk of overtopping from wave run-up. As sea levels rise and water depths in front of an embankment increase, the height of waves that can be sustained without breaking prior to impacting the embankment increase, consequently increasing the weight and dimensions of the riprap needed to protect against them. In addition, as the wave size increase, the height at which the run-up impacts the embankment and approach roadway increases as well.

It is important to note that this analysis showed that, under the three sea level rise scenarios considered, the bridge itself was not overtopped but the approach roadway located west of the abutment was susceptible to flooding. This finding is important because it supports other studies that have found that bridge approaches can be far more vulnerable to sea level rise than the main spans. Long before the actual bridge is overtopped, its approaches may be overtopped, making the bridge unusable.

This case studied notes that, when considering protection measures for this type of asset, potential stressors upon the abutment must be taken into account—including storm surge events. This analysis considered only the effect of sea level rise on wave heights and velocities, but sea level rise could also magnify the impact of storm-related surge events.

The general analytical methods demonstrated here can be applied to other coastal embankments, including causeways, coastal roadway embankments parallel to shorelines, or barrier island roads that are (or may become) subject to regular wave impacts due to increases in sea levels.

5.4.4 Bridge Abutment Exposure to Storm Surge

The combination of sea level rise and potentially more intense storm surges enhance the threat of potentially devastating impacts of coastal bridges. This case study examined how a bridge abutment can be analyzed for storm surge scenarios, using the elevated ramp leading from US 90/98 to I-10 (the west abutment of the US 90/98 Tensaw River Bridge) as an example.

The review concluded that while the abutment itself was not designed to be stable under scour conditions, the protection components of riprap, bulkhead, and willow mattress were so designed. The combined considerations for the abutment and the protection scheme showed that the system was stable and capable of performing for the current conditions and each of the projected surge events.

From a methodological standpoint, an interesting finding is that peak velocity of the surge does not coincide with the peak water surface elevation. Due to the bi-directional nature of coastal surges (flood and ebb surge), it was found that the peak velocity occurred at two points, first during the flood surge and later during the ebb surge. The peak velocity for each of these conditions occurred when the rate of water surface elevation change was at its greatest. In this case study, the abutment scour and protection computations were performed for the flood surge peak velocity, the controlling velocity for scour analysis. This would likely be the case for many other scour analyses in coastal environments.

The analysis highlighted the important role that protective features play in the resiliency of an asset. The abutment analyzed in this study was not designed in consideration of full abutment scouring conditions; in lieu of other protection factors, the abutment could be reasonably expected to fail. However, the presence of protective features (riprap, willow mattress pad, and a timber bulkhead) provide enough protection that the abutment would likely be able to withstand the surges analyzed. This finding is neither unique to this bridge design nor unexpected. Given the widely held view that abutment scour equations produce overly conservative results, many state agencies have chosen to armor or otherwise protect abutments from scour rather than design the foundations for the full scour depth. The lesson learned from the multi-protection strategy was that a combination of engineering options can be used to protect against scour from storm surges. Furthermore, inspectors should be informed that even if the structural portion of an abutment is situated on “dry” ground, other components such as bulkhead, riprap, or other stability measures may play a key role in the overall scour resistance of the abutment and should likewise be monitored.

5.4.5 Bridge Segment Exposure to Storm Surge

This case study of the US 90/98 ramp to I-10 eastbound at exit 30 demonstrated how a bridge can be analyzed for potential storm surge scenarios, including where sea level rise has been factored in. Using increasingly severe storm scenarios (with and without sea level rise), the analysis considered whether the bridge was susceptible to three different modes of failure: (1) the superstructure (e.g., deck) is uplifted by waves and washes away, (2) the substructure (e.g., bents, pier caps) fails due to lateral forces from waves, or gets uprooted from vertical forces acting on the superstructure, and (3) the substructure fails due to excessive scour.

The bridge analysis found that both the superstructure and the substructure could fail under the storm scenarios investigated. At specific bents, the superstructure could have bolt failures at the bottom of the girders and lift off and wash away. Meanwhile, the piles have sufficient axial capacity to resist the uplift force on the superstructure (up to the point the anchor bolts on the superstructure fail), but may not able to resist the lateral forces (shear and moment), and could fail due to shear and/or bending.

However, these results are not consistent with what actually occurred during Hurricane Katrina, which caused superstructure damage but did not damage any of the piles. This discrepancy is likely due to the fact that detailed geotechnical data—such as the shear strength parameters and physical properties (e.g., plasticity characteristics, unit weight) of soils—was lacking. The lesson here is that these analyses are very sensitive to the quality and completeness of the inputs. As with any actual project, a full analysis of the site should involve field collection of geotechnical data (i.e., soil borings) if this information is not already known.

