Preliminary Structural Analysis Results of Existing Bayway Structure

Level I Storm Surge March 18,2016

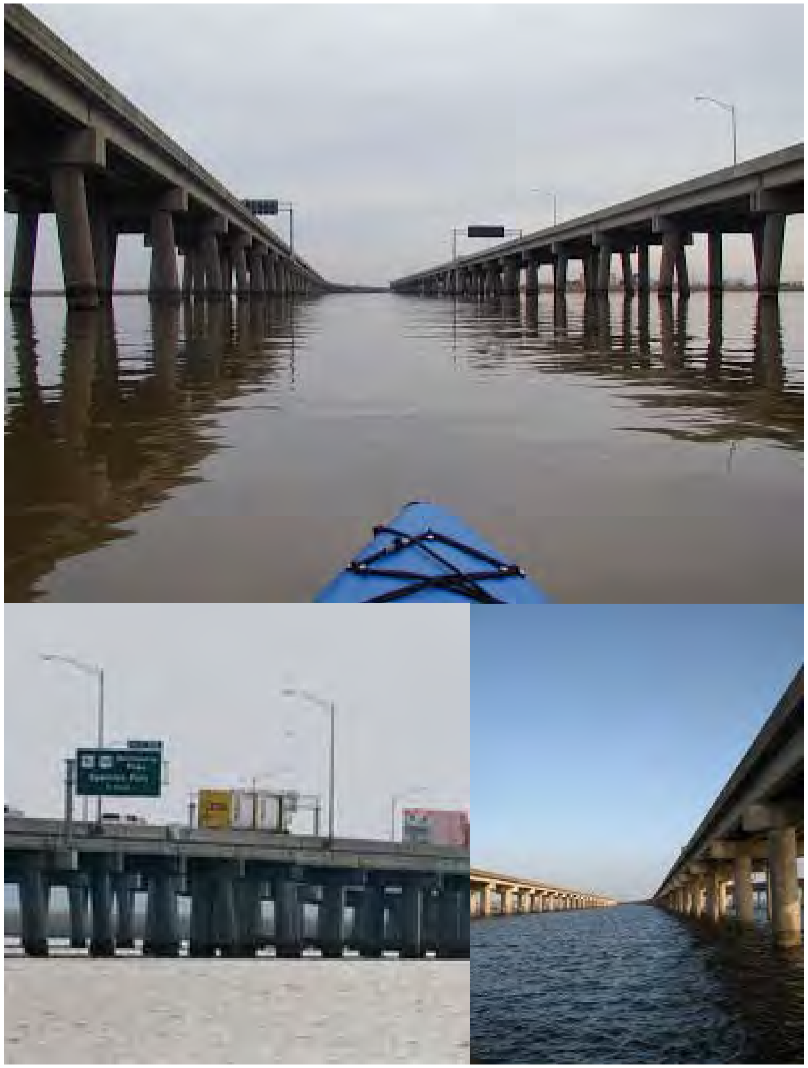
HDR

ALABAMA DEPARTMENT OF TRANSPORTATION

01 Review of Previous Meeting

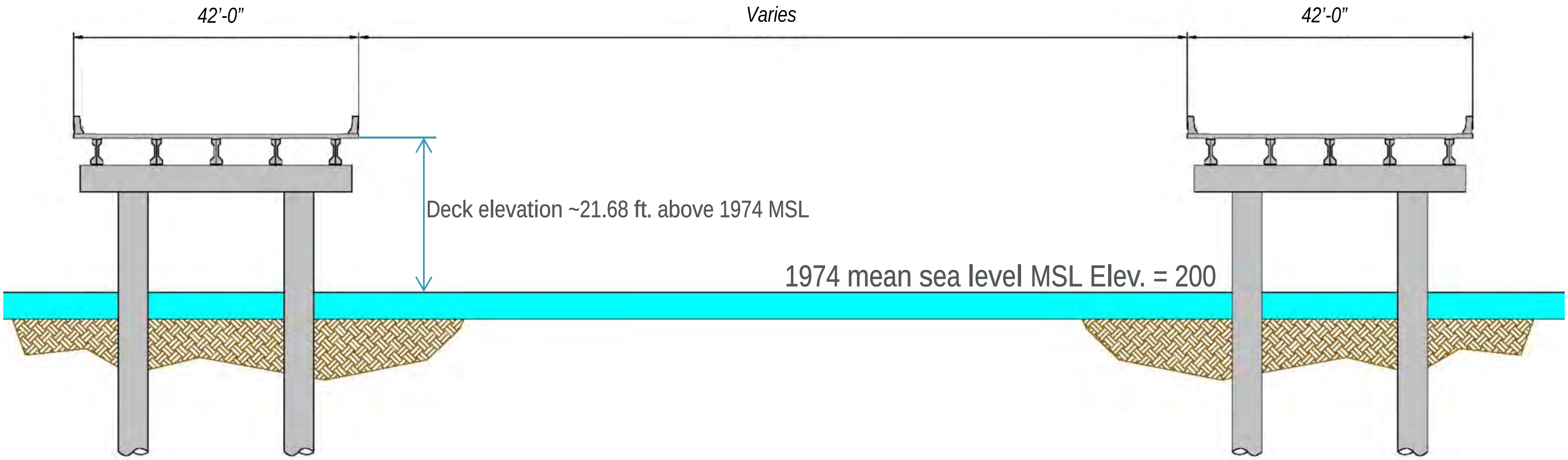
Action Items from our Last Meeting

- Inclusion of a 50yr event in the Level I Analysis (Done by HMM)
- Develop Conceptual Superstructure Retrofit Details and High-Level Cost Estimates
- Develop Substructure Retrofit Details and High-Level Cost Estimates



As we Discussed Last Time

- Past performance of Large Coastal Events on Bridges can be Broadly Grouped into three Categories:
  - Shifting of Spans on the Bent Caps
  - Damage to Girder Ends and Bent Caps from Impact of Superstructure on Substructure
  - Damage to bents from Lateral Loads Transferred to Them

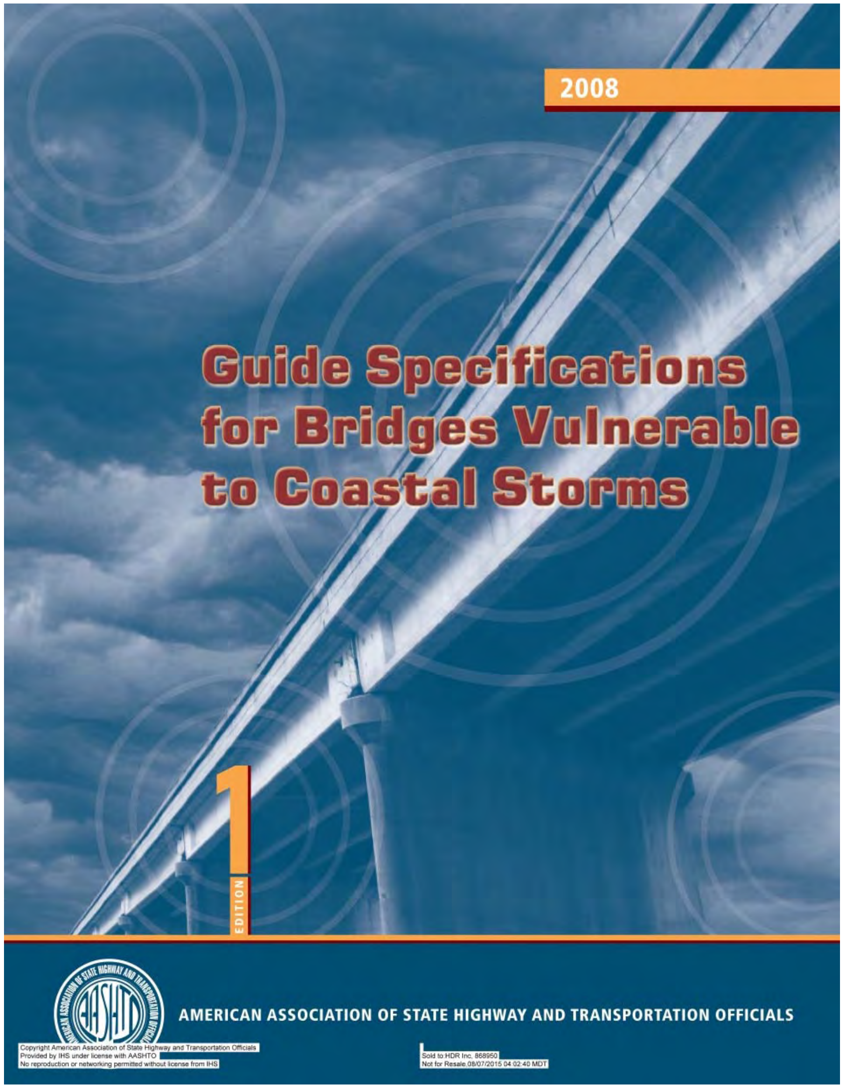


Bridge Cross-Section

Majority of the Spans Consist of this Geometry

AASHTO Guide Specifications

- Contains Specifications for the Design of Bridges Vulnerable to Coastal Storms
- In 2004 and 2005, Hurricanes Ivan and Rita Caused Significant damage to Numerous Bridges in the Gulf Coast
- FHWA initiated a Pooled Fund Contract for the Development of the Guide Specifications



2008

Guide Specifications for Bridges Vulnerable to Coastal Storms

1 EDITION

AMERICAN ASSOCIATION OF STATE HIGHWAY AND TRANSPORTATION OFFICIALS

Storm Surge Impact Analysis

- Prepared by Hatch Mott MacDonald
- Modified Level I Analysis
- Computations of Loads from a Range of Extreme Storms
- Also included future Sea Level Rise (SLR) Conditions
    - 1.3 ft. above current Condition in 2067
    - 3.0 ft. above current Condition in 2116
- Results are Presented for the EB Mainlanes (ramps are not included)

Hatch Mott
MacDonald

Mobile Bay Bridge
Storm Surge Impact Analysis

DRAFT Report

November 12, 2015

Submitted To:
Alabama Department of Transportation

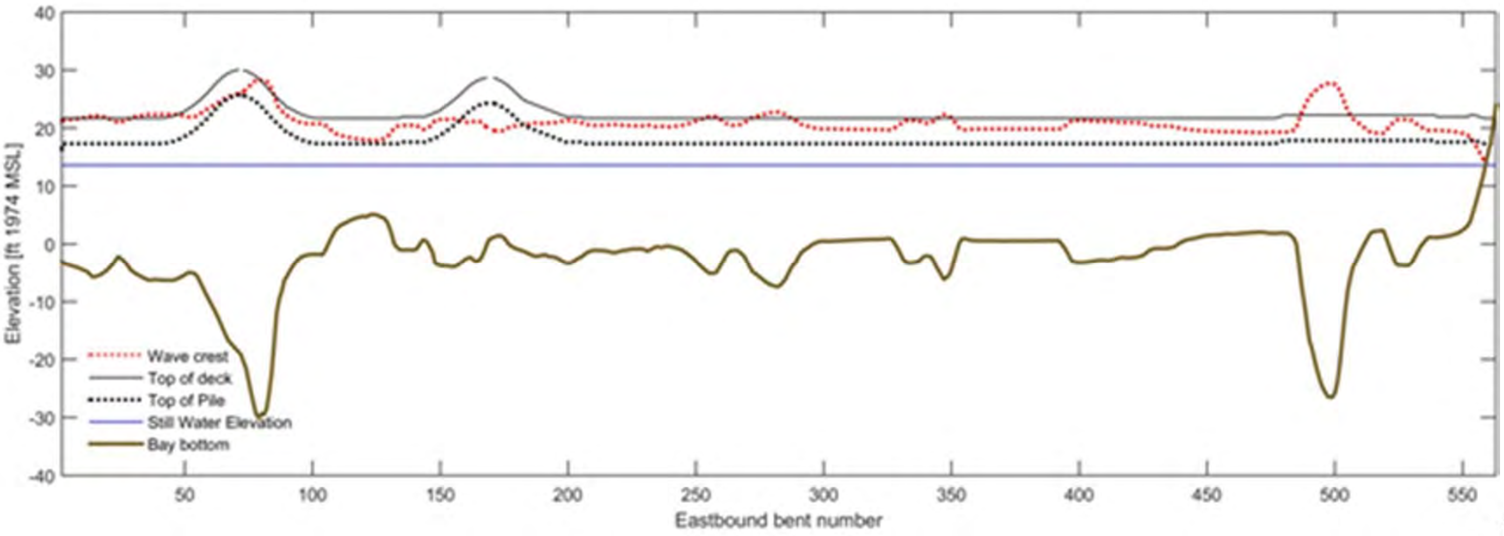
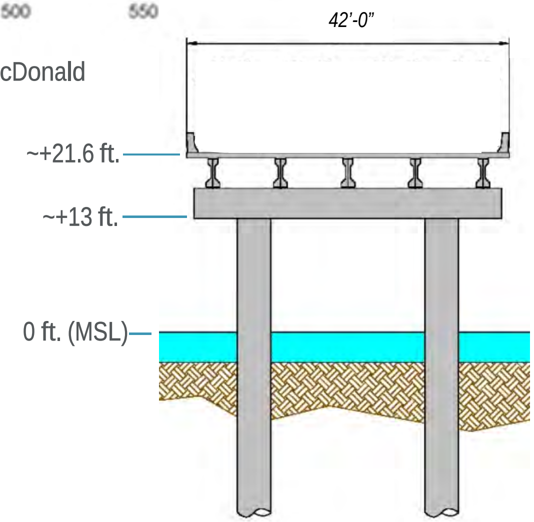


Figure Courtesy of Hatch Mott MacDonald



Still Water Elevation = Storm Surge at High Tide + SLR

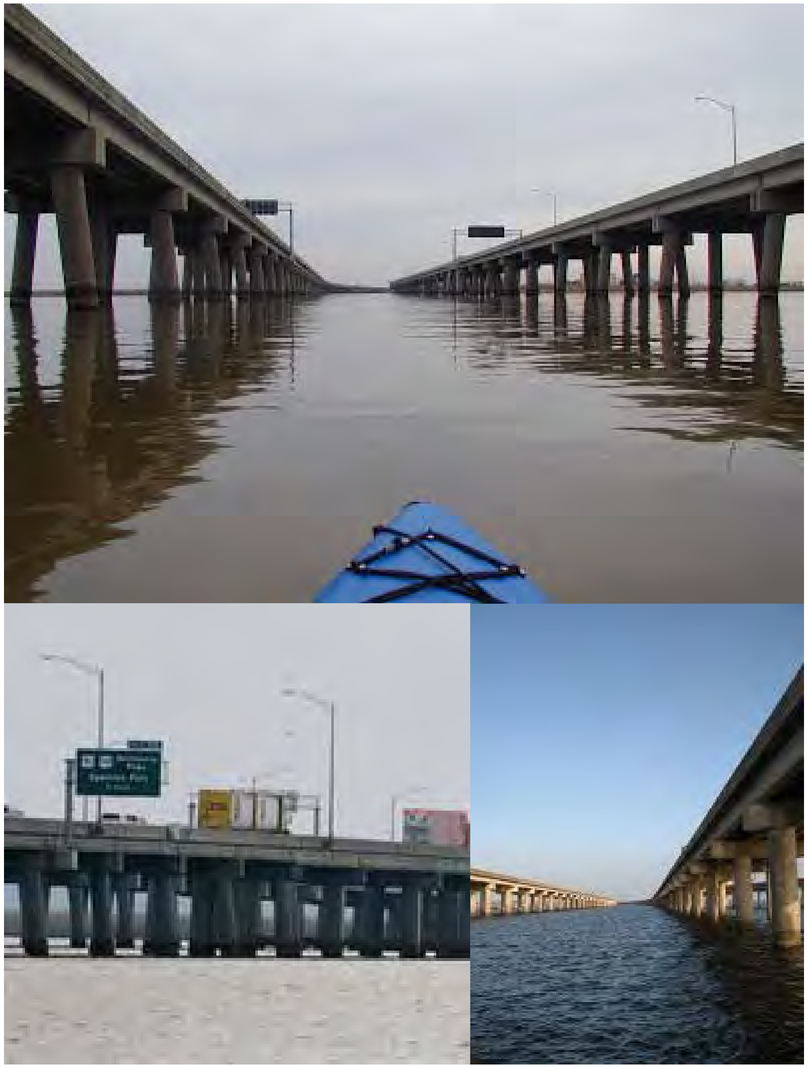
SLR = 0 for 2017 Sea-Level

Bayway Analysis

100yr Present Day Sea-Level (2017) Elevations

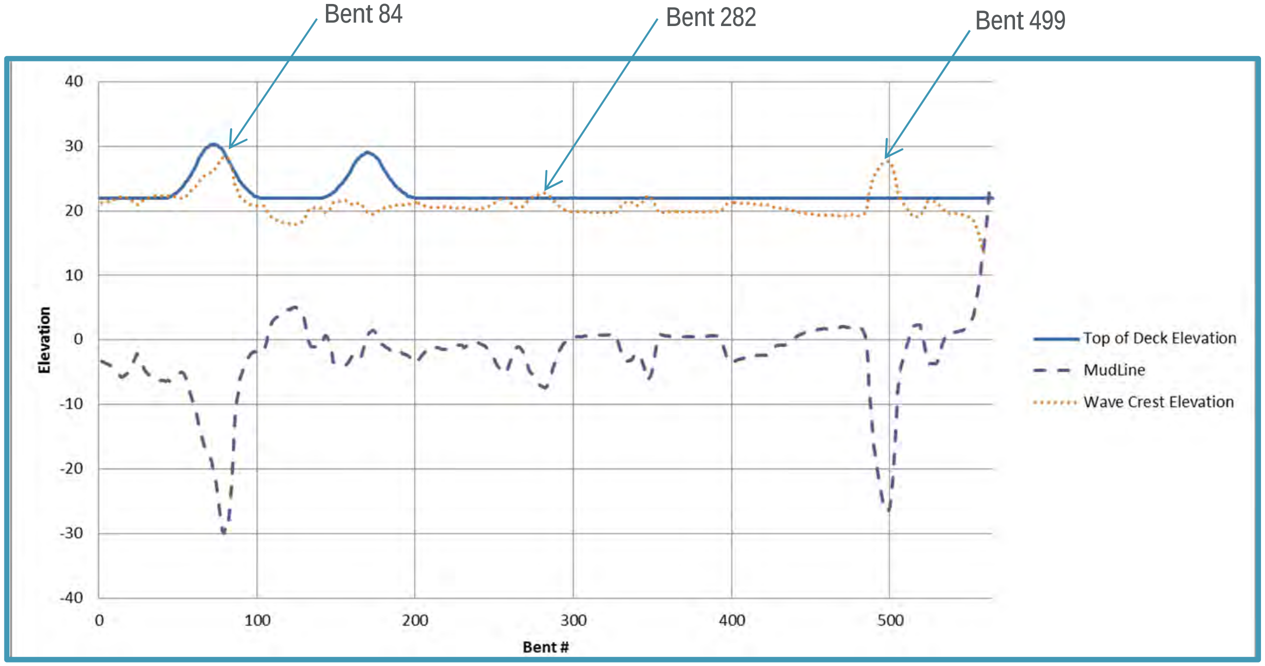
Storm Surge Impact Analysis

- Piles experience substantial loads over all 12 cases (always subject to wave loads when they are in the water)
- Deck elements only experience substantial horizontal and vertical loads when the maximum water level reaches the superstructure
- For the 2017 100yr Storm:
  - The still water level is below the top of the pile cap for the length of the bridge
  - The wave crest elevation is impacting the deck for almost the entire length of the bridge
  - Wave crests do not reach the bridge deck at the two high-rise sections



Selection of Critical Locations for Analysis

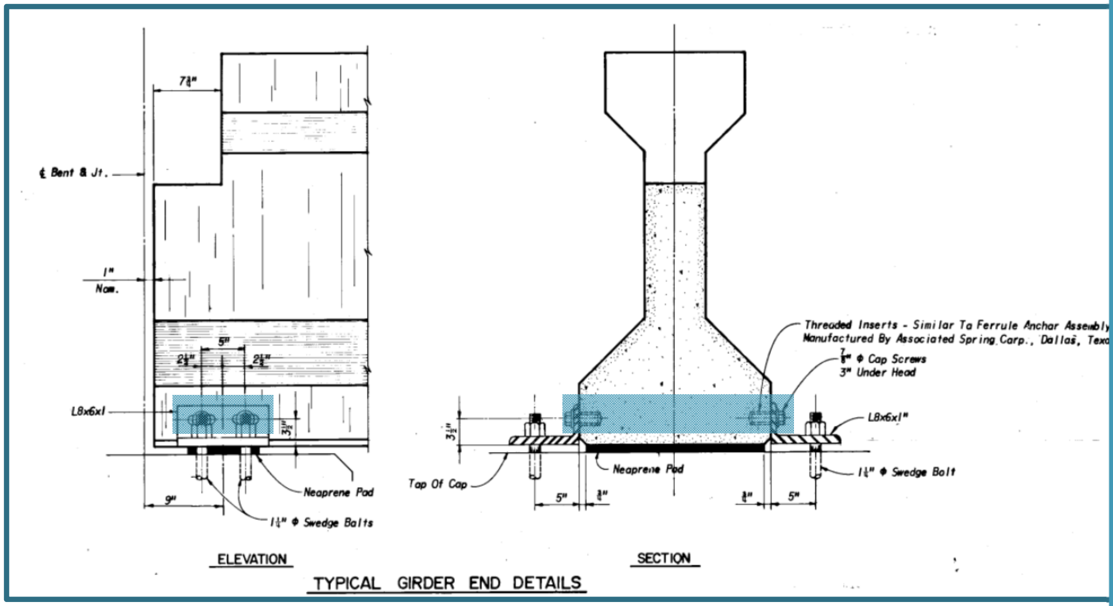
- Review Wave Crest Elevations for the 2017 Sea-Level 100yr Storm



Superstructure-Substructure Connection

- Capacity of Connection was Calculated.
- Uplift Capacity was Controlled by shear of the Threaded Insert (~10.3 Kips capacity)

For the 2017 100yr Design Event, 252 of the 564 EB Spans Would be Expected to be Unseated.