When considering ways to make a bridge more resilient to storm conditions, transportation offices could consider a design that would allow the superstructure to break away during a significant storm surge so that the substructure would remain intact. Although bridge would have technically "failed" under this situation, rebuilding the bridge would take less time and be less costly than if the substructure was damaged. In essence, the facility would be designed for a controlled failure if surge and wave forces become too great. A complementary adaptation measure would be to ensure that superstructure design documents be safely stored and made easily accessible after an event, so that the replacement could occur quickly. When considering controlled failure as an adaptation strategy, it would be important to consider the community needs served by the bridge, and whether the community can continue to function well if use of the bridge is temporarily lost. If not, then more aggressive protection measures may be warranted.

This analysis also found that the worst case storm surge scenario does not necessarily translate to the worst effect on the facility because the contours of the ground surface beneath the water body can influence current strength and direction, as well as wave height.

5.4.6 Road Alignment Exposure to Storm Surge

This case study of I-10 (from mileposts 24 to 25) examined how a road or rail alignment can be affected by surge flooding combined with sea level rise. The assessment considered the potential for:

- storm surge to overtop the I-10 roadway and underpasses,
- the I-10 roadway to breach due to overtopping flows,
- flooding to impact the nearby neighborhood of Oakdale, and
- the implications of flow velocities through the underpasses at the three bridge crossings.

The case study found that all of the storm surge scenarios tested present some threat of inundation and erosion to the roads and railroad passing through the underpasses in the study corridor. I-10 would be expected to overtop in the more severe surge scenarios (but not in the Katrina Base surge scenario), and some of the underpasses could flood in all three storm scenarios.

The roadway could breach due to overtopping flows in the two more extreme storm scenarios. It was estimated that the Hurricane Katrina Shifted Scenario storm surge could cause failure of the shoulder lane and four travel lanes on both sides of the roadway. The Hurricane Katrina Shifted

+ Intensified + SLR Scenario storm surge could result in breaching of the entire width of the roadway.

The case study showed that the neighborhoods surrounding the roadway facility would be impacted by storm surge that is funneled through roadway underpasses. The I-10 roadway embankment would not provide a significant barrier to the surge waters entering the Oakdale neighborhood.

The evaluation of the flood velocities through the underpasses indicated that, since all three bridge crossings within the study site have concrete abutments and concrete roadways under the bridges, the majority of the roadway is protected from erosive flow velocities for all storm surge scenarios. However, small sections of the median that consists of soil and grass could be affected by erosive flow velocities; this finding is significant because the bridge piers are located in the grass median. Finally, the rail line under one of the underpasses may be vulnerable to the flow velocities analyzed.

An important lesson from this case study is that additional erosion protection should be considered when designing roadway crossings that could be subjected to reverse flow from storm surges. Certain materials (such as concrete) are less vulnerable to erosive flow (such as soil grass), and the selection of building materials could influence the vulnerability of roadway crossings.

Another important aspect of an analysis of roads in coastal areas is that roadway pavement drainage could be impacted by increased precipitation intensity as well (in addition to the storm surge factors analyzed in this study), where the flat slope of a roadway may not meet the design spread conditions for a given precipitation event because of its location. A roadway drainage system could be impacted by both increased precipitation intensity and by increasing water levels at the system outlet (tail water) due to storm surge. This will likely decrease the ability of a drainage system to handle water flows.

Another important finding is that changing the width of the underpasses may not prevent storm surge from entering a community. The analysis showed that widening the underpasses does not increase the volume of storm surge flows entering the Oakland neighborhood because the existing underpasses are already capable of conveying the storm surge flows inland without significant attenuation. Widening also does not significantly reduce the maximum velocities through the underpasses.

Further, at this specific site, raising the roadway does not help to prevent storm surge flows from entering the neighborhood.

Thus, the adaptation options proposed (widening the underpasses and raising the roadway) do not significantly alter the amount of flooding in the neighborhood.

5.4.7 Coastal Tunnel Exposure to Storm Surge

This case study of the Wallace Tunnel[502] examined a tunnel's vulnerability to storm surge-related flooding during a hurricane and the additional protection provided by various adaptation modifications. The analysis relied upon a three-step modeling procedure to develop quantitative estimates of the risk of flooding in the existing tunnel. The use of this modeling procedure underscores the need in adaptation analysis to select appropriate, high-fidelity, storm surge and wave computer models and to include experienced coastal engineers on the study team. One of the primary lessons learned was that when evaluating the impacts of storm surge, wave height must be included in the analysis; otherwise, the tunnel could flood due to wave overtopping some of the portal walls even when the storm surge elevation is below the wall crest elevation as found in the study of the Wallace Tunnel. A second lesson learned from this case study was that the most commonly understood measure of storm strength - the Saffir-Simpson Hurricane "Category" Scale - was not particularly valuable for engineering decisions. There is no one-to-one relationship between storm surge and storm "category." The fact that commonly used measures to describe extreme weather events are not readily used in engineering design processes is not true only for tunnel and potential flooding analyses. It represents an important challenge to engineering design for extreme weather conditions.

A final lesson learned relates to the adaptation design options. Seemingly logical design options may not effectively achieve the goal of protecting the tunnel from flooding. Increasing the portal wall elevation just to account for storm surge alone would not have increased the level of flood protection much. One must consider wave impacts on top of the surge levels at the more exposed portals. Along with this broadened consideration, the adaptation design process also needs to include some iteration or "feedback-loop" process such as the search for more effective alternative design options as was done in the case study.

5.4.8 Shipping Pier Exposure to Storm Surge

Dock One at the McDuffie Coal Terminal was analyzed to determine the performance and survivability of the dock against environmental loads associated with sea level rise and storm surge.[503]

This case study determined that the likelihood of the pier structure performing well under all three climate scenarios was very high. The reason is that industrial piers like the one at McDuffie are designed for very large loads. They tend to be very robust and able to stand environmental forces. Berthing and mooring loads are typically much greater than the loads caused by storm conditions; thus, a pier designed to withstand berthing and mooring loads should be able to withstand storm surge loads and in fact, there are no code requirements with regards to storm

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[502] Based on the study *Storm Surge Analysis for the I-10 Tunnel* performed by Douglass et al. (2007) on the I-10 tunnel under the Mobile Ship Channel.

[503] Only the main pier structure was analyzed. In a comprehensive analysis, other features at this location would also be examined, e.g., the mooring dolphin, and mooring dolphin access walkway, electrical equipment, and mechanical equipment.

surge for facilities like Dock One for this reason. Historically, the survivability of these types of structures has been very high during storms.

The key weakness in terms of survivability to increased loads occurs at locations where different asset elements are connected. For example, the walkway connecting the pier to the shore is anchored at both the main dock and at each pile bent cap beam by anchor bolts either drilled or cast in concrete depending on their location to resist uplift. These anchorages also survived the lateral loads applied due to surge and waves for each of the scenarios analyzed. However, similar facilities in the region do not have these anchorages and would be more vulnerable to the surge scenarios analyzed. Additionally, where these connections exist, such as at Dock One, they should be inspected regularly and evaluated for their capacity to withstand loading from the proposed climate scenarios. This would include all areas requiring anchorages such as access walkways.

This observation also suggests the need for asset condition monitoring. When conducting climate change analyses of existing pier facilities, a thorough understanding of the facility's condition must be factored into the analysis. Information such as condition assessments, load ratings, and structural inspections should be consulted and, if not available, new inspections of the facility should be undertaken.

Finally, while the dock structures at industrial facilities will typically survive surge events, the same cannot always be said about the equipment and ancillary assets associated with pier operation. A full analysis of a port facility should consider all components of port operations including equipment, storage facilities, and access routes.

5.4.9 Pavement Mix Design Exposure to Temperature Changes

This case study examined the current pavement design practices of ALDOT and analyzed how climate change might affect mix designs of both asphalt and concrete for new highway facilities. Specifically, the effect of higher temperatures on the pavement in Mobile was evaluated, since temperature change is the key environmental factor affecting pavement performance. The focus of the case study was to evaluate how to help prevent premature pavement design failure from on joint cracking and rutting.

The most important lesson from this case study is that, while the current performance grades specified locally are adequate, higher temperatures may require adaptation strategies for pavement design, for example, under the "Hotter" narrative, temperature increases are projected to become great enough by mid-century that adaptation options should be considered. A variety of possible adaptation options were provided for coping with projected temperature changes under the "Hotter" narrative, including moving towards asphalt binders rated for higher temperatures and coarser aggregates.

For concrete pavements, modern specifications should account for the use of improved materials in order to ensure improved concrete performance under all placement conditions. To provide improved performance for sections paved under hot weather conditions, one option could be to revise the continuously reinforced concrete pavement (CRCP) reinforcement standards so that they provide steel quantities for specific use during hot weather conditions, and also to revise the specifications to limit the maximum in-place concrete temperature during hydration.

Finally, this case study found that there is very little information in the professional literature on the impact of climate change on pavement materials. Pavement engineers, with assistance from government agencies and climate change experts, should be encouraged to develop a protocol or guide for considering potential climate change in the development and evaluation of future designs. The guide should extend beyond the narrow focus on pavement mix design in this case study to incorporate all elements of climate change impacts on pavement design (e.g., sub-base, drainage, pavement texture). Researchers should explore if and how the AASHTO Ware Pavement Mechanist Empirical (ME) Design® software can be adapted to incorporate climate change. Amending the software to incorporate climate change projections would be a logical next step in its development.