Superstructure Restraint Details

Substructure Lateral Capacity

- Using FB-Multipier lateral Capacity of 2-Pile and 3-Pile Bents was Calculated
- 2-Pile Bents (Group I) have a Lateral Capacity of ~150 Kips per Bent
- Bent 282 is Representative Bent

Performance Ratio	2017 SLR	2067 SLR	2117 SLR
10 Yr Event	0.29	0.29	0.29
25 Yr Event	0.38	0.65	2.35
100 Yr Event	2.67	2.76	2.85
500 Yr Event	2.85	2.85	2.85

2-Pile Bent Performance Ratios

Substructure Lateral Capacity

- Using FB-Multipier lateral Capacity of 2-Pile and 3-Pile Bents was Calculated
- 3-Pile Bents (Group II) have a Lateral Capacity of ~136 Kips per Bent (longer Unsupported Lengths)
- Bent 499 is Representative Bent

Performance Ratio	**2017 SLR**	**2067 SLR**	**2117 SLR**
10 Yr Event	0.24	0.24	0.24
25 Yr Event	0.48	0.74	2.29
100 Yr Event	4.71	5.00	5.33
500 Yr Event	5.71	5.71	5.71

3-Pile Bent Performance Ratios

Substructure Lateral Capacity

- To Establish the Scope of the Substructure Damage we Analyzed 500 of the 563 EB Bents for the 2017 SLR condition
- Damaged is Considered when Demand/Capacity ration > 1.0

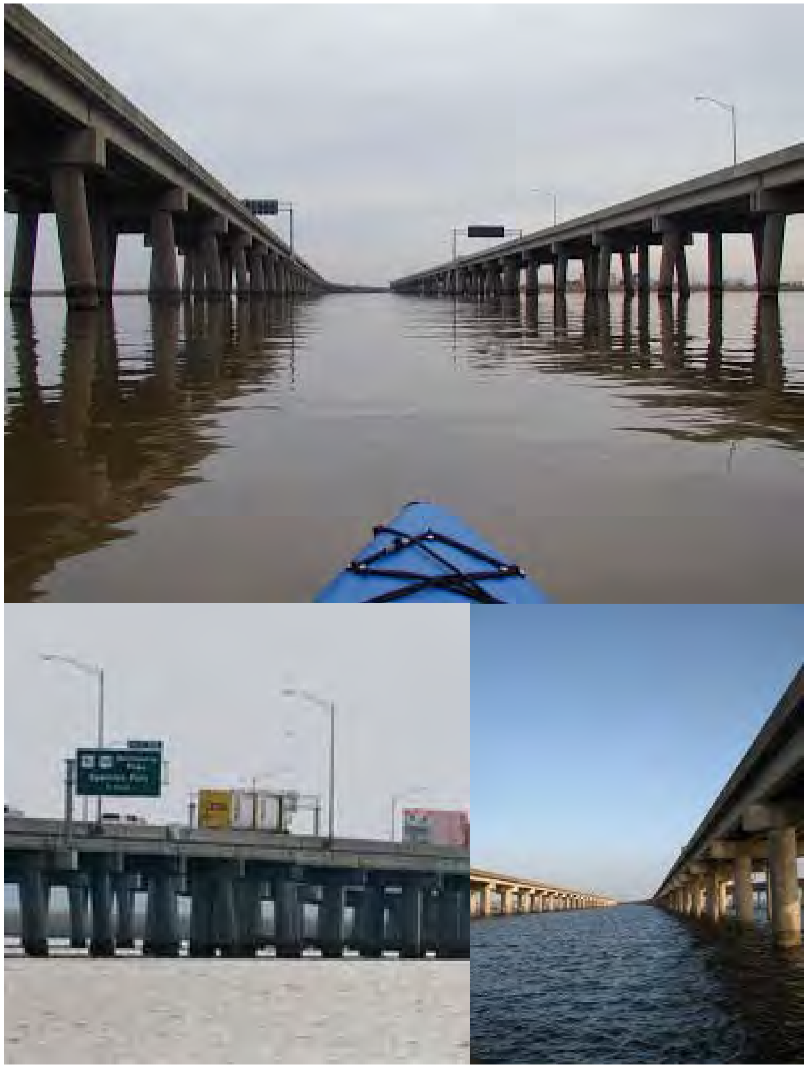
2017 SLR	# of Bents Damaged	% of Bents Damaged
10 Yr Event	0	0
25 Yr Event	0	0
100 Yr Event	247	44%
500 Yr Event	482	86%

02

Analysis Assumptions

Assumptions

- Bridge designated as “Critical/Essential” (Service Immediate) – Use Strength Load Combinations in AASHTO
- Retrofit Concepts for Future Widened Bayway not Considered
- No Scour Included – Needs to be Included during Design Phase
- Limited drawings of As-Built Superstructure
    - Unknown longitudinal Reinforcement/Prestressing in Beams
    - Assume 40 ksi Yield stress of Rebar
- Bond Strength Between Pile and Concrete Plug is Ignored

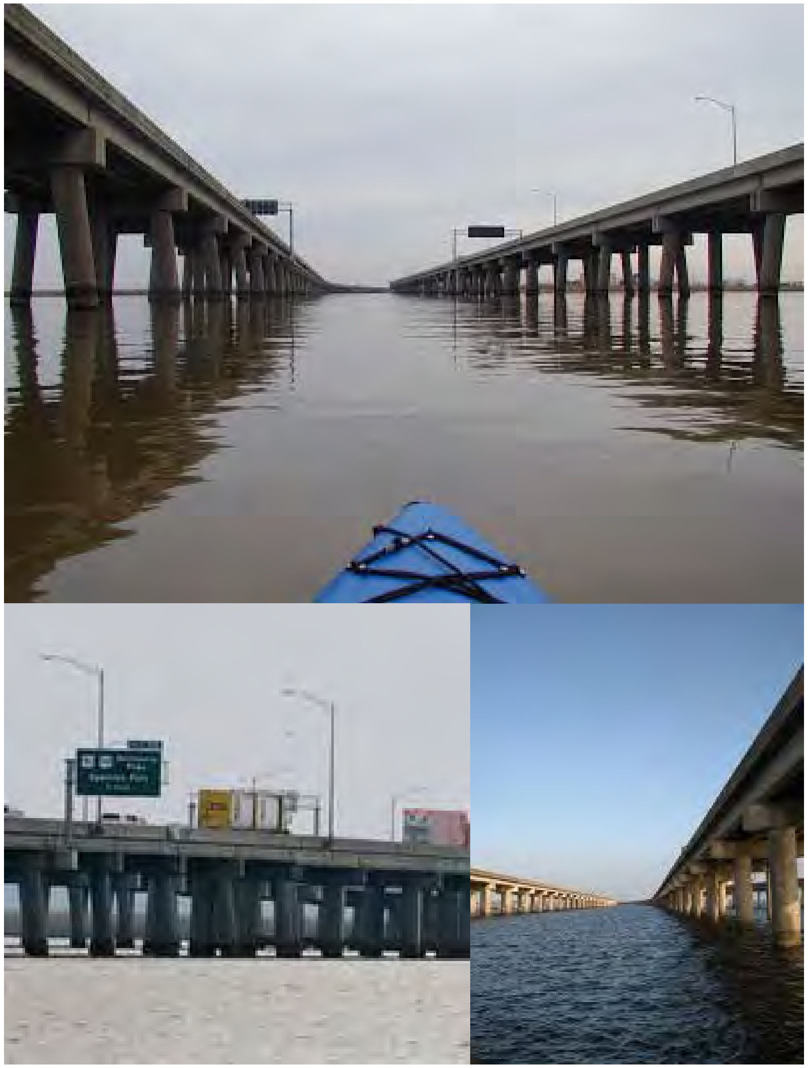


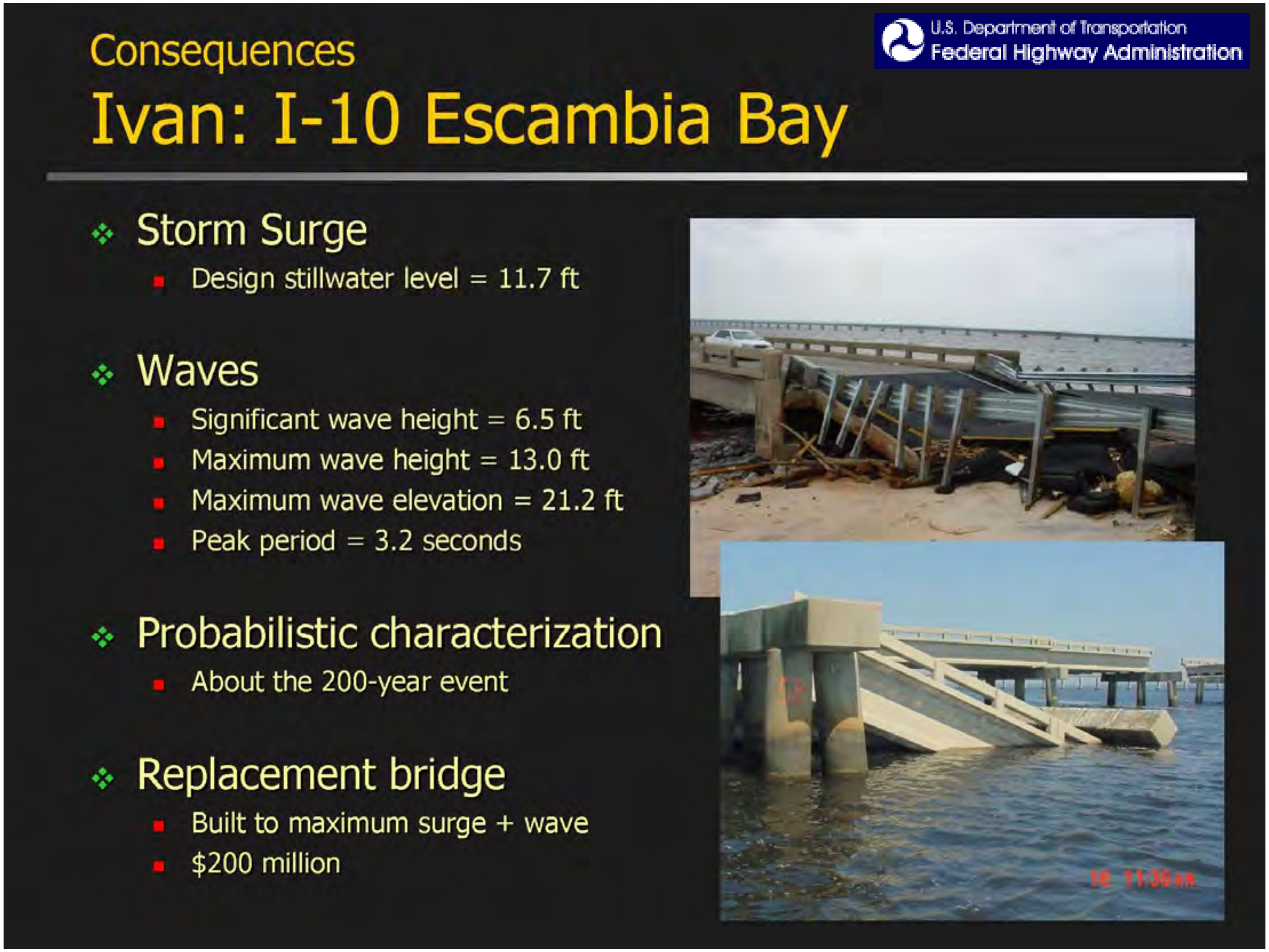
03

Storm Events Evaluated

Storm Events Evaluated

- 25-year Return Period (2017 and 2067 SLR)
- 50-year Return Period (2017 and 2067 SLR)
- 100-year Return Period (2017 SLR) - **Baseline**
- Level III Analysis (Future Task)
- Existing Bayway is 40 years Old so SLR beyond 2067 (50 yr.) was not Considered





U.S. Department of Transportation
Federal Highway Administration

Consequences

Ivan: I-10 Escambia Bay

- Storm Surge
  - Design stillwater level = 11.7 ft
- Waves
  - Significant wave height = 6.5 ft
  - Maximum wave height = 13.0 ft
  - Maximum wave elevation = 21.2 ft
  - Peak period = 3.2 seconds
- Probabilistic characterization
  - About the 200-year event
- Replacement bridge
  - Built to maximum surge + wave
  - $200 million

Consequences

Katrina: US-90 Biloxi Bay

- Storm Surge
  - Design stillwater level = 20 ft
- Waves
  - Significant wave height = 6.2 ft
  - Maximum wave height = 10.6 ft
  - Maximum wave elevation = 27.2 ft
  - Peak period = 5.1 seconds
- Probabilistic characterization
  - Slightly greater than 100-year event
- Replacement bridge
  - Built to maximum surge + wave
  - $250 million

Consequences

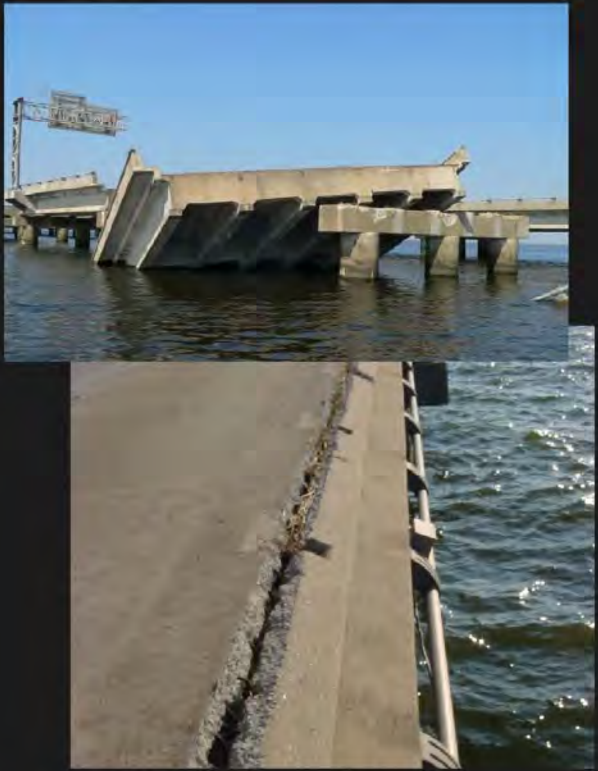
Katrina: US-90 Bay Saint Louis

- Storm Surge
  - Design stillwater level = 25 ft
- Waves
  - Significant wave height = 9.1 ft
  - Maximum wave height = 15.3 ft
  - Maximum wave elevation = 37.2 ft
  - Peak period = 6.1 seconds
- Probabilistic characterization
  - Much greater than 100-year event
- Replacement bridge
  - Built to maximum surge + wave
  - $300 million

U.S. Department of Transportation
Federal Highway Administration

Consequences

Katrina: I-10 Lake Pontchartrain



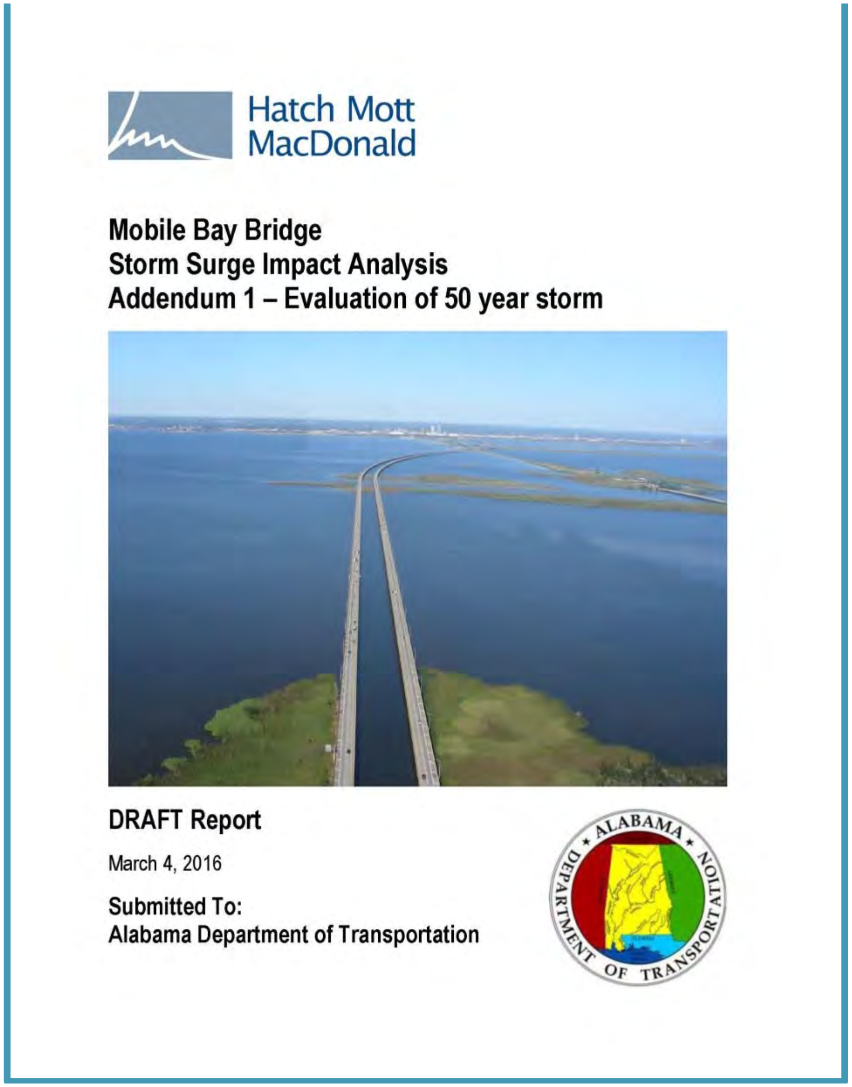
- Storm Surge
  - Design stillwater level = 12 ft
- Waves
  - Significant wave height = 6.0 ft
  - Maximum wave height = 12.6 ft
  - Maximum wave elevation = 22.8 ft
  - Peak period = unknown
- Probabilistic characterization
  - Katrina about a 130-year event
  - Used extreme event
- Replacement bridge
  - Built to extreme event surge + wave
  - $600 million

04

50 Year Storm Results

Storm Surge Impact Analysis

- Prepared by Hatch Mott MacDonald
- Modified Level I Analysis
- Also included future Sea Level Rise (SLR) Conditions
  - SLR 2017
  - SLR 2067
  - SLR 2117



Hatch Mott
MacDonald

Mobile Bay Bridge
Storm Surge Impact Analysis
Addendum 1 – Evaluation of 50 year storm

DRAFT Report

March 4, 2016

Submitted To:
Alabama Department of Transportation

Bayway Analysis

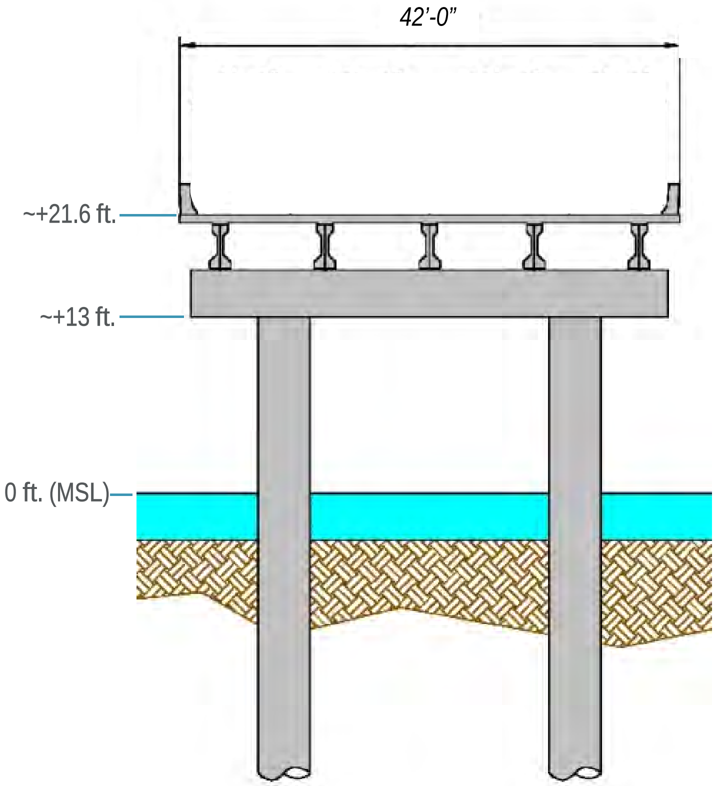
Combined Storm Surge and SLR Projections

Storm surge-frequency relation of the combined surge and SLR projections.