5.4.10 Continuous Welded Rail Exposure to Temperature Changes

If rail temperature is not properly considered in the laying of track, the track structure can become disturbed in periods of extreme heat or cold, requiring maintenance expenditures, causing train delays, and leading to a heightened risk of derailments. This case study examined the impacts of projected temperature changes on new track laying and maintenance of way practices. The focus was on continuously welded rail (CWR). Unlike for other case studies, it was not necessary to select a specific segment or asset, so this case study evaluated the impact of higher temperatures on CWR in Mobile generally.

Specifically, the analysis considered the impact that changing climate conditions would have on determining the neutral temperatures, which is the rail temperature that would result in zero thermal stress on the rail itself (i.e. result in neither expansion nor contraction). Neutral temperature is a key factor in determining how to lay rail, and this analysis examined how increased ambient temperatures could affect the neutral temperature (which is measured as the temperature of the rail itself).To assess the impact of on the, this case study considered the relationship between the maximum and minimum ambient air temperature (as output from climate models) and actual rail temperatures.

The case study found that the neutral temperature used by the railroad (100ºF or 37.8°C) would be inadvisable under the “Hotter” narrative at all future time periods. Continuing to use the adopted neutral temperature might increase the risk of sun kinks in the future. However, the current neutral temperature would still be adequate under projected temperatures of the

"Warmer" narrative. Thus, whether or not the current neutral temperature is adequate for the future depends on the assumed changes in future temperatures.

The case study also found that there are very limited options for handling temperature-related threats. The primary adaptation action that could be taken on new CWR track would be to increase the rail neutral temperature. Alternately, ballasted tracks could be constructed and maintained with a minimum of one foot (0.3 meter) wide shoulders to provide lateral support to the ties.

5.4.11 Operations and Maintenance Activity Exposure to Climate Change and Extreme Weather Events

Operations and maintenance (O&M) activities address such challenges as the aging of the infrastructure and the impacts of increasing traffic volumes as well as the significant stress that extreme weather events place on the transportation system. Heavy precipitation, lightning, and strong winds affect operations and maintenance activities. Severe storms will disrupt most O&M activities and even a light rain or moderate wind can be enough to delay activities like painting or sign replacement. In addition to such direct impacts, indirect and synergistic climate effects can also be of concern. For example, drought and wildfire conditions associated with climate change can increase sediment loading and cause trees to weaken and contribute to more dead wood in stream valleys, resulting in clogged culverts. This case study examined how weather and climate may influence current O&M activities and how those activities may be adapted to reduce the vulnerability of transportation systems to weather and climate-related risks.

Because Mobile has historically experienced extreme weather events, their community and transportation agencies have developed important best practices that could be employed by other communities expected to experience more extreme events in the future. First and foremost, O&M personnel in the Gulf Coast region need to be prepared for the unique and continuing challenges of extreme weather, particularly when it comes to cooperation between organizations. In Mobile, emergency operations and training have become vital components of O&M programs. Emergency response and operations training occurs annually and is embedded in the work culture. In the event of an emergency, response operations become a collaborative effort among the city, county, and state departments of transportation.

Other best practices include:

- Have a tested "play book" for emergency operations and, in particular, evacuation protocols.
- Include key stakeholders (e.g., the state emergency operations agency, police, fire, schools, hospitals) in routine information dissemination so that all will be in sync during an emergency.
- Pre-develop contracts so that reimbursements from FEMA can be easily processed. ALDOT has these vehicles in place for concrete, erosion control, traffic control, striping, etc. where the contractor charges a set rate based on what they bid on plus materials and equipment.

- Pre- position emergency equipment like backhoes and chainsaws away from locations vulnerable to storm surge prior to hurricanes or other extreme weather events for quick and flexible deployment so that if one location becomes inaccessible, equipment may be available from others.
- Cross-train operations staff with maintenance staff to enhance their working relationship during emergency events.
- Work with local colleges and universities to incorporate maintenance engineering courses into the curriculum**.** A lot of on-the-job training is required to get engineers prepared for extreme weather events and emergencies. Maintenance engineering classes at the college or university level would help prepare entry level engineers and improve their understanding of what they can expect to experience when they start working for a city or DOT.
- Harden communications and power systems for emergency use.
- Supplement and adapt ITS resources for disaster monitoring and response.
- Provide detours and signage as may be needed for evacuation
- Keep the system in a state-of-good-repair.
- Work resilience into the asset management system.
- Budget and plan for extreme events. Run operations like a business. With limited funding, it's important to know which assets or projects are most critical to the safety, reliability, and performance of the system. When funding becomes available, you'll know where and how to direct funds. When possible, develop regular replacement cycles to reduce maintenance costs associated with aging equipment.
- Communicate well and communicate often**.** Coordination within the agency, between and across agencies, departments, jurisdictions and levels (e.g., municipal, county, metropolitan planning organization, state) of leadership is necessary. Develop coordination and communication plans and ensure that reliable communication devices are available in the event of an emergency or extreme weather event

5.5 Conclusions

The engineering case studies, and the process of developing the inputs that are used in this design process, have shown that climate-change related factors can be successfully incorporated into the engineering design process. The *General Process for Transportation Facility Adaptation Assessments* used here has been shown to be a useful approach for considering the uncertainties associated with future environmental variables, and as an organizing concept for adaptation engineering studies. The case studies also showed the lack of available technical guidance for conducting adaptation engineering, and the barrier this represents to many engineers who rely on accepted practice for the design approach.