Return period [yr]	Storm surge [ft]		
	2017	**2067**	**2117**
10	6.6	7.5	9.3
25	9.1	10.3	12.1
50	11.0	12.2	14.0
100	12.8	14.0	15.8
500	16.7	17.9	19.7

Table Courtesy of Hatch Mott MacDonald

Storm Surge for 50 yr. 2067 SLR is similar to 100 yr. 2017 SLR



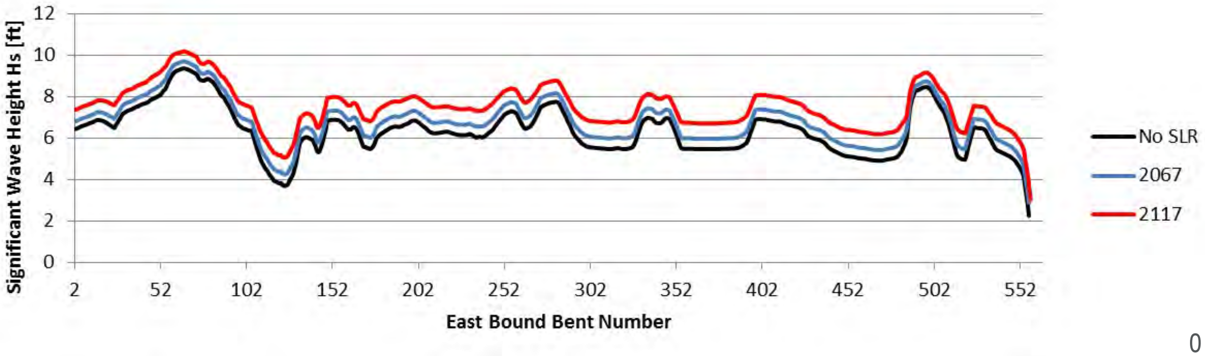
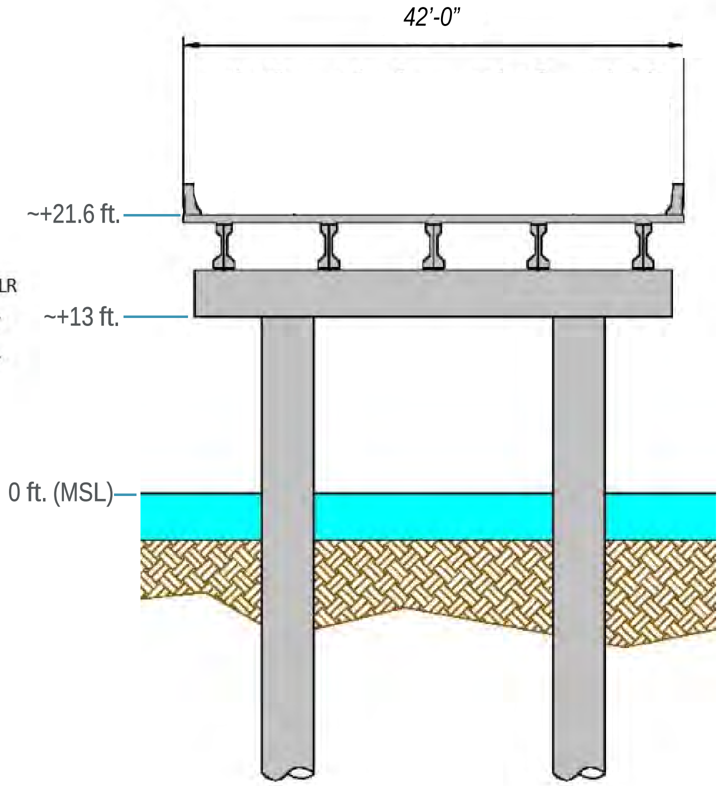


Figure Courtesy of Hatch Mott MacDonald



Bayway Analysis

Wave Heights – 50yr Wave Heights for 2017, 2067 and 2117 Sea-Levels

Bayway Analysis

Wave Heights – 5 Design Storms Present Day Sea-Level (2017)

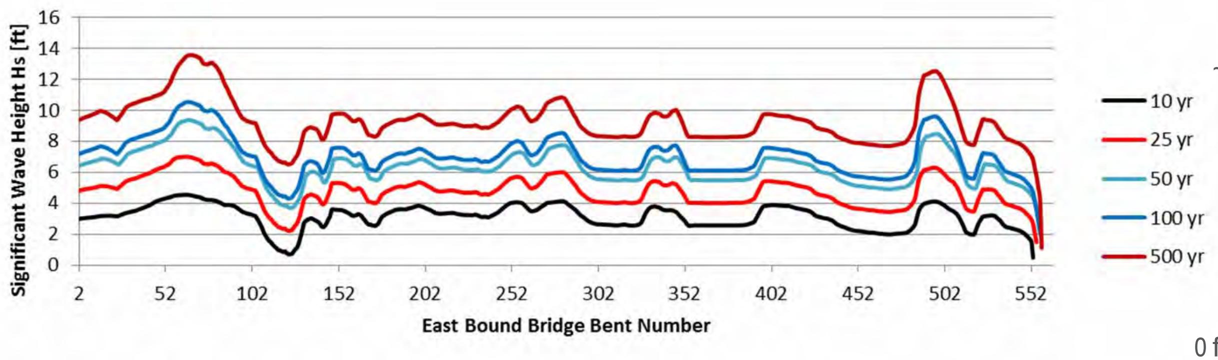
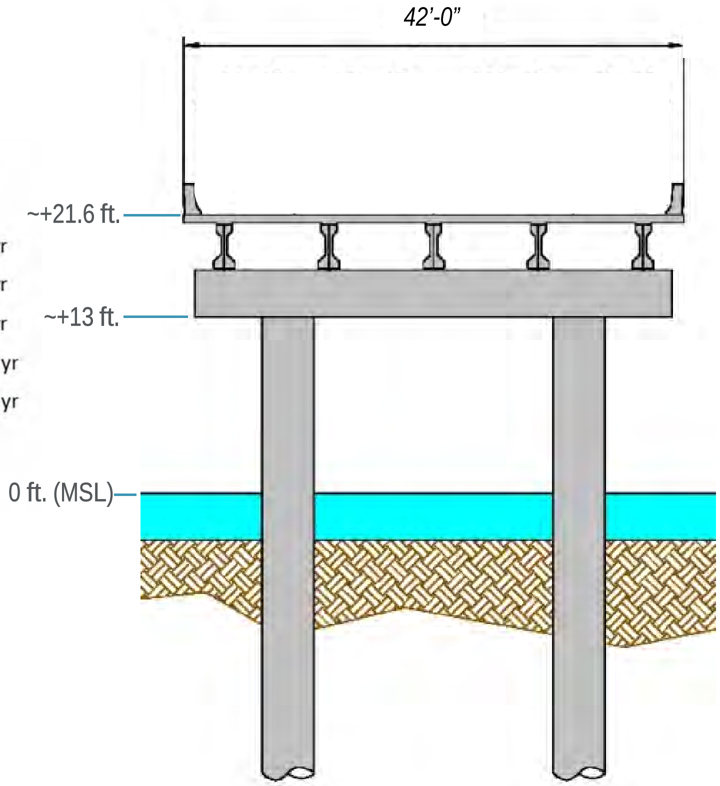


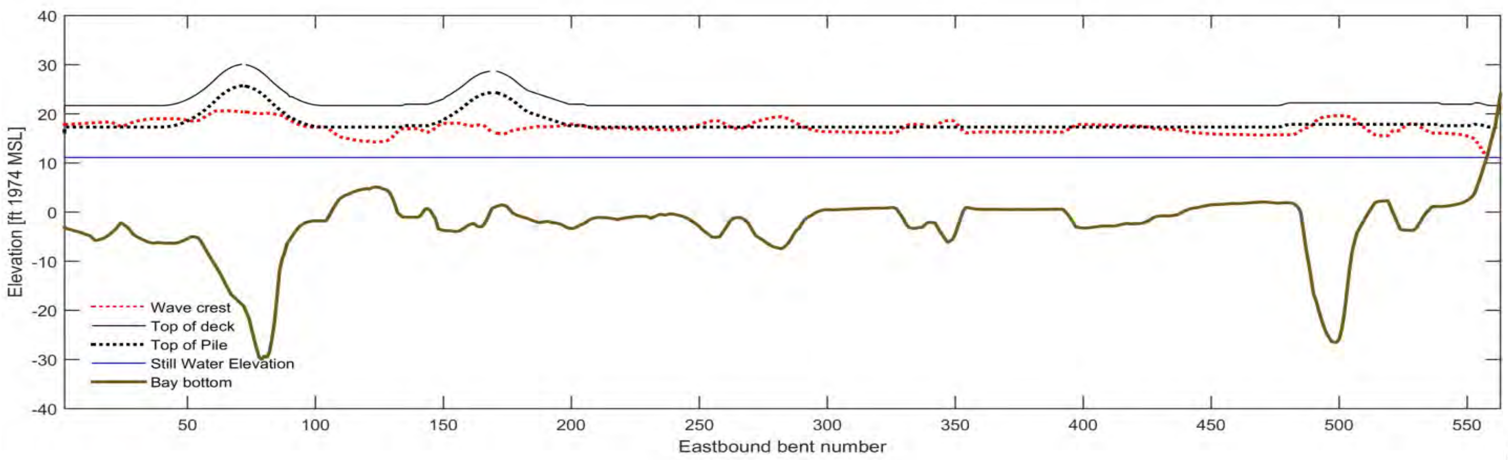
Figure Courtesy of Hatch Mott MacDonald

Wave Height for 50 yr. 2017 SLR is close to 100 yr. 2017 SLR



Bayway Analysis

25yr Storm and 2067 SLR Elevations



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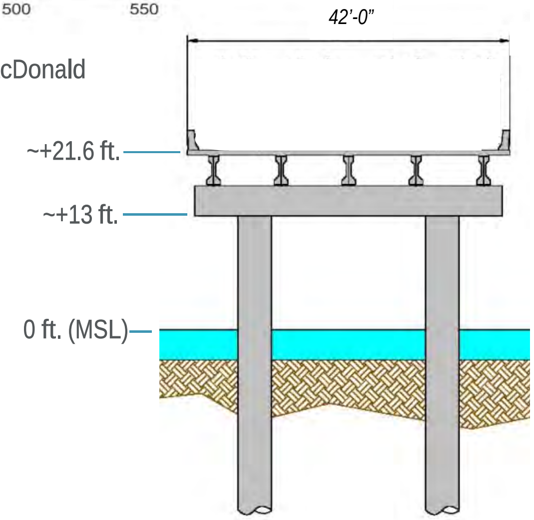


Still Water Elevation = Storm Surge at High Tide + SLR

SLR = 1.2 for 2067 Sea-Level

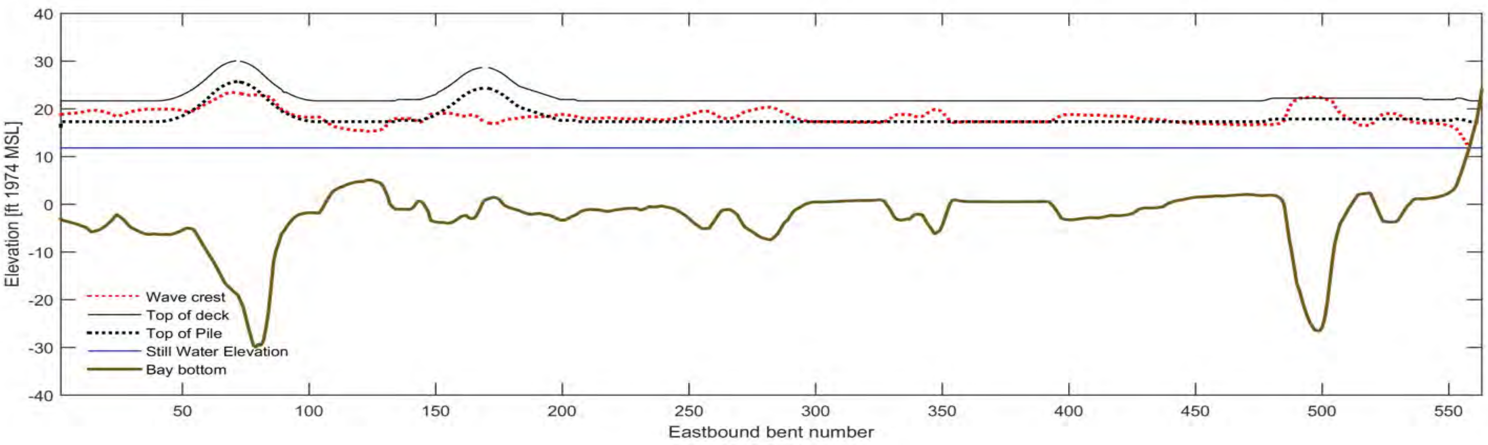
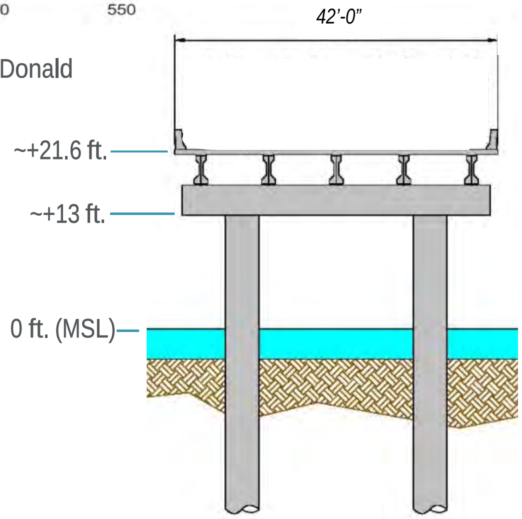


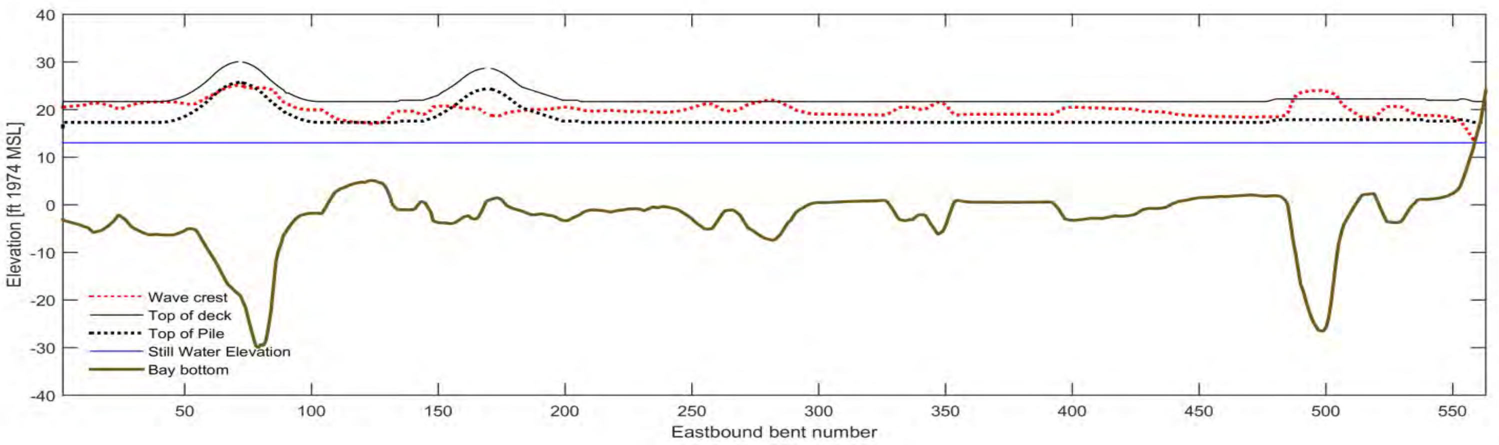
Figure Courtesy of Hatch Mott MacDonald

Still Water Elevation = Storm Surge at High Tide + SLR
SLR = 0 for 2017 Sea-Level

Bayway Analysis

50yr Storm and Present Day Sea-Level (2017) Elevations





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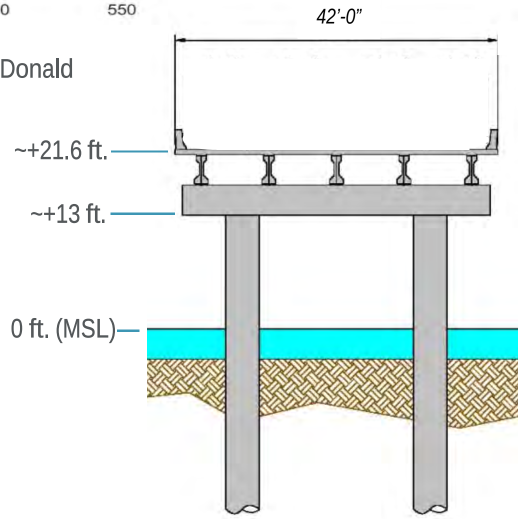
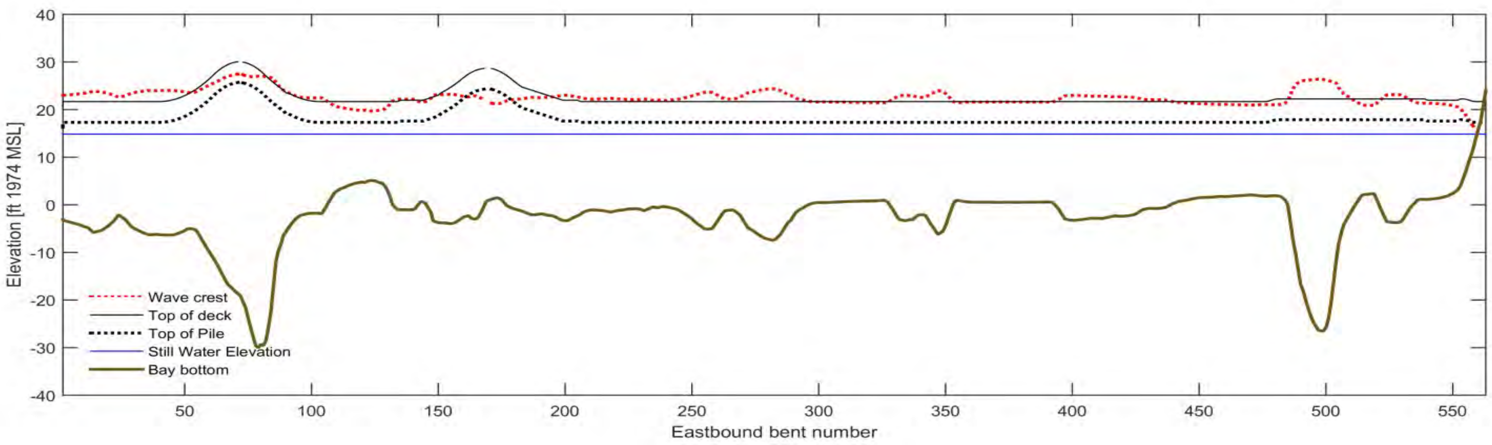


Still Water Elevation = Storm Surge at High Tide + SLR

SLR = 1.2 for 2067 Sea-Level

Bayway Analysis

50yr Storm and 2067 SLR Elevations



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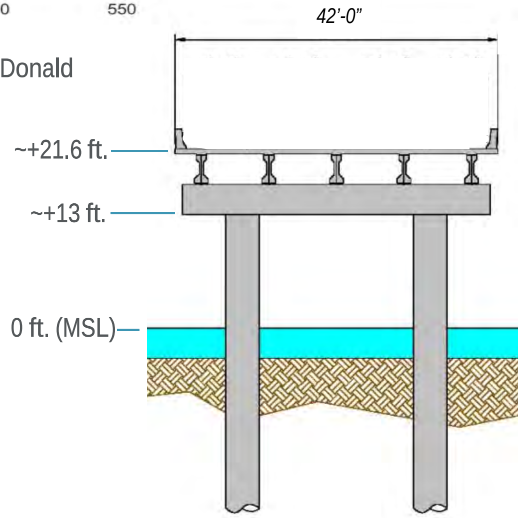
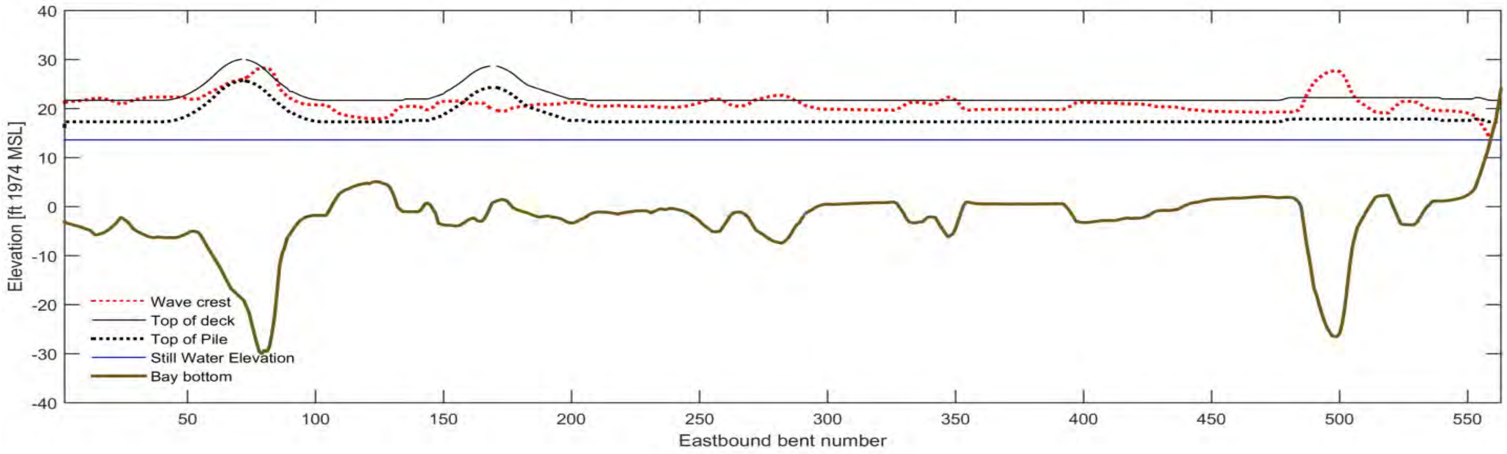