Data is always a key issue when considering the design of a facility. Engineers want to provide a safe, durable, and stable facility that will withstand the stresses and loads coming from the environment and from the use of the facility itself. When considering the uncertainties associated with the future values of design parameters with different climatic conditions, credible and defensible values for design inputs become even more of a challenge. The approach illustrated in this study has been to use climate narratives that represent possible future conditions that become the input to the design process. These narratives provide a range in the level of stress that is placed on different types of assets for different climate-related variables. Given that the ultimate design will reflect the values of the design parameters utilized in the design process, great care must be taken in defining the scenarios that are used and consideration of the cost and practicality of the adaptive designs.

Finally, and perhaps most importantly, the engineering case studies showed that many of the assets analyzed in the Mobile study area were indeed vulnerable to changing environmental conditions. Whether due to storm surge and wave action combined with higher sea levels or higher temperatures, the assets as currently designed in accordance with accepted current design approaches may not withstand some of the stresses that might occur given a changing climate. There is a strong need to provide technical guidance and design methods to account for such uncertainties as the nation builds new infrastructure or rebuilds existing infrastructure.

6. Future Research

6.1 Introduction

This section identifies research that is needed to better understand the implications of climate change and extreme weather events on transportation infrastructure. Similar to the prior section on lessons learned, this section presents overarching research needs from the perspectives of adaptation planning and engineering design, as well as specific research needs pertaining to individual asset types. The following section focuses on research needs relating to the overall process of conducting adaptation planning and engineering. The next section focuses on research related to the input values for the key variables in the design process. The final section proposes research topics for asset-specific adaptation engineering.

6.2 Research on the Approach to Adaptation Related Planning and Engineering

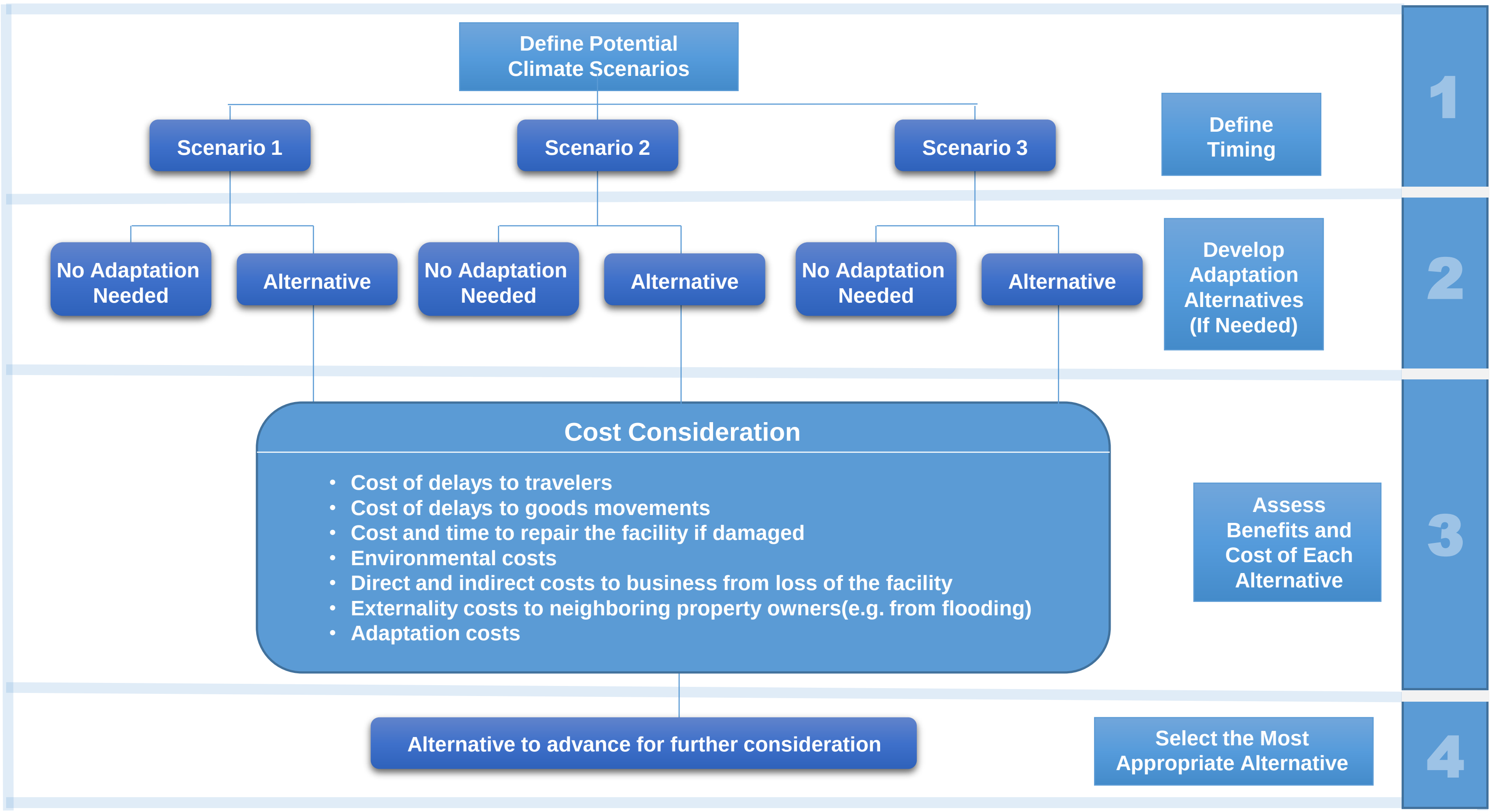
The *General Process for Transportation Facility Adaptation Assessments* (the *Process*) used in this study provided a useful construct for how planning and engineering design can be conducted taking into account the uncertainties associated with future design inputs. From the perspective of the overall approach, Step 4 – Decide on Climate Scenarios and Determine the Magnitude of Changes, is one that provides the greatest change from traditional engineering design. In essence, this step states that in addition to historical data and the trends associated with such data, engineers should examine alternative possibilities for future conditions through the use of scenarios, and then determine the level of asset vulnerability under a range of scenarios. Depending on the resulting vulnerabilities, it may be appropriate to develop a design for each scenario.

The following three recommended research topics relate to the process of adaptation planning and engineering; that is, elements of the overall approach that structure how data for engineering analyses are developed and ultimately how the resulting information is used to determine priorities.

6.2.1 Use of Scenarios in Adaptation Planning and Engineering

One of the key areas for future research is continuing to examine alternative ways that climate change scenarios can be used in adaptation planning and engineering. For example, Figure 99 shows one perspective on how scenarios can be used - in this case leading to one alternative design that provides the most robust strategy in light of future environmental uncertainties. The consideration of scenarios is one of the major approaches being used by planners to account for uncertainties in future conditions or characteristics. As shown in the engineering case studies, the scenario approach used in this study was very influential in the case study results. The proposed research would look at different ways that scenarios could be used in adaptation planning and engineering. In addition, the research would examine the role of land use scenarios in identifying future climate change-related risks to a community (see next research topic). Although it may be important to look at different climate stressors simultaneously (such as sea level rise *plus* precipitation, which could have an additive impact on drainage), care must be taken when doing so. Projections for each climate variable were developed independently, and there is often no justification for assuming, for example, that a certain precipitation scenario would happen under the same "future" as a certain sea level rise scenario. This is not to discourage consideration of multi-stressor climate scenarios when managing risks to infrastructure. Indeed, all climate projections and scenarios are developed based on certain assumptions, and none are considered more/less likely to occur than others; developing future scenarios with multiple climate stressors is another form of scenario-planning, but doing so may increase the overall uncertainty regarding the scenarios. Additional research should look at ways to better evaluate simultaneous climate stressors without exaggerating a "worst case" scenario and without introducing unacceptable levels of uncertainty.

Figure 99: Use of Scenarios in Adaptation Planning and Engineering



6.2.2 Development of Future Surge Scenarios

As was noted in Section 4.4.4, the method for developing storm surge scenarios with respect to the return period event and wave modeling is an important area of research. Different approaches can be used based on the data and models that are available, the desires of local groups to link an event to local experience, and level of desired consistency with engineering practice. Research could examine the degree to which approaches differ in terms of level of impact, the relevance of the results to local decision making and the incremental benefit in terms of scenario output of using a more complex and sophisticated approach versus one that simplifies the process.

6.2.3 Consideration of Future Land Uses

Note that in the "cost considerations" element of Figure 99 that one category of costs is "externality costs to neighboring property owners." Many of the climate change-related vulnerability assessments in the U.S. have assumed current land use patterns, and have not included a consideration of future land use types or densities (with a notable exception being the Environmental Protection Agency's Integrated Climate and Land Use Scenario [ICLUS] initiative).[504] For a process that is examining a 50- to 80-year timeframe, assuming no change in land use patterns is unrealistic, especially when one needs to determine levels of criticality and risk for different types of assets in the transportation system. Another use for scenario analysis would thus be to consider land use scenarios that reflect possible changes to the community that should be considered in the analysis.