Still Water Elevation = Storm Surge at High Tide + SLR
SLR = 3.0 for 2117 Sea-Level

Bayway Analysis

50yr Storm and 2117 SLR Elevations



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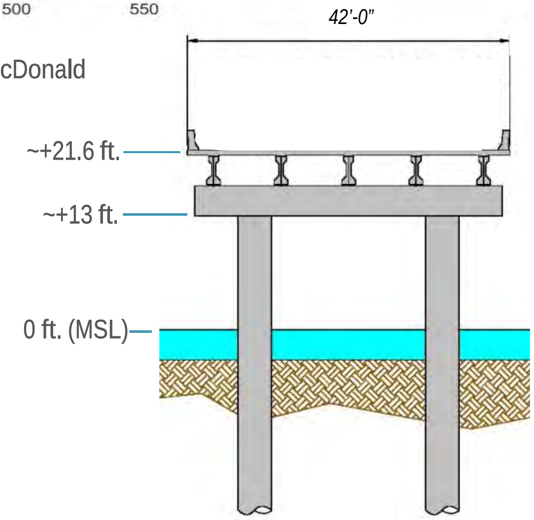
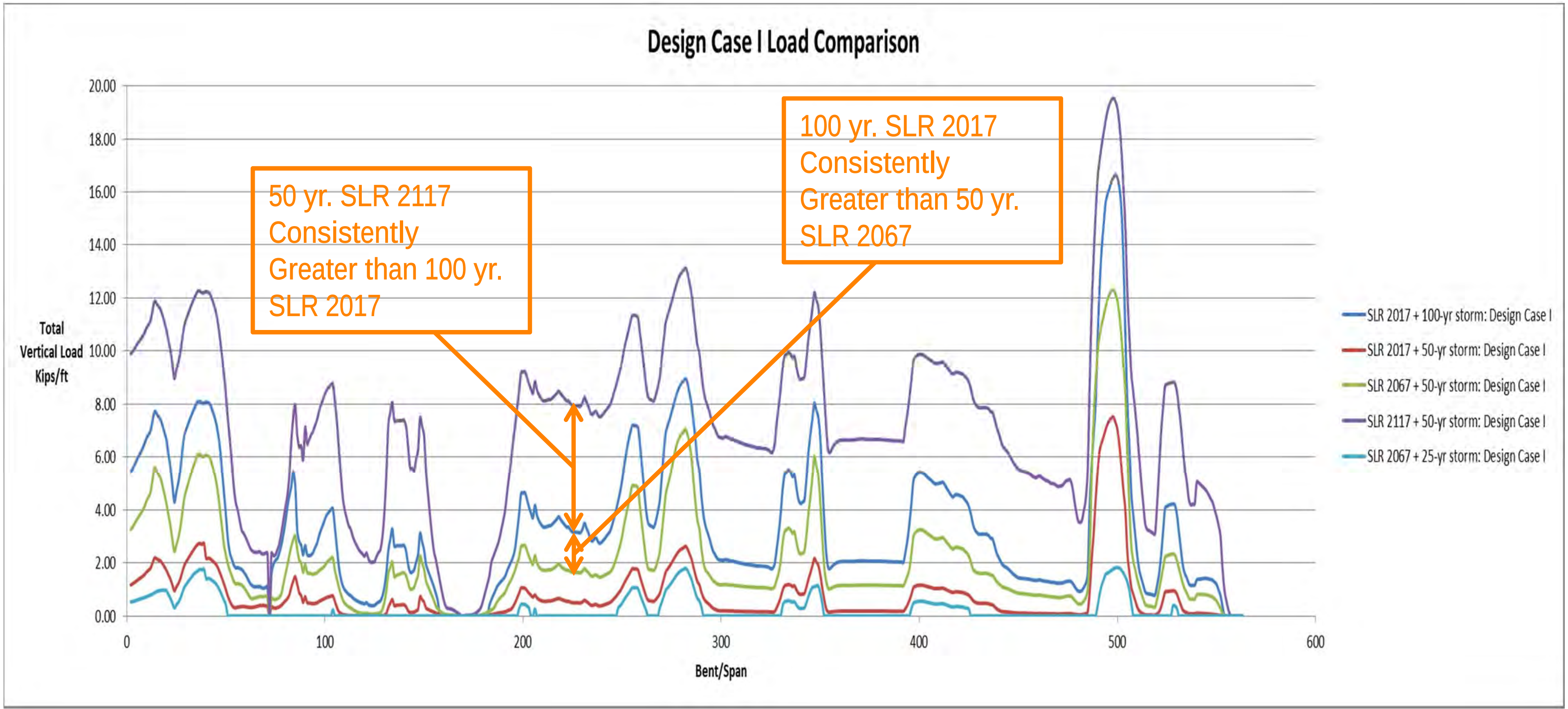


Still Water Elevation = Storm Surge at High Tide + SLR

SLR = 0 for 2017 Sea-Level

Bayway Analysis

100yr Present Day Sea-Level (2017) Elevations

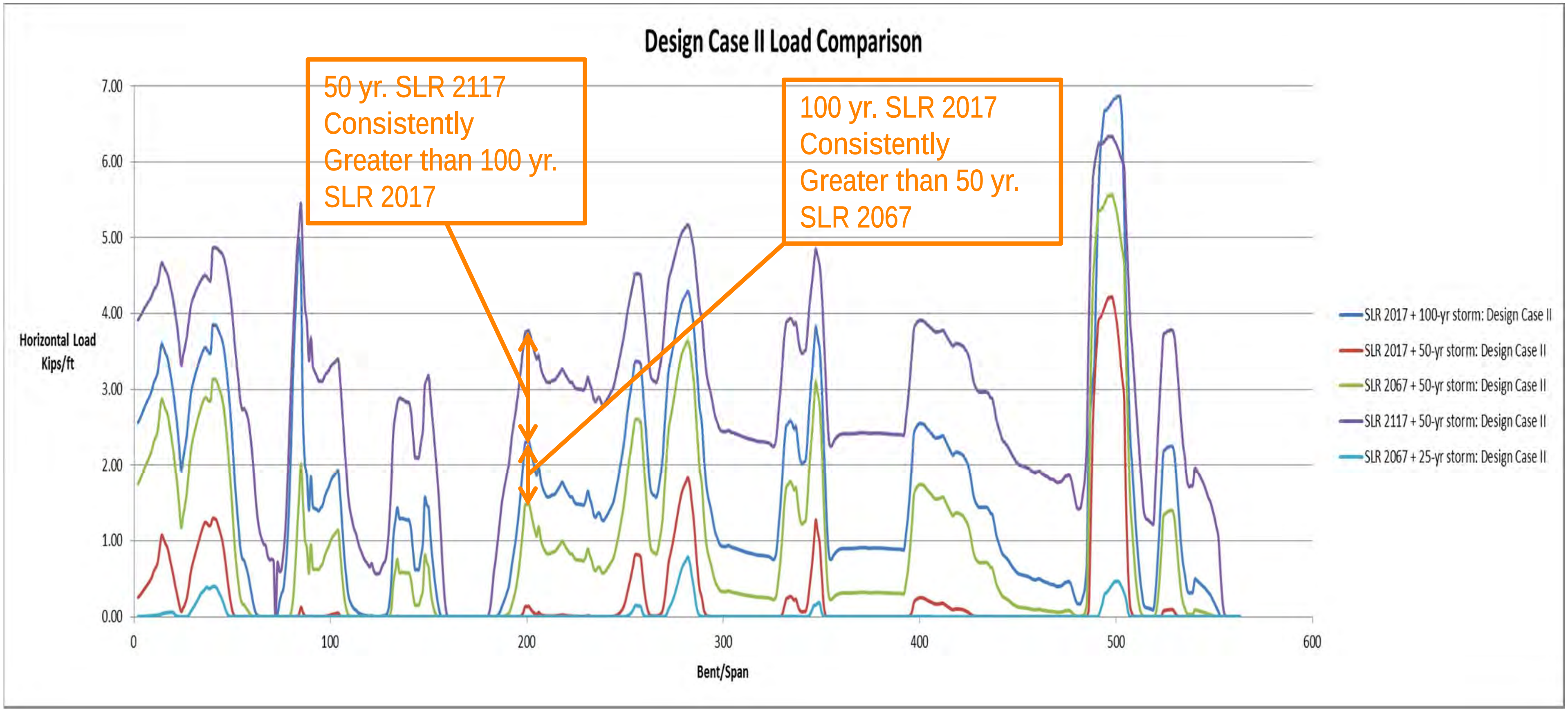


50 yr. SLR 2117 Consistently Greater than 100 yr. SLR 2017

100 yr. SLR 2017 Consistently Greater than 50 yr. SLR 2067

Bayway Analysis

Vertical Load Comparison



50 yr. SLR 2117 Consistently Greater than 100 yr. SLR 2017

100 yr. SLR 2017 Consistently Greater than 50 yr. SLR 2067

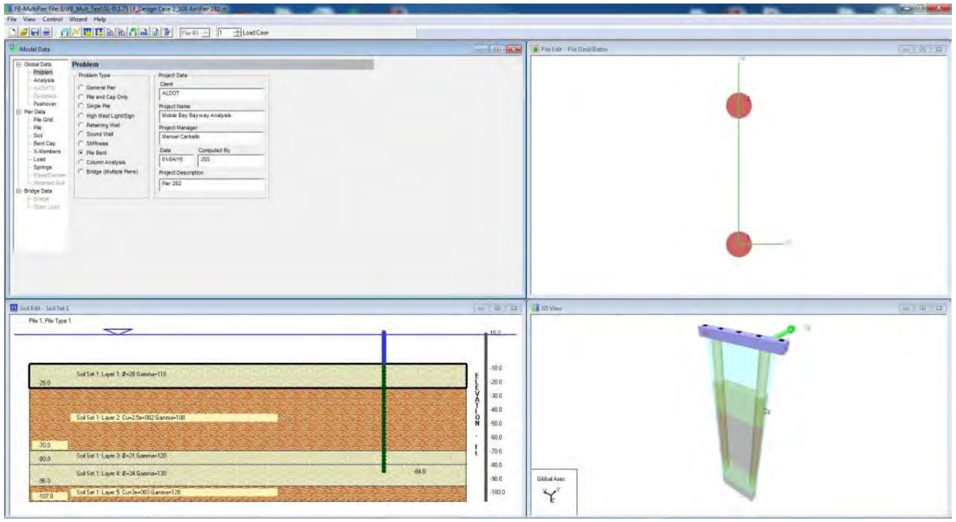
Bayway Analysis

Horizontal Load Comparison

05 Analytical Approach

Analytical Approach

- Initially developed a diagnostic model to analyze the structure in the As-built condition
- It is essential that this step be performed even if there are obvious vulnerabilities in order to establish a benchmark of member demands
- The purpose of this analysis was to evaluate the state of the structure and identify all possible failure modes for the "Design Storm"



Problem

Problem Type
○ General Pier
○ Pile and Cap Only
○ Single Pile
○ High Mast Light/Sign
○ Retaining Wall
○ Sound Wall
○ Stiffness
● Pile Bent
○ Column Analysis
○ Bridge (Multiple Piers)

Project Data
Client: ALDOT
Project Name: Mobile Bay Bayway Analysis
Project Manager: Manuel Carballo
Date: 01/04/16
Computed By: JSS
Project Description: Pier 282

Soil Set 1: Layer 1: Ø=28 Gamma=110
Soil Set 1: Layer 2: Cu=2.5e+002 Gamma=100
Soil Set 1: Layer 3: Ø=31 Gamma=120
Soil Set 1: Layer 4: Ø=34 Gamma=130
Soil Set 1: Layer 5: Cu=3e+003 Gamma=120

Analytical Approach

- A diagnostic model with the proposed retrofit was then ran. If a failure mechanism for the structural system still exists, additional retrofit measures are required
- If the retrofit model indicates there is no failure mechanism and that the associated member demands are significantly less than their capacities considered reducing the amount of retrofit and rerun the model
- This procedure was repeated until an optimal, or “preferred”, retrofit strategy was obtained

Limit States and Performance Levels

- Critical/essential-Strength Limit State should be used. Performance levels:
- “Service Immediate”
  - Sufficiently undamaged, stable and aligned for rescue and recovery after cursory inspection
  - Backfill behind abutments can be sacrificial
- “Repairable Damage”
  - Some repairs needed to go in service
  - Owner species outage duration
  - Load posting can be considered
  - Pre-positioned replacement spans may be used to meet outage limit

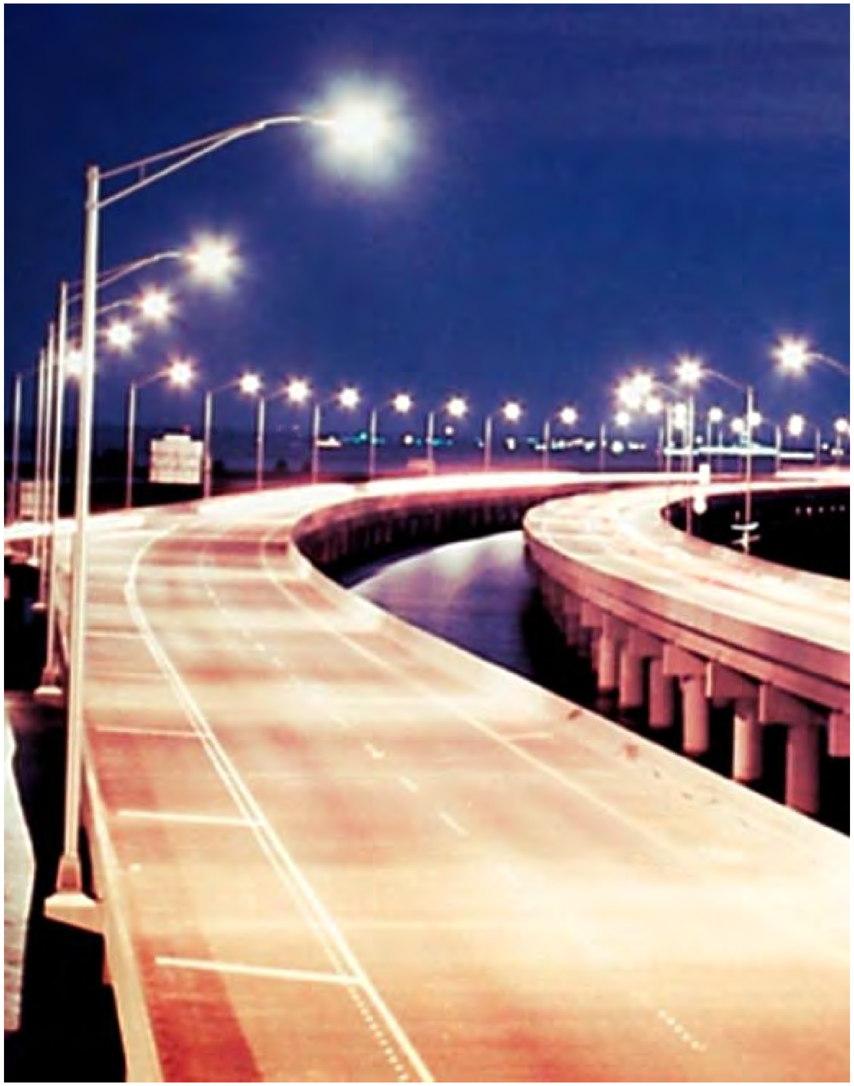
This is Our Performance Target

06

Retrofit Strategies and Approaches

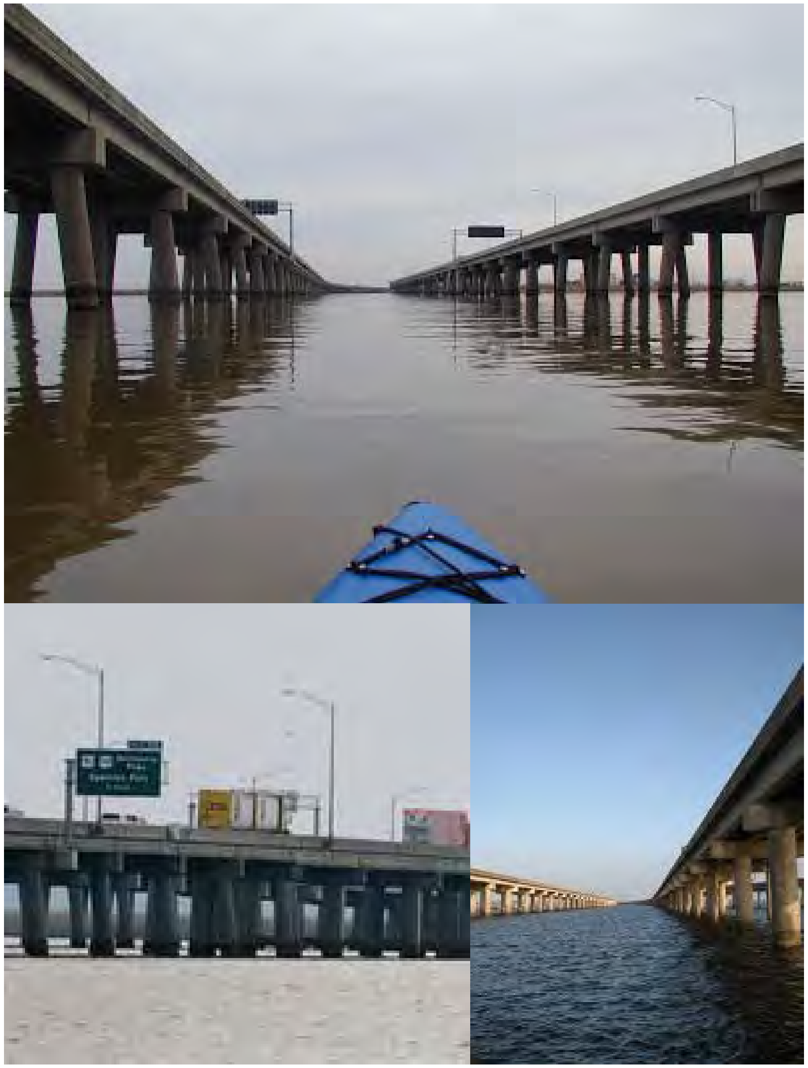
Approach to Retrofit Options

- Develop:
  - Method for **Screening** the Bayway structure's Vulnerability to Coastal Storms – Presented Results during Last Meeting
  - Method for **Evaluating** the Structures to Determine Specific Vulnerabilities
  - Potential **Retrofit Strategies**, Approaches and Measures



Screening

- Provide a Reasonably Simple method for Determining vulnerability to Coastal Storm Events.
- During our Last Meeting we Discussed Vulnerability of Superstructure and Superstructure Restraints to Uplift Loads and Lateral Loads on Substructure
- Performed Initial Assessments on Vulnerability of Superstructure Restraints and Foundation Capacities



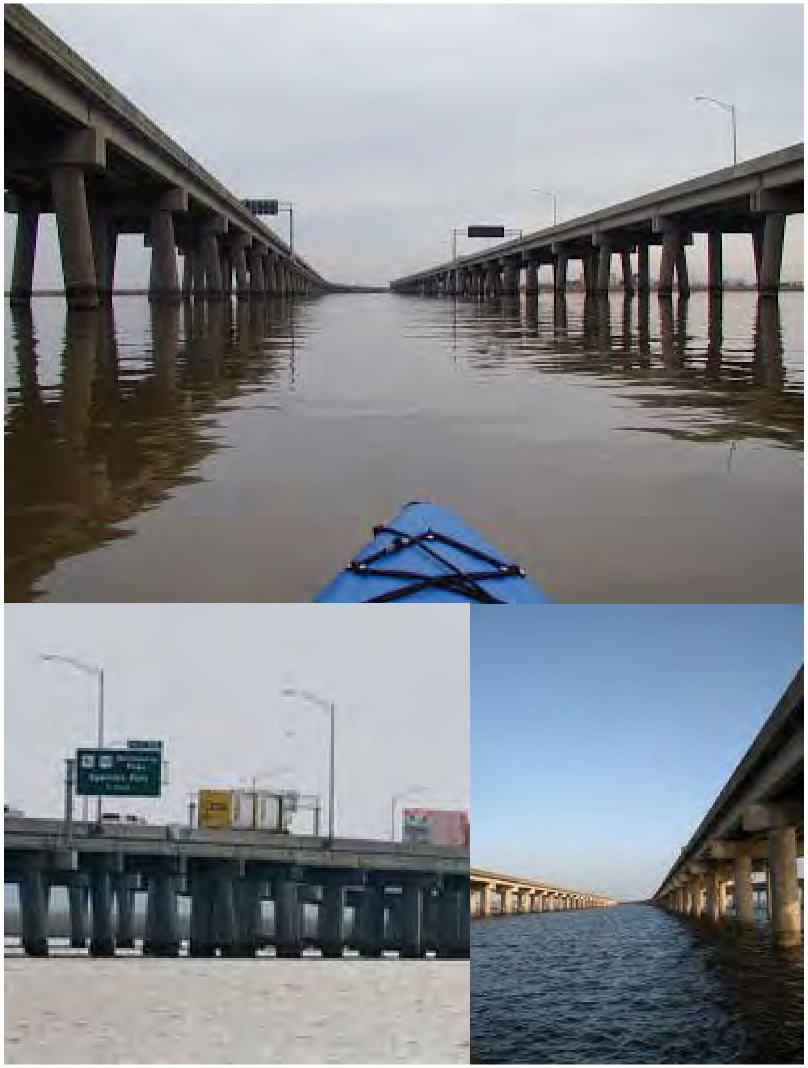
Evaluation

- Many Choices to be Made:
  - Selection of Design Event – What event or Events are to be Considered
- Retrofit Strategies – Which ones will be Used
- Levels of Analysis – Which of the Three levels of Analysis is to be Used?

Evaluations

Selection of Design Event

- AASHTO Guide Specifications are based on a 100-year Return Period Design Event.
- 100-year Event has approximately a 50 percent Chance of Exceedence (52.9%) during the 75-year Life of a New Bridge
  - $R = 1-(1-1/T)^n$
    - T = 100 yr (return interval)
    - n = 75 yr (expected life)
- For an Existing Bridge the First Step is to Evaluate the Feasibility of Retrofitting it to Withstand the Wave Forces (100-year Event)



Evaluations

Selection of Design Event

- If these retrofits are Impractical and/or cost Prohibitive, two Approaches may be Considered:
  1. Account for the Bridge's Remaining Service Life and Adjust the Design Event such that it Approximates the Mean Value during the Remaining life of the Existing Bridge

N/0.69 = RP

RP = Design Event Return Period

N = Remaining Service Life of the Bridge

Bridge is 40 Years Old

If we Get 35 more yrs. Service Life

N	RP
5	7
10	14
15	22
20	29
25	36
30	43
35	51
40	58
45	65
50	72

Evaluations

Selection of Design Event

- If we want to Assess Various Levels of Risk

	N - Expected Life (Years)						
RP - Return (Years)	**25**	**30**	**40**	**50**	**60**	**75**	**100**
25	64	71	80	87	91	95	98
50	40	45	55	64	70	78	87
75	29	33	42	49	55	63	74
100	22	26	33	39	45	53	63

AASHTO

Probability of Exceedance in Expected Life (%)

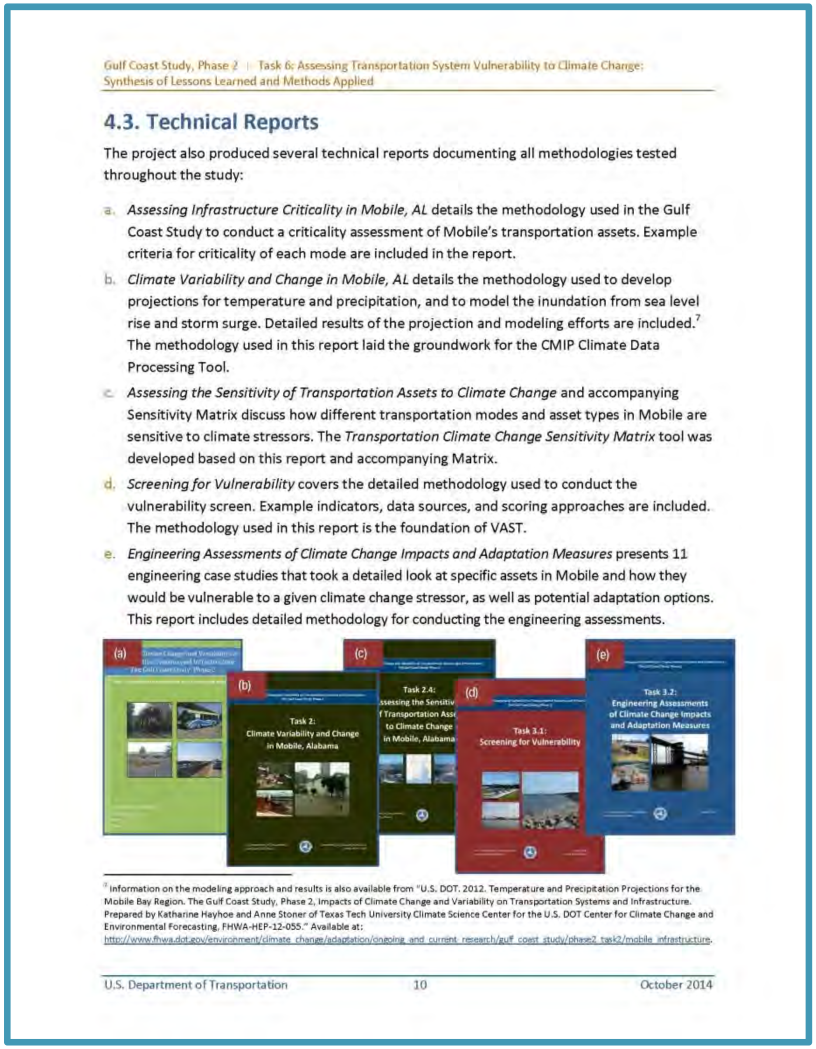
Evaluations

Selection of Design Event

- If these retrofits are Impractical and/or cost Prohibitive, two Approaches may be Considered:
  2. Perform a More Rigorous Optimization Routine regarding a Cost Assessment Model

Gulf Coast Study

- To better understand potential climate change impacts on transportation infrastructure and identify adaptation strategies, the U.S. Department of Transportation (U.S. DOT) conducted a comprehensive, multi-phase study of climate change impacts in the Central Gulf Coast region.
- This multi-modal study is sponsored by the U.S. DOT's Center for Climate Change and Environmental Forecasting in partnership with the U.S. Geological Survey (USGS) and is managed by FHWA.



Gulf Coast Study, Phase 2 | Task 6: Assessing Transportation System Vulnerability to Climate Change: Synthesis of Lessons Learned and Methods Applied

4.3. Technical Reports

The project also produced several technical reports documenting all methodologies tested throughout the study:

a. *Assessing Infrastructure Criticality in Mobile, AL* details the methodology used in the Gulf Coast Study to conduct a criticality assessment of Mobile's transportation assets. Example criteria for criticality of each mode are included in the report.

b. *Climate Variability and Change in Mobile, AL* details the methodology used to develop projections for temperature and precipitation, and to model the inundation from sea level rise and storm surge. Detailed results of the projection and modeling efforts are included.[7] The methodology used in this report laid the groundwork for the CMIP Climate Data Processing Tool.

c. *Assessing the Sensitivity of Transportation Assets to Climate Change* and accompanying Sensitivity Matrix discuss how different transportation modes and asset types in Mobile are sensitive to climate stressors. The *Transportation Climate Change Sensitivity Matrix* tool was developed based on this report and accompanying Matrix.

d. *Screening for Vulnerability* covers the detailed methodology used to conduct the vulnerability screen. Example indicators, data sources, and scoring approaches are included. The methodology used in this report is the foundation of VAST.

e. *Engineering Assessments of Climate Change Impacts and Adaptation Measures* presents 11 engineering case studies that took a detailed look at specific assets in Mobile and how they would be vulnerable to a given climate change stressor, as well as potential adaptation options. This report includes detailed methodology for conducting the engineering assessments.

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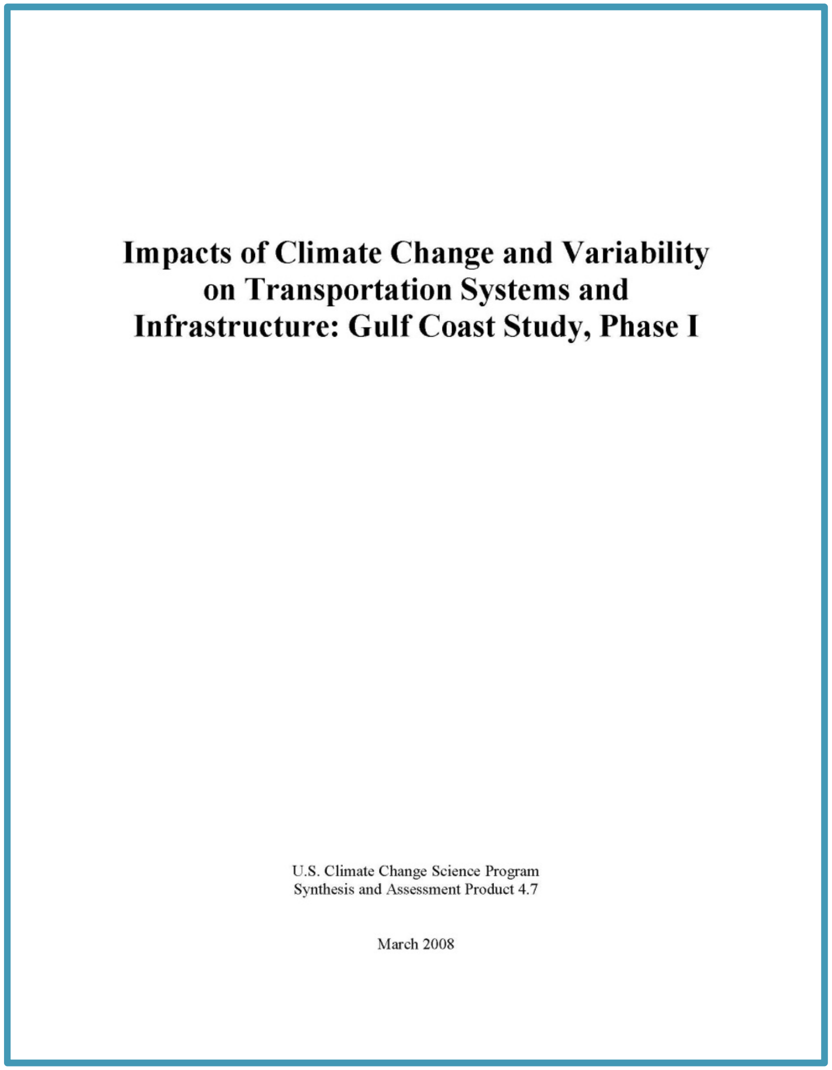
[7] Information on the modeling approach and results is also available from "U.S. DOT. 2012. Temperature and Precipitation Projections for the Mobile Bay Region. The Gulf Coast Study, Phase 2, Impacts of Climate Change and Variability on Transportation Systems and Infrastructure. Prepared by Katharine Hayhoe and Anne Stoner of Texas Tech University Climate Science Center for the U.S. DOT Center for Climate Change and Environmental Forecasting, FHWA-HEP-12-055." Available at: http://www.fhwa.dot.gov/environment/climate_change/adaptation/ongoing_and_current_research/gulf_coast_study/phase2_task2/mobile_infrastructure.

U.S. Department of Transportation 10 October 2014

Gulf Coast Study

- Phase 1 (completed in 2008) examined the impacts of climate change on transportation infrastructure at a regional scale, investigating risks and impacts on coastal ports, road, air, rail, and public transit systems in the central Gulf Coast, with a study area stretching from Houston/Galveston, Texas, to Mobile, Alabama.
- The study assessed likely changes in temperature and precipitation patterns, sea level rise, and increasing severity and frequency of tropical storms.

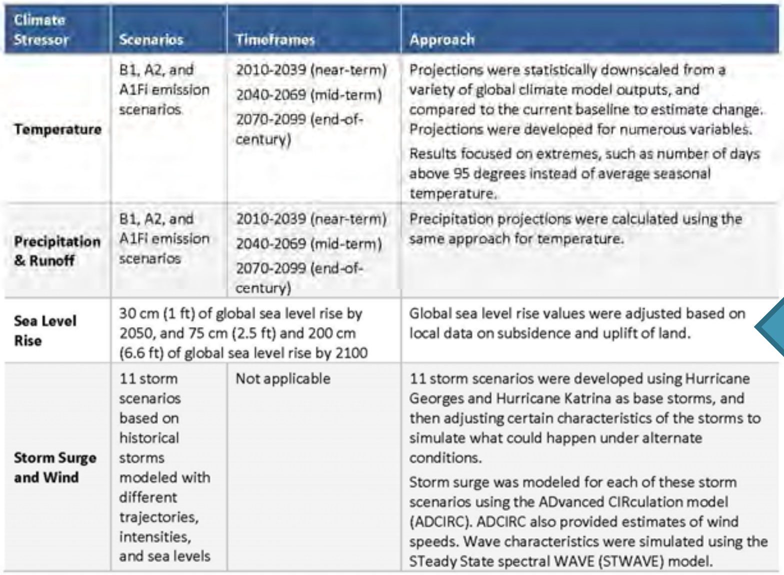


Impacts of Climate Change and Variability on Transportation Systems and Infrastructure: Gulf Coast Study, Phase I

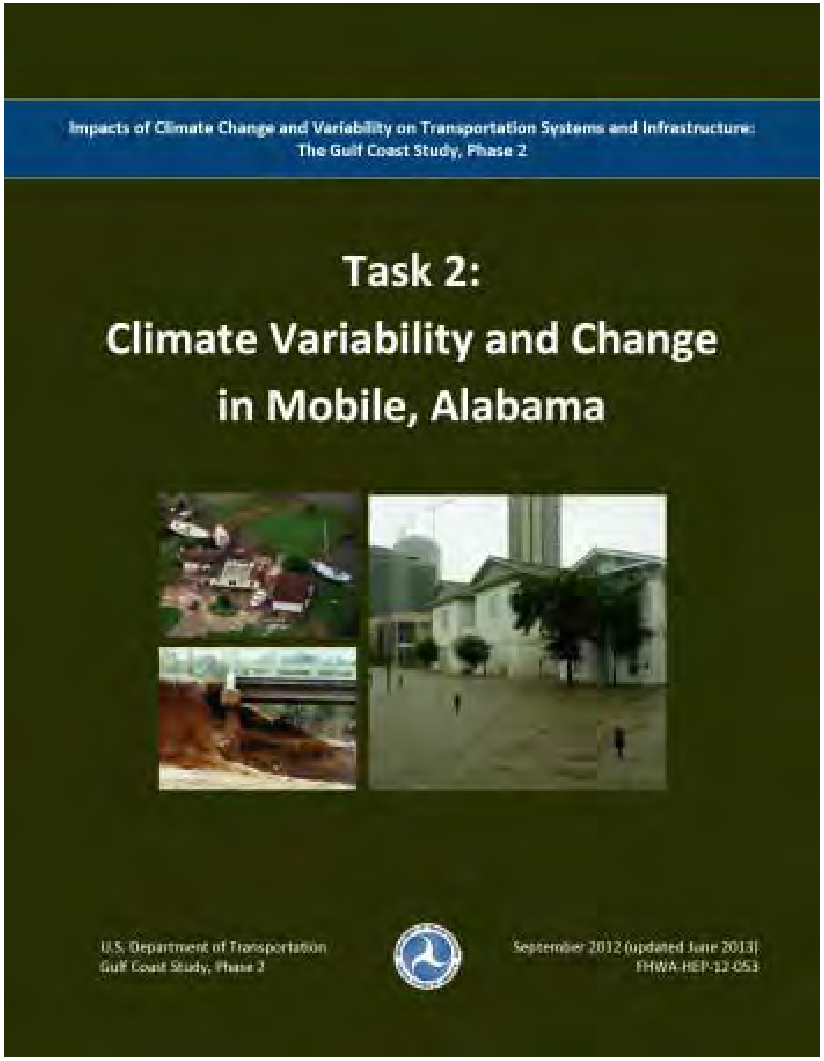
U.S. Climate Change Science Program
Synthesis and Assessment Product 4.7

March 2008

Gulf Coast Study



Climate Stressor	Scenarios	Timeframes	Approach
Temperature	B1, A2, and A1Fi emission scenarios	2010-2039 (near-term) 2040-2069 (mid-term) 2070-2099 (end-of-century)	Projections were statistically downscaled from a variety of global climate model outputs, and compared to the current baseline to estimate change. Projections were developed for numerous variables. Results focused on extremes, such as number of days above 95 degrees instead of average seasonal temperature.
Precipitation & Runoff	B1, A2, and A1Fi emission scenarios	2010-2039 (near-term) 2040-2069 (mid-term) 2070-2099 (end-of-century)	Precipitation projections were calculated using the same approach for temperature.
Sea Level Rise	30 cm (1 ft) of global sea level rise by 2050, and 75 cm (2.5 ft) and 200 cm (6.6 ft) of global sea level rise by 2100		Global sea level rise values were adjusted based on local data on subsidence and uplift of land.
Storm Surge and Wind	11 storm scenarios based on historical storms modeled with different trajectories, intensities, and sea levels	Not applicable	11 storm scenarios were developed using Hurricane Georges and Hurricane Katrina as base storms, and then adjusting certain characteristics of the storms to simulate what could happen under alternate conditions. Storm surge was modeled for each of these storm scenarios using the ADvanced CIRculation model (ADCIRC). ADCIRC also provided estimates of wind speeds. Wave characteristics were simulated using the STeady State spectral WAVE (STWAVE) model.



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

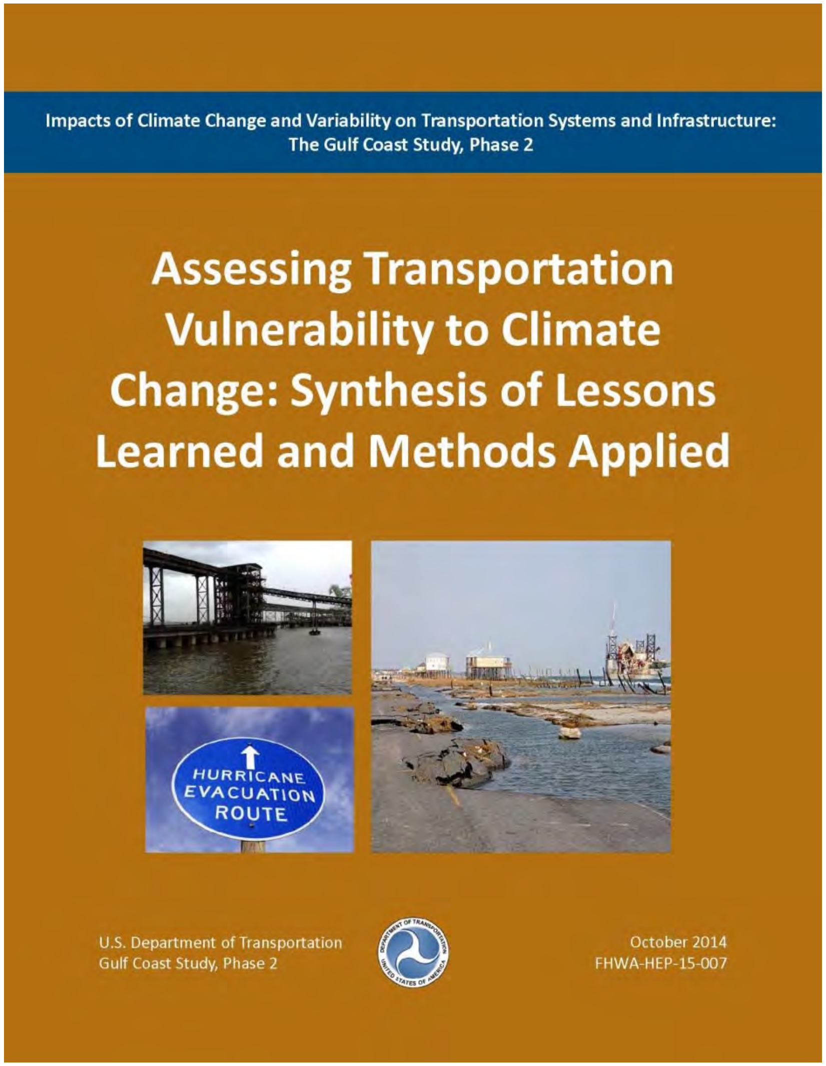
Task 2:
Climate Variability and Change
in Mobile, Alabama

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

September 2012 (updated June 2013)
FHWA-HEP-12-053

Gulf Coast Study

- Phase 2 focused on the Mobile, Alabama region, with the goal of enhancing regional decision makers' ability to understand potential impacts on specific critical components of infrastructure and to evaluate adaptation options.
- U.S. DOT assessed the vulnerability of the most critical transportation assets to climate change impacts. U.S. DOT then developed risk management tools to help transportation system planners, owners, and operators determine which systems and assets to protect and how to do so.