6.2.4 Incorporation of Adaptation Results into Decision-Making

The engineering case studies recommended adaptation strategies for individual transportation assets. Except in one case, the culvert case study, the economic analysis was not done to determine the overall cost effectiveness of the recommended strategy. No effort was made to compare the adaptation strategies among different assets to determine which ones would be better from the perspective of benefits and costs. And no effort was made to take the next step, which is to investigate how individual adaptation projects should be considered in the context of larger investment programs.

In particular, there is a disconnect between the timeframe for most metropolitan long-range transportation plans (20 to 25 years) and the 50 to 80-year timeframe associated with most adaptation planning.[505] Important questions relate to how the results from adaptation planning and engineering design efforts can be included in the transportation planning and programming process of typical metropolitan planning organizations (MPOs), states, and other investors. How does one prioritize adaptation projects in conjunction with capacity, safety, security, economic

[504] EPA, 2014

[505] A similar, more striking disconnect can occur in situations where federal funds are released after an extreme weather event. These funds may be released with the intention of quickly bringing a damaged transportation system back to its normal state of operation. These funds often have requirements that they be spent within a certain near-term timeframe, making it particularly difficult to undertake thoughtful analyses of climate risk.

development and other types of projects? Where will the funding come from to pay for adaptation-related strategies? With limited resources, most state transportation agencies and MPOs may find it difficult to implement an adaptation-specific investment program. It will thus become important for planners and engineers to identify the co-benefits associated with adaptation strategies. What benefits will a reconstruction project have for safety, congestion mitigation as well as adaptation?

6.3 Research on Generating Input Values for Engineering Designs

One of the most important factors in determining engineered adaptations to threatened assets is the value of the input variables that guide engineering design. As noted in Section 5.3, future climatic conditions are likely to produce different values than what has been seen historically. How one determines such values is still a critical research question. The climate scenario approach used in this study provided values based on modeling results and on assumptions about the type and magnitude of climate-related stresses on transportation assets. As also noted in the previous section, this approach is new to engineering design, which has relied on design storms that are defined by historical record. Engineering designers have traditionally based decisions on probability of occurrence or exceedance of certain input variables and are comfortable with that approach. The probability of exceedance approach to input variables has a long and relatively-well understood history. In some cases, a site-specific evaluation based on benefit/cost ratios may be warranted, for example, when replacing a culvert that was put in place before an area was fully developed and now would represent a high economic loss if the culvert were to fail if replaced in kind. Developing tools that can simplify the analysis process would be useful.

6.3.1 Deriving Input Values for Design Variables

Given the importance of the values for design variables, research is needed on examining alternative methods for estimating such values. For example, the development of climate change intensity-duration-frequency (IDF) curves that reflect possible changing climatic conditions is recommended to aid in the translation of climate model outputs into inputs useful for engineering design. This is particularly important for some asset types, such as smaller culverts, where 24-hour projections may not be applicable; such design relies on IDF curves for which downscaled climate model data is not readily available.

Updating design flow equations based on more recent or projected climate data could be another way to incorporate changing climate data. However, more thought is needed about how to uniformly incorporate new climate data across all design activities.

6.3.2 Using Asset Management Systems as a Platform for Vulnerability Assessment

As was repeatedly found in every engineering case study, data on asset condition and performance was difficult to obtain and monitoring of supporting infrastructure was limited.

Infrastructure fails more frequently due to compromised structural integrity exacerbated by moderate climate events rather than by extreme conditions. This increased wear and tear on assets emphasizes the need for good asset management systems that proactively identify assets vulnerable to failure.

In addition, it was a challenge finding data on the latest geotechnical studies or structural integrity tests. Research is needed to illustrate how asset management systems can be used to monitor asset response to changes in climate and provide the data desired and needed for conducting adaptation engineering analysis. Moving Ahead for Progress in the 21st- Century (MAP-21) has placed greater emphasis on risk-based asset management and every state DOT is in the process of either developing or upgrading asset management systems to meet Federal requirements. Now is the time for the link between climate change and extreme weather-related asset considerations and evolving asset management systems to be examined closely.

6.4 Research on Asset-specific Needs

The following research topics resulted from the engineering work that was conducted as part of the case studies.

6.4.1 Culvert Vulnerability

Culverts are one of the most important components of any linear facility. The level of analysis conducted in the culvert case study for understanding the level of vulnerability was quite extensive and most likely beyond the time and resource constraints of most transportation agencies. There is thus a need to refine the culvert vulnerability analysis methods developed in this study, and also investigate new measures of culvert vulnerability to increased flows and other factors that could be applied more generally or as part of an asset management system. This topic could be expanded to look at failure modes for a range of possibly at-risk facilities.

6.4.2 Controlled Failure Approaches

The bridge case studies found that the bridge deck was vulnerable to each of the three surge scenarios tested. Design of a controlled failure superstructure was, in some cases, found to be a reasonable design option for low lying bridges in such environments. Research is needed to determine if other types of transportation assets can also be designed with breakaway components, such that the level of disruption to facility users would be minimized as the recovery process proceeds.