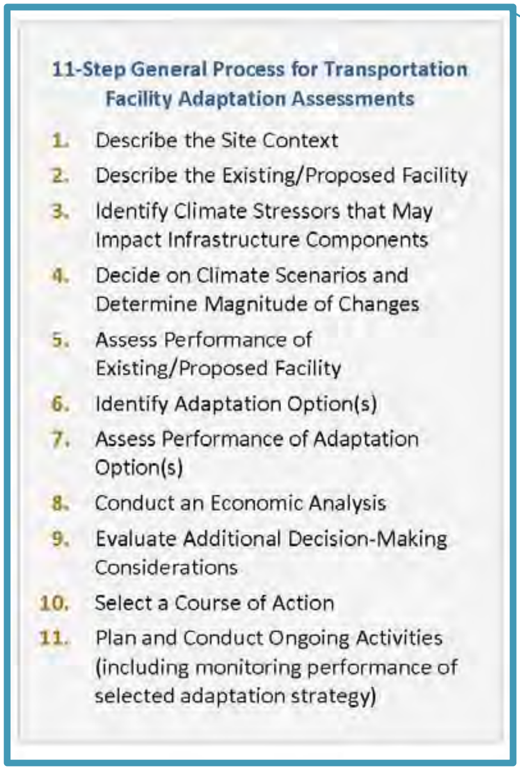
Impacts of Climate Change and Variability on Transportation Systems and Infrastructure:
The Gulf Coast Study, Phase 2

Assessing Transportation Vulnerability to Climate Change: Synthesis of Lessons Learned and Methods Applied

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

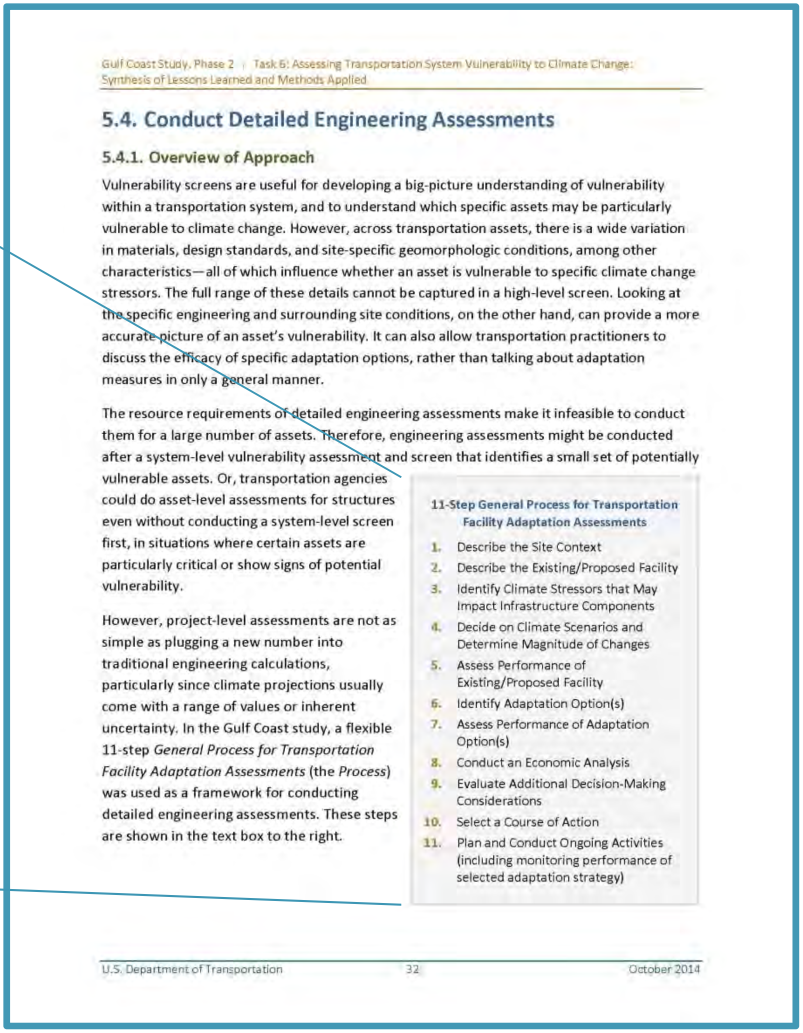
October 2014
FHWA-HEP-15-007

Gulf Coast Study



11-Step General Process for Transportation Facility Adaptation Assessments

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 Magnitude of Changes
5. Assess Performance of Existing/Proposed Facility
6. Identify Adaptation Option(s)
7. Assess Performance of 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 (including monitoring performance of selected adaptation strategy)



Gulf Coast Study, Phase 2 | Task 6: Assessing Transportation System Vulnerability to Climate Change: Synthesis of Lessons Learned and Methods Applied

5.4. Conduct Detailed Engineering Assessments

5.4.1. Overview of Approach

Vulnerability screens are useful for developing a big-picture understanding of vulnerability within a transportation system, and to understand which specific assets may be particularly vulnerable to climate change. However, across transportation assets, there is a wide variation in materials, design standards, and site-specific geomorphologic conditions, among other characteristics—all of which influence whether an asset is vulnerable to specific climate change stressors. The full range of these details cannot be captured in a high-level screen. Looking at the specific engineering and surrounding site conditions, on the other hand, can provide a more accurate picture of an asset's vulnerability. It can also allow transportation practitioners to discuss the efficacy of specific adaptation options, rather than talking about adaptation measures in only a general manner.

The resource requirements of detailed engineering assessments make it infeasible to conduct them for a large number of assets. Therefore, engineering assessments might be conducted after a system-level vulnerability assessment and screen that identifies a small set of potentially vulnerable assets. Or, transportation agencies could do asset-level assessments for structures even without conducting a system-level screen first, in situations where certain assets are particularly critical or show signs of potential vulnerability.

However, project-level assessments are not as simple as plugging a new number into traditional engineering calculations, particularly since climate projections usually come with a range of values or inherent uncertainty. In the Gulf Coast study, a flexible 11-step *General Process for Transportation Facility Adaptation Assessments* (the *Process*) was used as a framework for conducting detailed engineering assessments. These steps are shown in the text box to the right.

11-Step General Process for Transportation Facility Adaptation Assessments

1. Describe the Site Context
2. Describe the Existing/Proposed Facility
3. Identify Climate Stressors that May Impact Infrastructure Components
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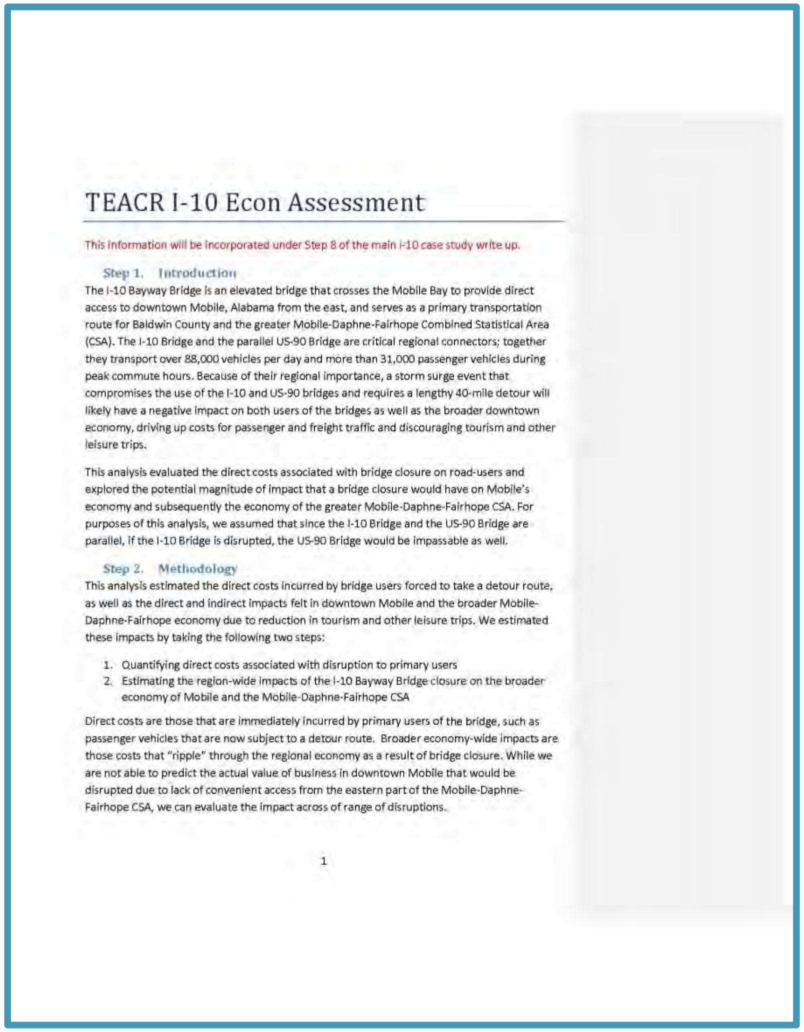
10. Select a Course of Action
11. Plan and Conduct Ongoing Activities (including monitoring performance of selected adaptation strategy)

U.S. Department of Transportation 32 October 2014

Cost Assessment

- Research Team Found that Loss of the I-10 Bridge could Result in Significant Costs to Daily Users of the Bridge as well As the Region-Wide Economy
- Cost to Primary Users: $1,130,804 per Day
- Daily Loss in Industry Activity: $495,000



TEACR I-10 Econ Assessment

This information will be incorporated under Step 8 of the main I-10 case study write up.

Step 1. Introduction

The I-10 Bayway Bridge is an elevated bridge that crosses the Mobile Bay to provide direct access to downtown Mobile, Alabama from the east, and serves as a primary transportation route for Baldwin County and the greater Mobile-Daphne-Fairhope Combined Statistical Area (CSA). The I-10 Bridge and the parallel US-90 Bridge are critical regional connectors; together they transport over 88,000 vehicles per day and more than 31,000 passenger vehicles during peak commute hours. Because of their regional importance, a storm surge event that compromises the use of the I-10 and US-90 bridges and requires a lengthy 40-mile detour will likely have a negative impact on both users of the bridges as well as the broader downtown economy, driving up costs for passenger and freight traffic and discouraging tourism and other leisure trips.

This analysis evaluated the direct costs associated with bridge closure on road-users and explored the potential magnitude of impact that a bridge closure would have on Mobile's economy and subsequently the economy of the greater Mobile-Daphne-Fairhope CSA. For purposes of this analysis, we assumed that since the I-10 Bridge and the US-90 Bridge are parallel, if the I-10 Bridge is disrupted, the US-90 Bridge would be impassable as well.

Step 2. Methodology

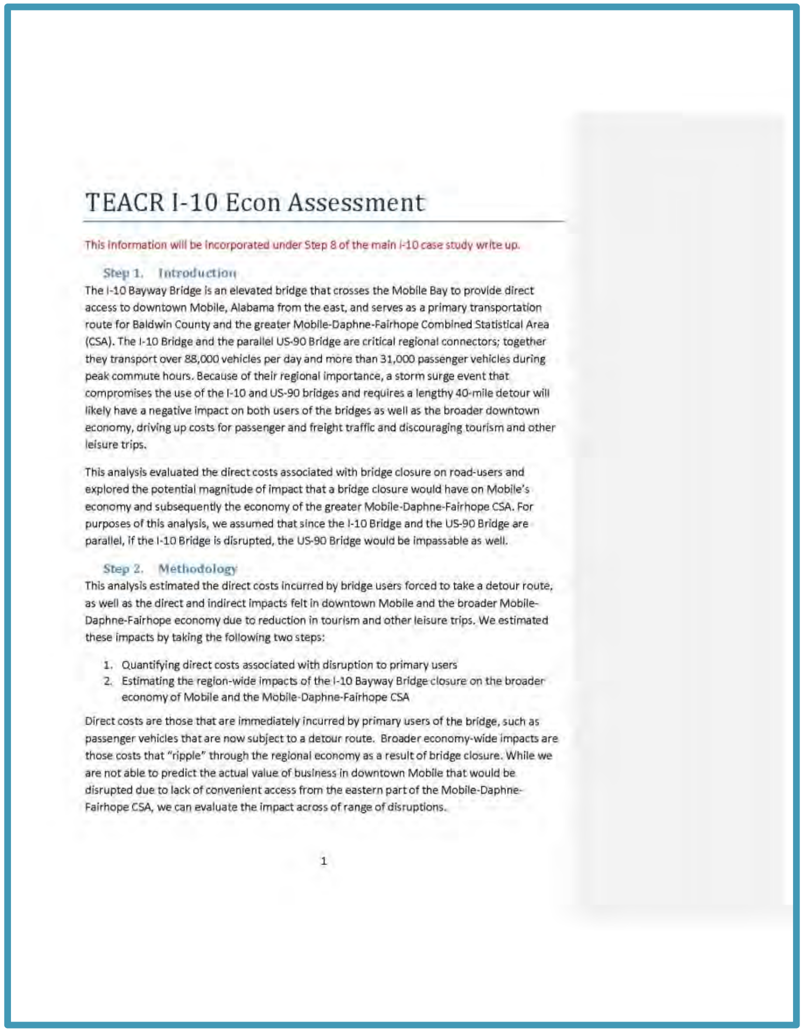
This analysis estimated the direct costs incurred by bridge users forced to take a detour route, as well as the direct and indirect impacts felt in downtown Mobile and the broader Mobile-Daphne-Fairhope economy due to reduction in tourism and other leisure trips. We estimated these impacts by taking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

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

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

Cost Assessment

- Cost-Effectiveness can be Used to Evaluate and Determine Appropriate Retrofit Strategies
- This Cost Assessment Procedure would be used to Evaluate Existing Bridges where Retrofit Could not be Fulfilled due to Unreasonable or Prohibitively high Costs
- <u>Does not Address the Issue of what is an Appropriate Level of Safety; its focus is on the Ratio of Benefits to Cost</u>



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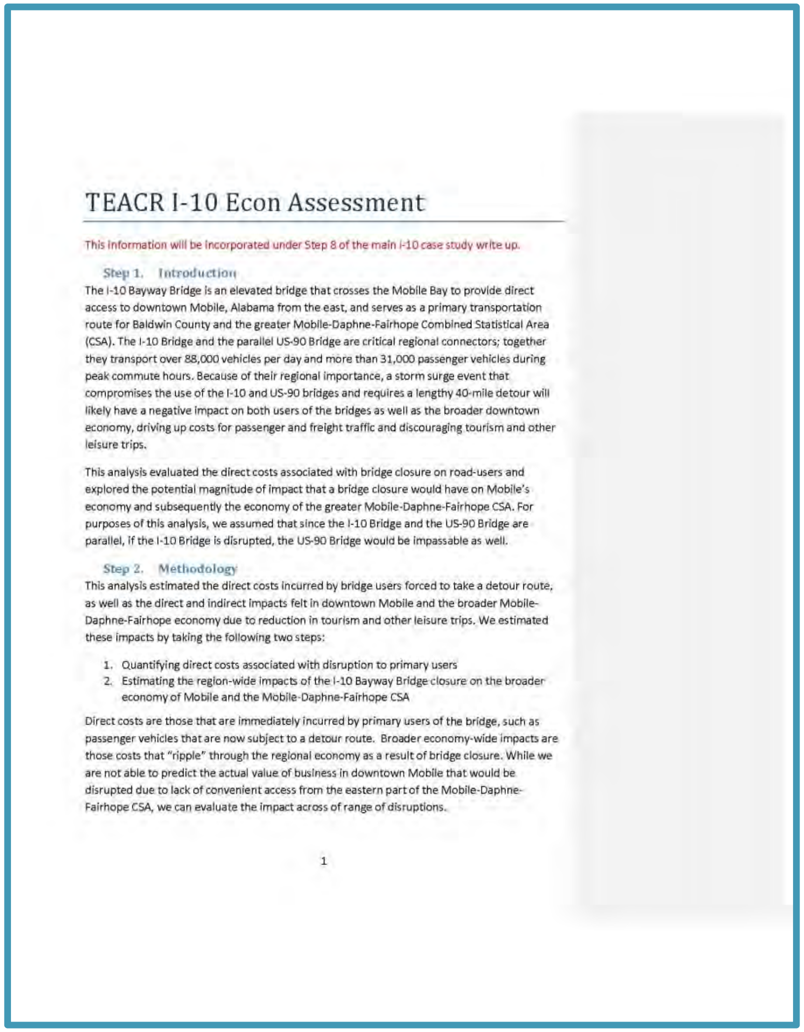
1

Cost Assessment

- Cost of Strengthening the Bridge (or Selected Components) is Compared to the Benefits of Risk Reduction
- Present Worth of the Disruption Cost (PW), for each Year of the Anticipated Bridge Life is Compared Against the Total Present Worth of the Costs to Replace ad Maintain the Bridge Structure, or the Retrofit Measures
- Present Worth of Costs and Benefits should be Computed over a Specific Time Period (life, or Remaining Life of the Bridge)



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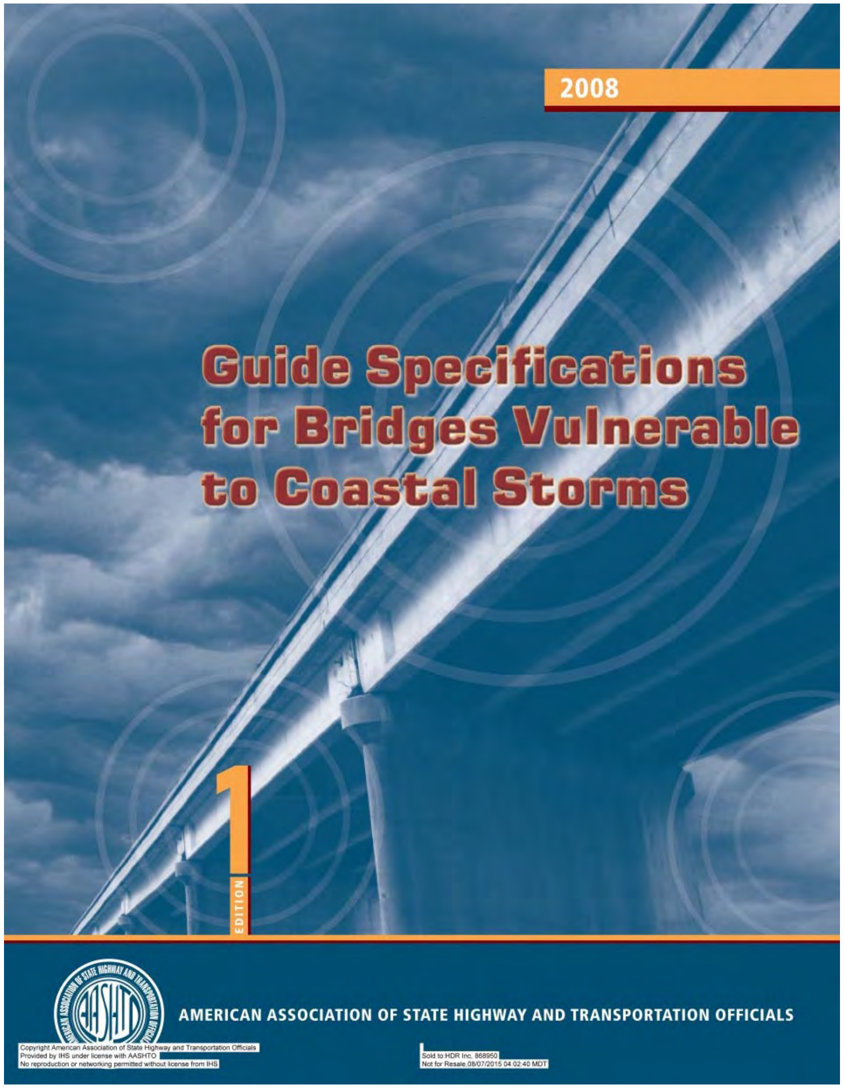
Evaluation

Level of Analysis

- Initial Studies are done for a Level I Analysis
  - SLR for 2017, 2067 and 2117 were Calculated
  - 10yr, 25yr, 50yr, 100yr and 500yr return Periods

- Level III Analysis will be Evaluated later



2008

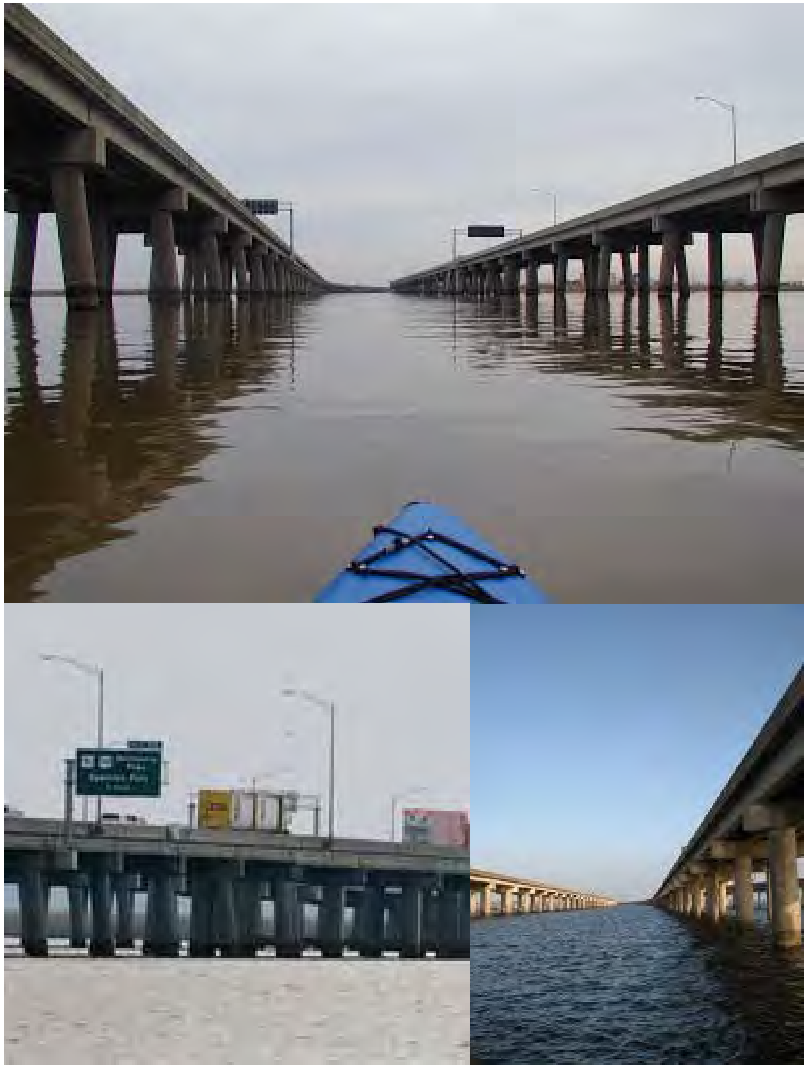
Guide Specifications for Bridges Vulnerable to Coastal Storms

1 EDITION

AMERICAN ASSOCIATION OF STATE HIGHWAY AND TRANSPORTATION OFFICIALS

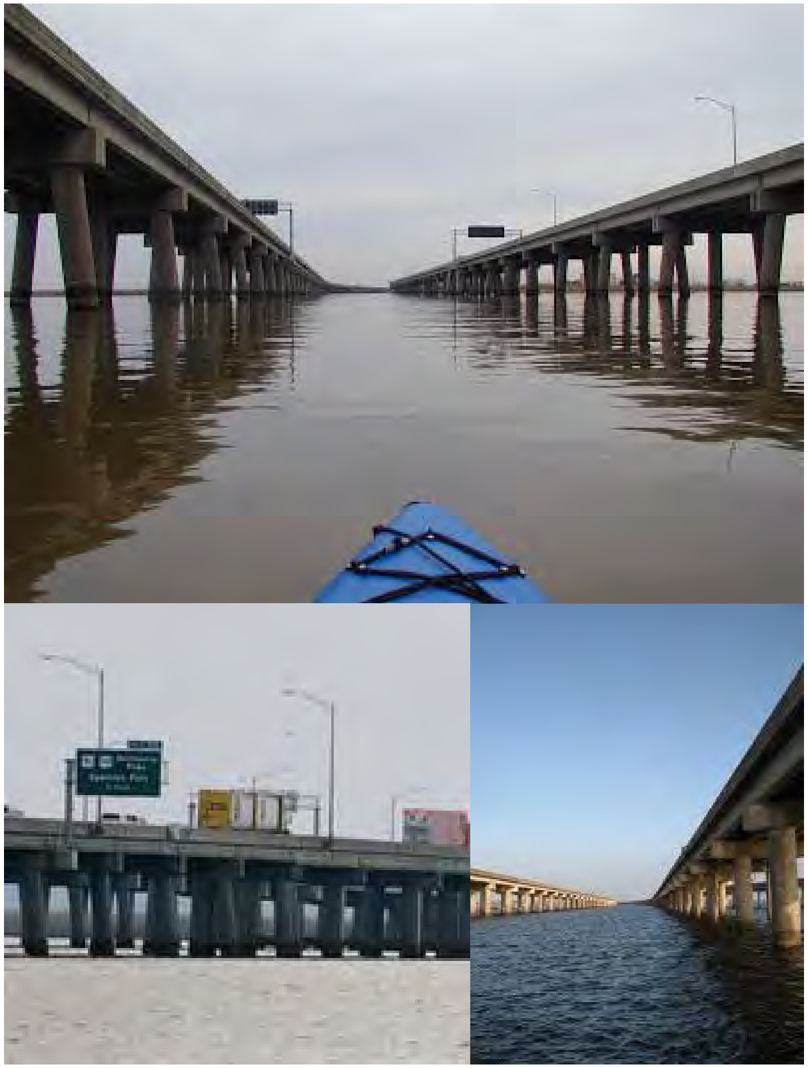
Retrofit Strategies and Approaches

1. Strategy
    - Bridge designated as “critical/essential”
    - Bridge to be “service immediate”
    - Maintenance/Durability
    - Constructibility
    - No Weakening of Existing Piles - No drilling into Precast Cylinder Piles
    - Ignore uplift and horizontal restraint capacity provided by original bearings



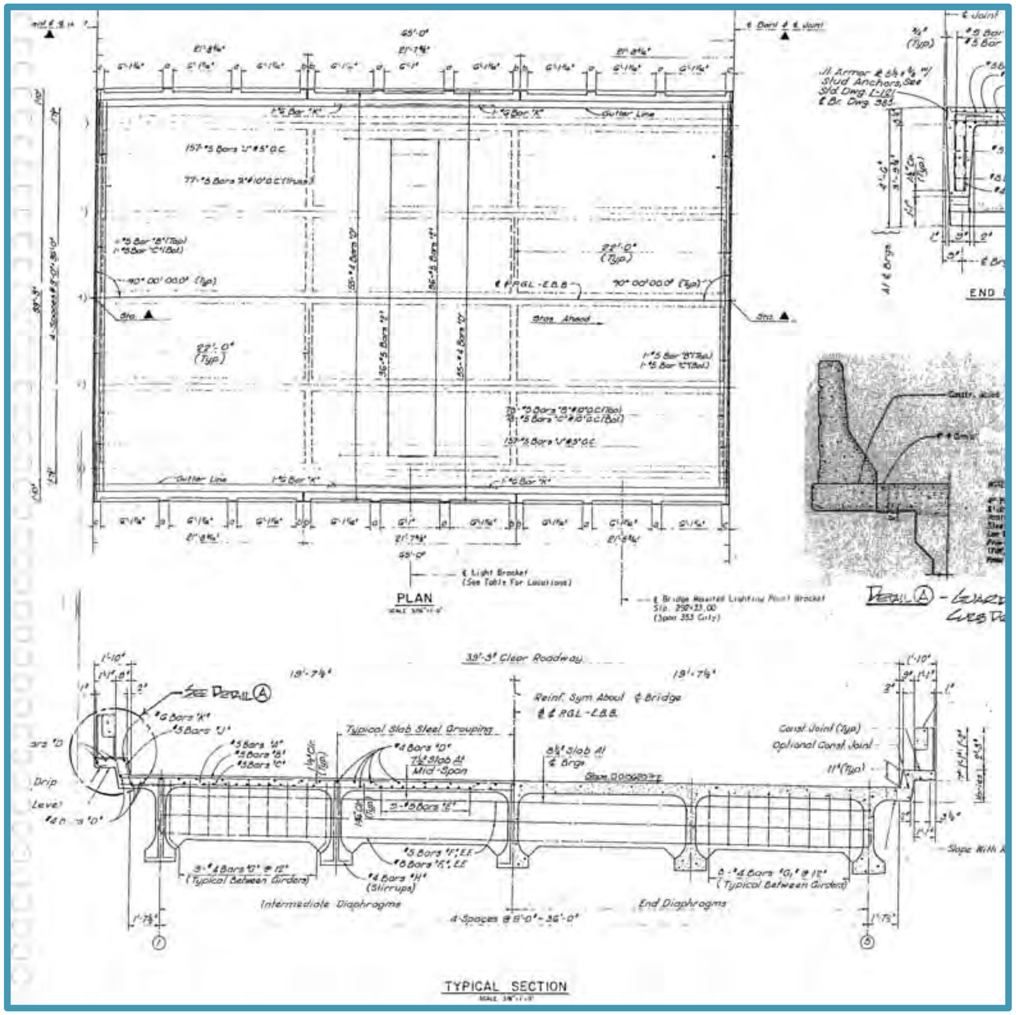
Retrofit Strategies and Approaches

2. Approach
    a. Reduction of Buoyancy Loads
    b. Reduction of Wave Loads
    c. Strengthening Connection of Superstructure to Substructure
    d. Strengthening the Structural Capacity of the Substructure
    e. Strengthening the Geotechnical Capacity of the Substructure
    f. Accepting loss of Superstructure to Protect Substructure



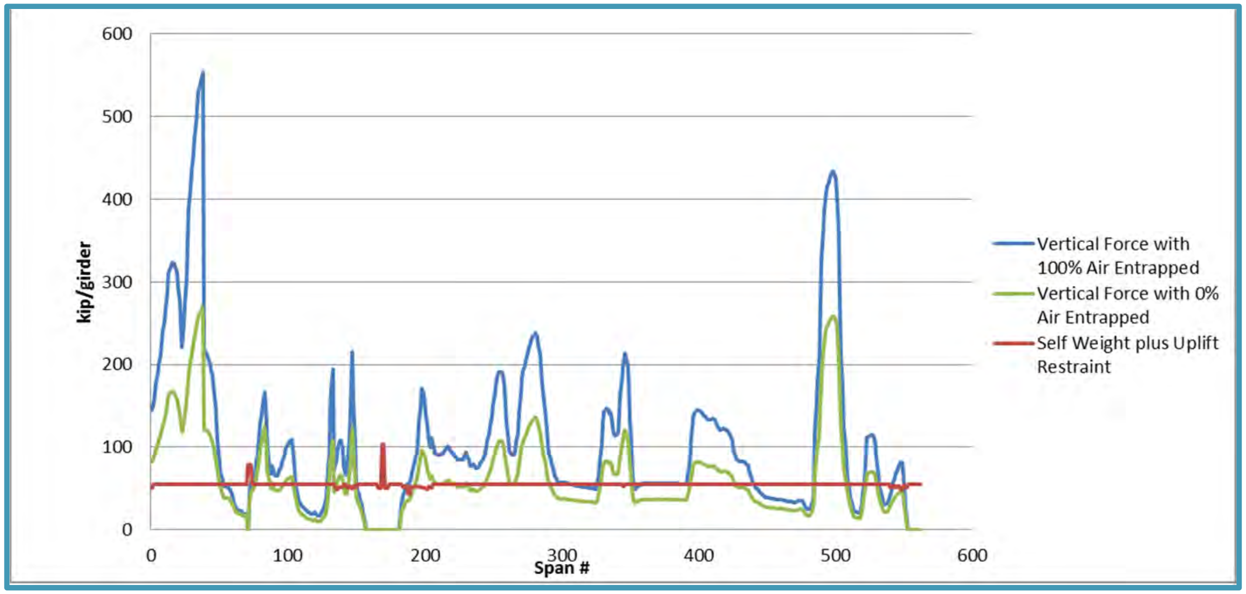
a. Reduction of Buoyancy Loads

- Trapped air Creates a Buoyancy Force that may Increase the Uplift Force during a Surge or Wave Event
- May be Reduced by providing Vents through the Deck or Diaphragms, or Replacing solid Diaphragms with Steel Frame Diaphragms



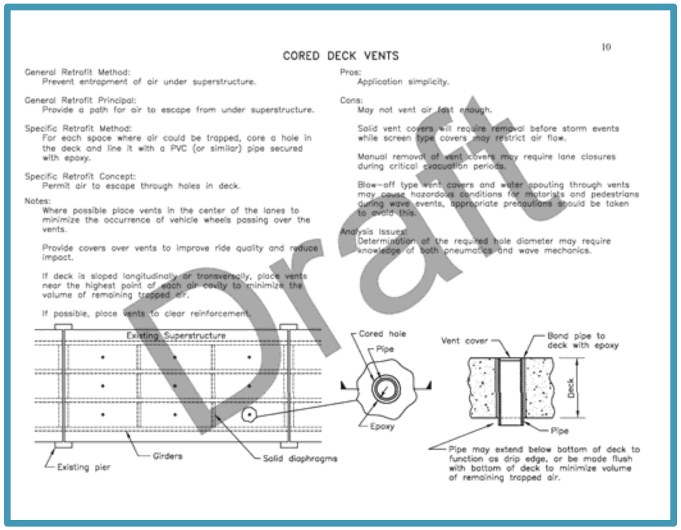
a. Reduction of Buoyancy Loads

- Air Entrapped within the Superstructure is a Significant contributor of Vertical Loads
- Auburn Report Discussed Potential Benefits of venting the Superstructure
- HMM Provided revised loads with Zero Entrapped Air (2017 SLR and 100yr Storm)



a. Reduction of Buoyancy Loads

- FHWA prepared Preliminary Calculations:
  - 65 ft. Span with Full Depth Diaphragms (similar to Bayway)
  - Evacuate 4 ft. Deep Air Cavity in 3 Seconds (Vent Storm Surge – a Slower Process)
  - 80-4” Diameter Holes per Span are Required
  - To Evacuate the Cavity in 1 Second (Reduce Buoyancy from Waves – a Quicker Process)
  - 240-4” Diameter Holes per Span Required
- Vents may not be Practical for Significantly Reducing Buoyancy Forces during Short Duration Events



10

CORED DECK VENTS

General Retrofit Method:
Prevent entrapment of air under superstructure.

General Retrofit Principal:
Provide a path for air to escape from under superstructure.

Specific Retrofit Method:
For each space where air could be trapped, core a hole in the deck and line it with a PVC (or similar) pipe secured with epoxy.

Specific Retrofit Concept:
Permit air to escape through holes in deck.

Notes:
Where possible place vents in the center of the lanes to minimize the occurrence of vehicle wheels passing over the vents.

Provide covers over vents to improve ride quality and reduce impact.

If deck is sloped longitudinally or transversally, place vents near the highest point of each air cavity to minimize the volume of remaining trapped air.

If possible, place vents to clear reinforcement.

Pros:
Application simplicity.

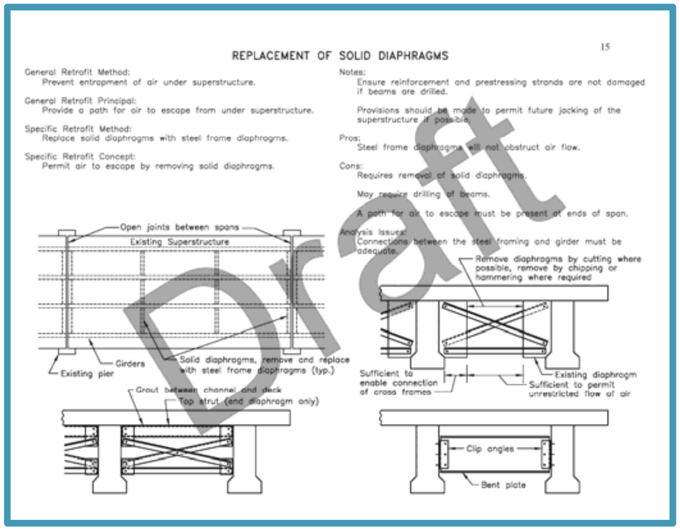
Cons:
May not vent air fast enough.

Solid vent covers will require removal before storm events while screen type covers may restrict air flow.

Manual removal of vent covers may require lane closures during critical evacuation periods.

Blow-off type vent covers and water spouting through vents may cause hazardous conditions for motorists and pedestrians during wave events, appropriate precautions should be taken to avoid this.

Analysis Issues:
Determination of the required hole diameter may require knowledge of both pneumatics and wave mechanics.



15

REPLACEMENT OF SOLID DIAPHRAGMS

General Retrofit Method:
Prevent entrapment of air under superstructure.

General Retrofit Principal:
Provide a path for air to escape from under superstructure.

Specific Retrofit Method:
Replace solid diaphragms with steel frame diaphragms.

Specific Retrofit Concept:
Permit air to escape by removing solid diaphragms.

Notes:
Ensure reinforcement and prestressing strands are not damaged if beams are drilled.

Provisions should be made to permit future jacking of the superstructure if possible.

Pros:
Steel frame diaphragms will not obstruct air flow.

Cons:
Requires removal of solid diaphragms.

May require drilling of beams.

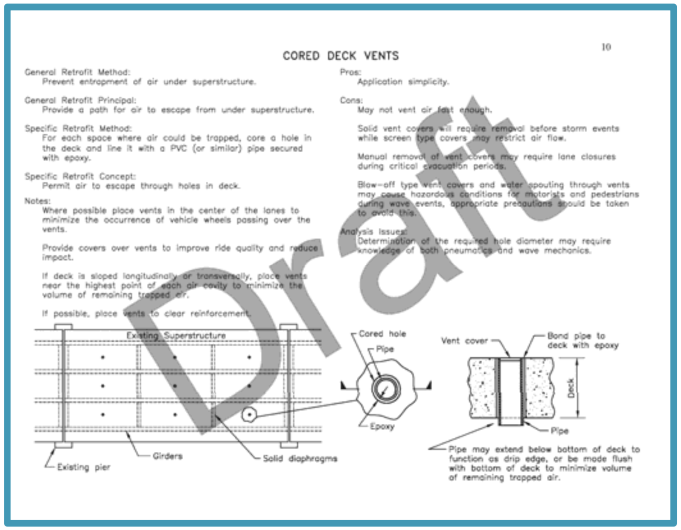
A path for air to escape must be present at ends of span.

Analysis Issues:
Connections between the steel framing and girder must be adequate.

FHWA Draft Details

a. Reduction of Buoyancy Loads

- Deck Drilling and Coring present the Possibility of Adverse effects on the Structure by Severing Reinforcing Bars
- Replacement of Solid Diaphragms with Steel Diaphragms is a very Intrusive approach
- Requires saw-cutting of the Concrete Diaphragms and Anchoring of Steel Frames in Their Place



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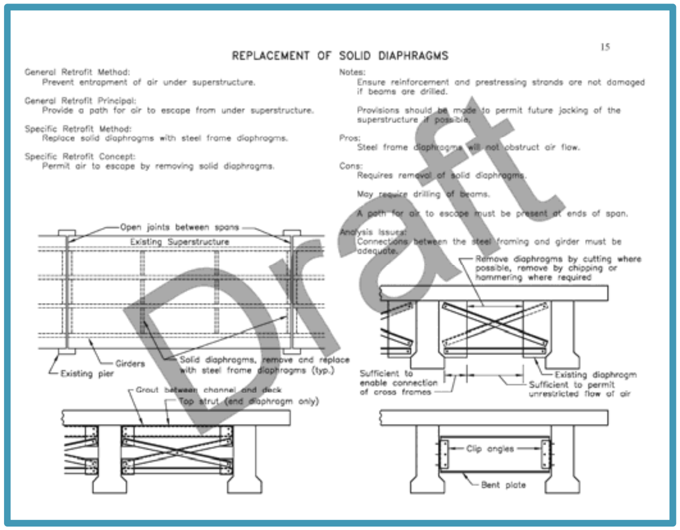

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Pros:
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Cons:
Requires removal of solid diaphragms.

May require drilling of beams.

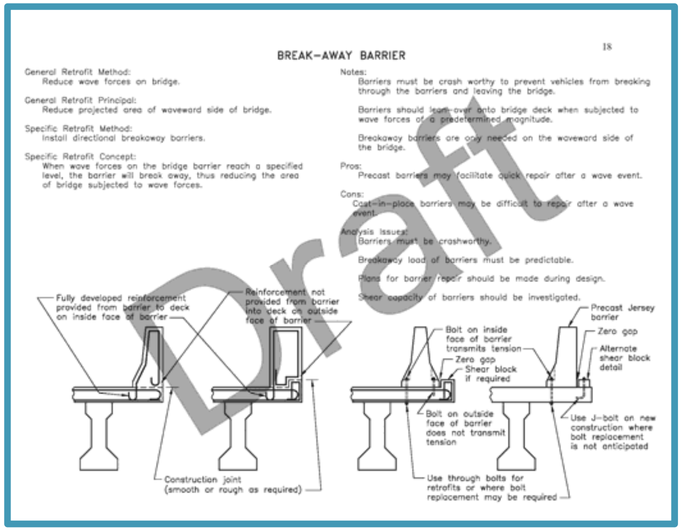
A path for air to escape must be present at ends of span.

Analysis Issues:
Connections between the steel framing and girder must be adequate.

FHWA Draft Details

b. Reduction of Wave Loads

- Waves Hitting the Deck and Girders Leads to Significant Forces.
- In some Cases it may be Possible to Reduce these Forces
    - Changing the Cross-Section that the Waves Strike
    - Raising the Cross-Section
- This Approach is Likely to be More Costly than Other Approaches



18

BREAK–AWAY BARRIER

General Retrofit Method:
Reduce wave forces on bridge.

General Retrofit Principal:
Reduce projected area of waveward side of bridge.

Specific Retrofit Method:
Install directional breakaway barriers.

Specific Retrofit Concept:
When wave forces on the bridge barrier reach a specified level, the barrier will break away, thus reducing the area of bridge subjected to wave forces.

Notes:
Barriers must be crash worthy to prevent vehicles from breaking through the barriers and leaving the bridge.

Barriers should lean-over onto bridge deck when subjected to wave forces of a predetermined magnitude.

Breakaway barriers are only needed on the waveward side of the bridge.

Pros:
Precast barriers may facilitate quick repair after a wave event.

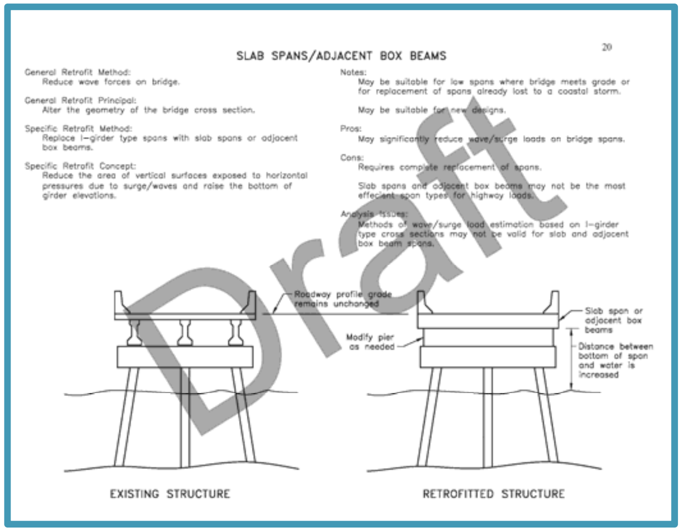
Cons:
Cast-in-place barriers may be difficult to repair after a wave event.

Analysis Issues:
Barriers must be crashworthy.

Breakaway load of barriers must be predictable.

Plans for barrier repair should be made during design.

Shear capacity of barriers should be investigated.