6.4.3 Embankment Breaching

The case studies that examined the impact of storm surge and wave action on embankments and abutment erosion found that the information most lacking for design purposes is the phenomena related to embankment breaching. Many studies have established estimates of flow rates and breach dimensions for earthen dams and levees, but not many have developed methods to predict the onset of embankment breaching or focus on highway embankments that are somewhat protected by pavement on top. This is an area of future research that would be needed in order to more accurately predict the impact of inflow and outflow from storm surge flooding on highway embankments.

6.4.4 Shipping Pier-related Research for Extreme Loads

Research and testing is necessary in order to establish a procedure for both load development and structural analysis of shipping piers that can be adopted universally for structures of the type examined in the case study with varying configurations. This research should culminate in a credible design guide that could act as a resource to shipping pier designers. The limited guidance that is currently available should continue to be vetted by comparing theoretical results with the loads and stresses on shipping piers that result from actual events.

6.4.5 Erosion Due to Backflows

Although engineering design today considers the potential of erosion due to water flows at embankments and other types of supporting structures, the potential for such erosion at underpasses (that funnel water flows during surge events) as water flows back to the coast deserves greater attention.

6.4.6 Abutment Scour

Abutment scour analysis procedures should be developed to allow for more accurate prediction and characterization of abutment scour in coastal areas. For example, with improved prediction methods and tools, structural design guidelines might focus on providing stable abutments without the need for outside protection schemes, such as riprap or bulkheads.

6.4.7 Temperature-related Design Parameters

The results of the pavement and rail case studies showed that under the “Hotter” narrative of future climate in Mobile, design had to move towards asphalt binders rated for higher temperatures and coarser aggregates. Rail track design practices as they relate to rail heat kinks do not consider the uncertainty relating to higher expected temperatures. For example, more research is needed on the appropriate value of rail neutral temperatures in the context of rail track design. Specifically with regard to rail design, the viability of using the Federal Railroad Administration’s (FRA) rail weather system for rail engineering analysis should be subject to further research.

6.4.8 Guidance on Pavement Design

One of the lessons learned from the pavement design case study is that additional guidance is needed with respect to how potential changes in climate will affect pavement performance, and thus the design process. Although not a research topic per se, the need to develop such a guide and to modify existing pavement design software to account for projected temperature conditions is an important next step that results from the work conducted in this study.

6.4.9 Weighing Costs versus Benefits of Adaptation in Light of Uncertainty

The analyses described in this report relied upon the development of certain future climate scenarios. There is inherent uncertainty associated with these scenarios, and it is not possible to make a determination as to which scenarios are more likely to occur. As discussed earlier, the choice of scenario(s) can have significant influence over the outcome of these analyses, and looking across a range of scenarios could yield a significant range of uncertainty regarding possible impacts on an asset.

This uncertainty raises challenges when evaluating costs and benefits. However, future research could evaluate not just the benefits and costs of implementing a specific adaptation measure, but it could also investigate how to determine the point at which marginal costs of adaptation get significantly larger or smaller. If more modest improvements are relatively inexpensive, it may make sense to focus on making those improvements rather trying to justify more extensive expenditures that yield only marginal benefits. Conversely, some adaptation measures may have significant upfront costs, and additional improvements result in small marginal costs. In this situation, it would be useful to understand whether the benefits associated with the minimal level of improvement outweigh the initial costs, or whether it is important to be able to justify more extensive improvements.

6.4.10 Additional Research on Prioritizing Assets to Undergo Analyses

The analyses presented in this document demonstrate potential methodologies for evaluating climate change impacts and adaptation strategies at the project level. To date, there have been only limited efforts to develop detailed methodologies appropriate for the project level; most climate change vulnerability assessments and adaptation evaluations have either been at a broader level, or were very narrowly focused on a specific asset without regard to replicability of the methodologies to similar assets elsewhere.

However, conducting these analyses are resource-intensive, and it is unrealistic to think that most transportation managers would be able to conduct these analyses for all assets under their purview. In fact, attempts to do so could result in wasted time and money if efforts to narrow down the list were not made first. To narrow the list of assets for this project, the project team conducted a criticality assessment and vulnerability screen earlier in this project, to identify which assets were both highly critical and potentially vulnerable. However, for many asset types, there were still a number of assets that were considered highly critical and vulnerable. It would be useful to explore other approaches for further narrowing down the list of potential assets to analyze. Such approaches could even be employed if no criticality or vulnerability assessment were conducted.

Prioritization approaches could consider the remaining useful lifetime of assets against the timeframe of climate changes, the potential costs of damages associated with impaired use of the asset or repair costs, or more specific indicators of vulnerabilities that are geared toward very particular asset types.

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