20

SLAB SPANS/ADJACENT BOX BEAMS

General Retrofit Method:
Reduce wave forces on bridge.

General Retrofit Principal:
Alter the geometry of the bridge cross section.

Specific Retrofit Method:
Replace I–girder type spans with slab spans or adjacent box beams.

Specific Retrofit Concept:
Reduce the area of vertical surfaces exposed to horizontal pressures due to surge/waves and raise the bottom of girder elevations.

Notes:
May be suitable for low spans where bridge meets grade or for replacement of spans already lost to a coastal storm.

May be suitable for new designs.

Pros:
May significantly reduce wave/surge loads on bridge spans.

Cons:
Requires complete replacement of spans.

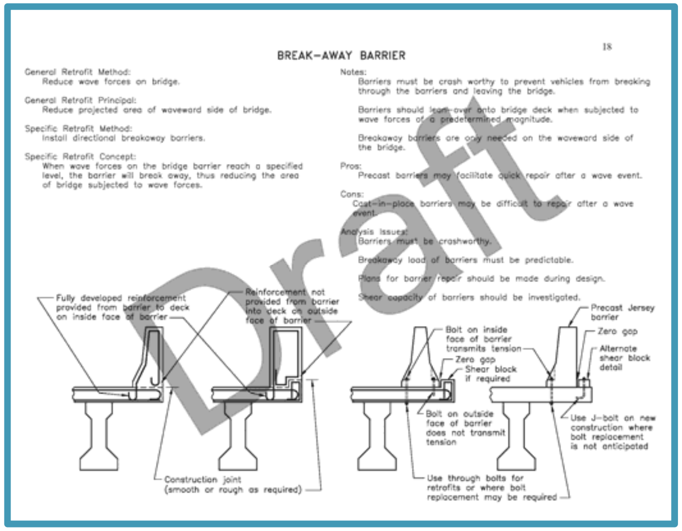
Slab spans and adjacent box beams may not be the most effecient span types for highway loads.

Analysis Issues:
Methods of wave/surge load estimation based on I–girder type cross sections may not be valid for slab and adjacent box beam spans.

FHWA Draft Details

b. Reduction of Wave Loads

- Barriers would Need to be Repaired/Replaced before Bridge is Opened to Traffic
- Span Replacement could be Considered at Selected locations
- Span Replacement will Require Bridge closures that could be Done at Night



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BREAK–AWAY BARRIER

General Retrofit Method:
Reduce wave forces on bridge.

General Retrofit Principal:
Reduce projected area of waveward side of bridge.

Specific Retrofit Method:
Install directional breakaway barriers.

Specific Retrofit Concept:
When wave forces on the bridge barrier reach a specified level, the barrier will break away, thus reducing the area of bridge subjected to wave forces.

Notes:
Barriers must be crash worthy to prevent vehicles from breaking through the barriers and leaving the bridge.

Barriers should lean-over onto bridge deck when subjected to wave forces of a predetermined magnitude.

Breakaway barriers are only needed on the waveward side of the bridge.

Pros:
Precast barriers may facilitate quick repair after a wave event.

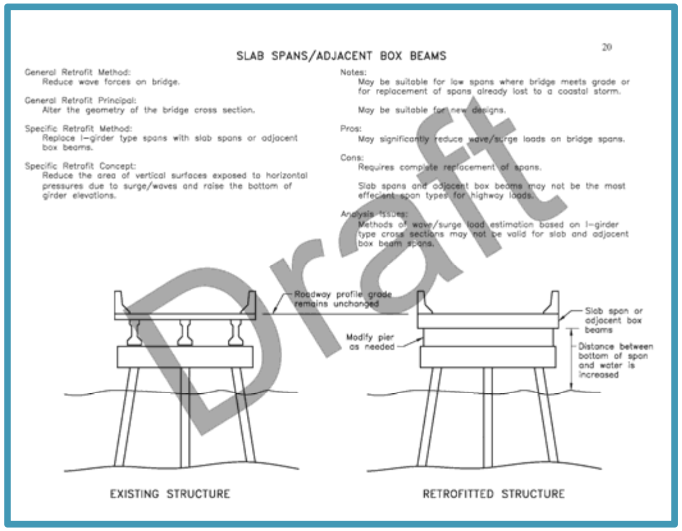
Cons:
Cast-in-place barriers may be difficult to repair after a wave event.

Analysis Issues:
Barriers must be crashworthy.

Breakaway load of barriers must be predictable.

Plans for barrier repair should be made during design.

Shear capacity of barriers should be investigated.



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SLAB SPANS/ADJACENT BOX BEAMS

General Retrofit Method:
Reduce wave forces on bridge.

General Retrofit Principal:
Alter the geometry of the bridge cross section.

Specific Retrofit Method:
Replace I–girder type spans with slab spans or adjacent box beams.

Specific Retrofit Concept:
Reduce the area of vertical surfaces exposed to horizontal pressures due to surge/waves and raise the bottom of girder elevations.

Notes:
May be suitable for low spans where bridge meets grade or for replacement of spans already lost to a coastal storm.

May be suitable for new designs.

Pros:
May significantly reduce wave/surge loads on bridge spans.

Cons:
Requires complete replacement of spans.

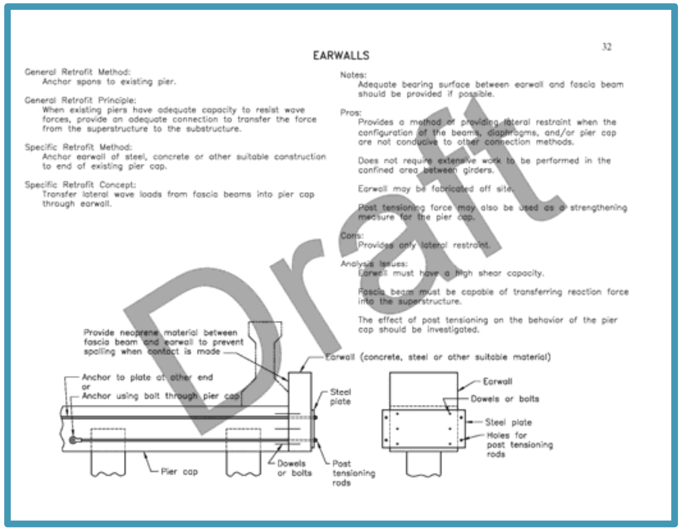
Slab spans and adjacent box beams may not be the most effecient span types for highway loads.

Analysis Issues:
Methods of wave/surge load estimation based on I–girder type cross sections may not be valid for slab and adjacent box beam spans.

FHWA Draft Details

c. Strengthening the Connection between the Super and Substructure

- Tying the Superstructure to the Substructure may Prevent Shifting of Spans due to Uplift and Lateral Loads
- Any Connection Retrofit Should Account for Normal Displacement:
  - Thermal expansion and Contraction
  - Live Load Rotation
  - Vertical Deformation of Elastomeric Bearings
- Normal Maintenance Needs of the Bridge Should Also be Provided:
  - Jacking Access for Bearing Replacement



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EARWALLS

General Retrofit Method:
Anchor spans to existing pier.

General Retrofit Principle:
When existing piers have adequate capacity to resist wave forces, provide an adequate connection to transfer the force from the superstructure to the substructure.

Specific Retrofit Method:
Anchor earwall of steel, concrete or other suitable construction to end of existing pier cap.

Specific Retrofit Concept:
Transfer lateral wave loads from fascia beams into pier cap through earwall.

Notes:
Adequate bearing surface between earwall and fascia beam should be provided if possible.

Pros:
Provides a method of providing lateral restraint when the configuration of the beams, diaphragms, and/or pier cap are not conducive to other connection methods.

Does not require extensive work to be performed in the confined area between girders.

Earwall may be fabricated off site.

Post tensioning force may also be used as a strengthening measure for the pier cap.

Cons:
Provides only lateral restraint.

Analysis Issues:
Earwall must have a high shear capacity.

Fascia beam must be capable of transferring reaction force into the superstructure.

The effect of post tensioning on the behavior of the pier cap should be investigated.

Provide neoprene material between fascia beam and earwall to prevent spalling when contact is made

Earwall (concrete, steel or other suitable material)

Anchor to plate at other end
or
Anchor using bolt through pier cap

Steel plate

Pier cap

Dowels or bolts

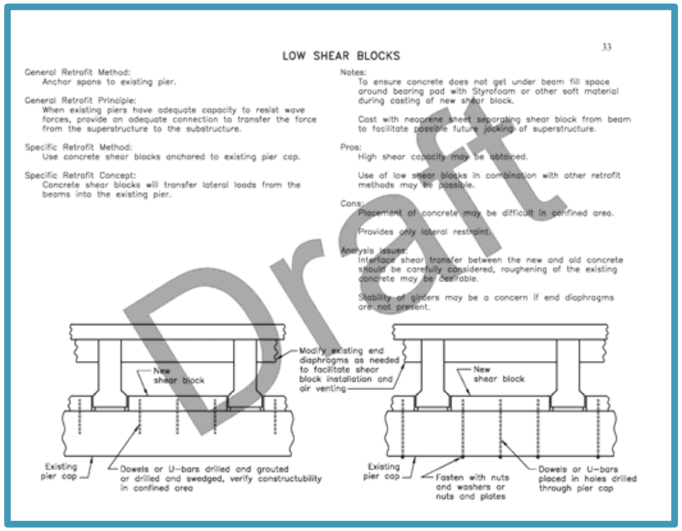
Post tensioning rods

Earwall

Dowels or bolts

Steel plate

Holes for post tensioning rods



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LOW SHEAR BLOCKS

General Retrofit Method:
Anchor spans to existing pier.

General Retrofit Principle:
When existing piers have adequate capacity to resist wave forces, provide an adequate connection to transfer the force from the superstructure to the substructure.

Specific Retrofit Method:
Use concrete shear blocks anchored to existing pier cap.

Specific Retrofit Concept:
Concrete shear blocks will transfer lateral loads from the beams into the existing pier.

Notes:
To ensure concrete does not get under beam fill space around bearing pad with Styrofoam or other soft material during casting of new shear block.

Cast with neoprene sheet separating shear block from beam to facilitate possible future jacking of superstructure.

Pros:
High shear capacity may be obtained.

Use of low shear blocks in combination with other retrofit methods may be possible.

Cons:
Placement of concrete may be difficult in confined area.

Provides only lateral restraint.

Analysis Issues:
Interface shear transfer between the new and old concrete should be carefully considered, roughening of the existing concrete may be desirable.

Stability of girders may be a concern if end diaphragms are not present.

New shear block

Modify existing end diaphragms as needed to facilitate shear block installation and air venting

New shear block

Existing pier cap

Dowels or U-bars drilled and grouted or drilled and swedged, verify constructubility in confined area

Existing pier cap

Fasten with nuts and washers or nuts and plates

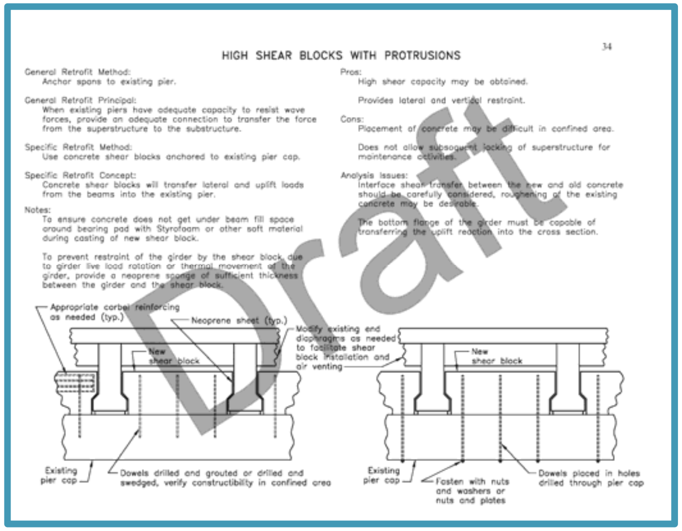
Dowels or U-bars placed in holes drilled through pier cap

FHWA Draft Details

c. Strengthening the Connection between the Super and Substructure

- Loads Need to be followed Through the Structural Load Path to the Ground
- Therefore need to Check Additional Failure Modes:
  - <u>Negative Bending in the Superstructure at Midspan due to Uplift Forces</u>
  - Shear at the Ends of the Girder and Cap
  - Reduction in Bending Capacity of the Piles due to Decreased Compression, or even tensile Forces
  - Increased Lateral Loads on the Substructure
  - Shear and Bending on the Piles and Concrete Plug

Focus of our Analysis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HIGH SHEAR BLOCKS WITH PROTRUSIONS

General Retrofit Method:
Anchor spans to existing pier.

General Retrofit Principal:
When existing piers have adequate capacity to resist wave forces, provide an adequate connection to transfer the force from the superstructure to the substructure.

Specific Retrofit Method:
Use concrete shear blocks anchored to existing pier cap.

Specific Retrofit Concept:
Concrete shear blocks will transfer lateral and uplift loads from the beams into the existing pier.

Notes:
To ensure concrete does not get under beam fill space around bearing pad with Styrofoam or other soft material during casting of new shear block.

To prevent restraint of the girder by the shear block due to girder live load rotation or thermal movement of the girder, provide a neoprene sponge of sufficient thickness between the girder and the shear block.

Pros:
High shear capacity may be obtained.

Provides lateral and vertical restraint.

Cons:
Placement of concrete may be difficult in confined area.

Does not allow subsequent jacking of superstructure for maintenance activities.

Analysis Issues:
Interface shear transfer between the new and old concrete should be carefully considered, roughening of the existing concrete may be desirable.

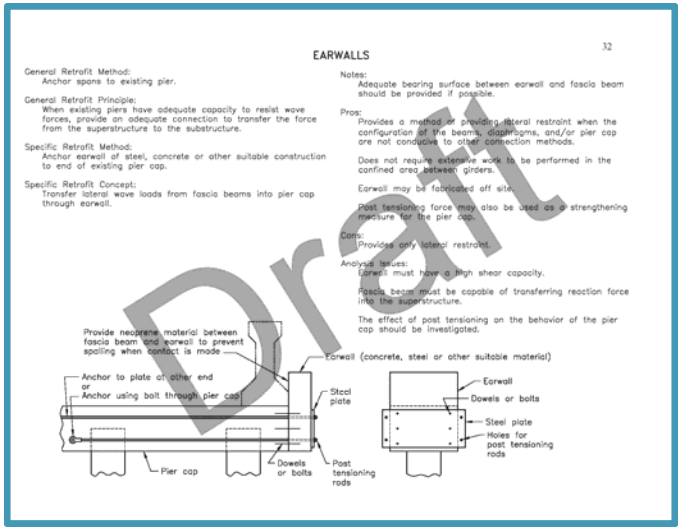
The bottom flange of the girder must be capable of transferring the uplift reaction into the cross section.

FHWA Draft Details

d. Strengthening the Connection between the Super and Substructure

- Horizontal Strengthening:
  - Earwalls give us the Most Flexibility to Accommodate different Substructure Geometry's
  - Eliminates Uncertainty in Distributing the Transverse Loads among Several Resistant Elements
  - Ease of Construction
- Vertical Restraint
  - Cable Restraints Give us the most Flexibility (ignore horizontal contribution for Conservatism)
  - Have been Used in Seismic Areas.

FHWA Draft Details



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EARWALLS

General Retrofit Method:
Anchor spans to existing pier.

General Retrofit Principle:
When existing piers have adequate capacity to resist wave forces, provide an adequate connection to transfer the force from the superstructure to the substructure.

Specific Retrofit Method:
Anchor earwall of steel, concrete or other suitable construction to end of existing pier cap.

Specific Retrofit Concept:
Transfer lateral wave loads from fascia beams into pier cap through earwall.

Notes:
Adequate bearing surface between earwall and fascia beam should be provided if possible.

Pros:
Provides a method of providing lateral restraint when the configuration of the beams, diaphragms, and/or pier cap are not conducive to other connection methods.

Does not require extensive work to be performed in the confined area between girders.

Earwall may be fabricated off site.

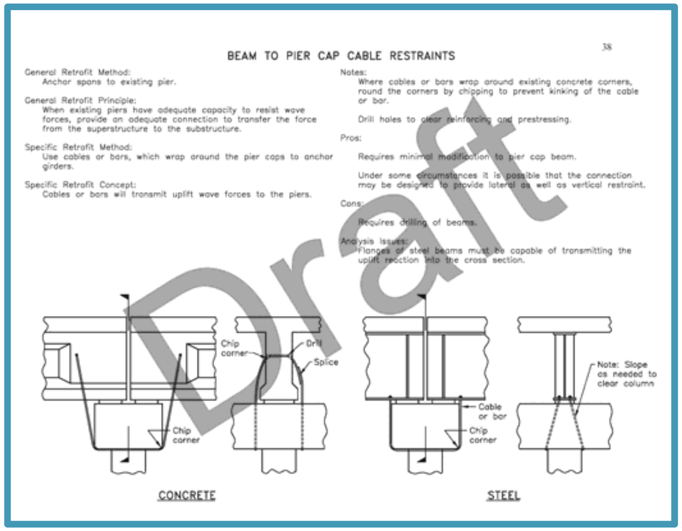
Post tensioning force may also be used as a strengthening measure for the pier cap.

Cons:
Provides only lateral restraint.

Analysis Issues:
Earwall must have a high shear capacity.

Fascia beam must be capable of transferring reaction force into the superstructure.

The effect of post tensioning on the behavior of the pier cap should be investigated.



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BEAM TO PIER CAP CABLE RESTRAINTS

General Retrofit Method:
Anchor spans to existing pier.

General Retrofit Principle:
When existing piers have adequate capacity to resist wave forces, provide an adequate connection to transfer the force from the superstructure to the substructure.

Specific Retrofit Method:
Use cables or bars, which wrap around the pier caps to anchor girders.

Specific Retrofit Concept:
Cables or bars will transmit uplift wave forces to the piers.

Notes:
Where cables or bars wrap around existing concrete corners, round the corners by chipping to prevent kinking of the cable or bar.

Drill holes to clear reinforcing and prestressing.

Pros:
Requires minimal modification to pier cap beam.

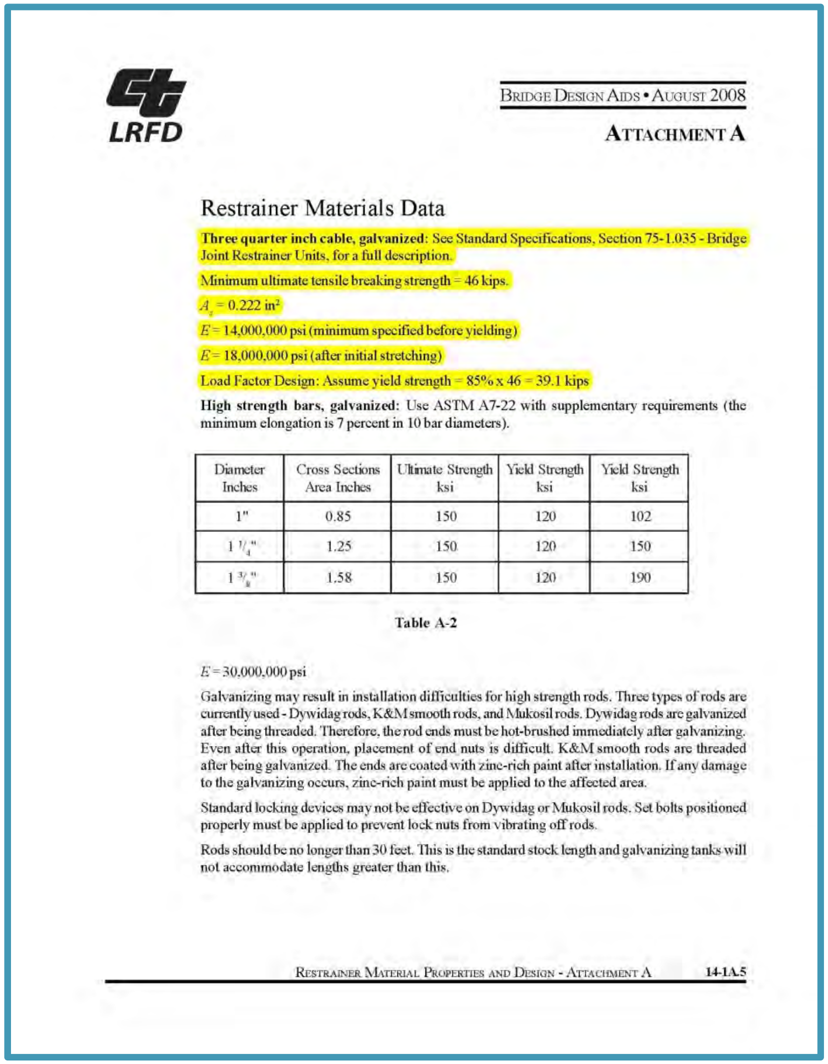
Under some circumstances it is possible that the connection may be designed to provide lateral as well as vertical restraint.

Cons:
Requires drilling of beams.

Analysis Issues:
Flanges of steel beams must be capable of transmitting the uplift reaction into the cross section.

Cable Restrainers

- Approached used in States where Bridges are retrofitted to resist seismic loads.
- California, South Carolina, Oregon, Washington use similar Approach



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BRIDGE DESIGN AIDS • AUGUST 2008

ATTACHMENT A

Restrainer Materials Data

Three quarter inch cable, galvanized: See Standard Specifications, Section 75-1.035 - Bridge Joint Restrainer Units, for a full description.

Minimum ultimate tensile breaking strength = 46 kips.

$A_s = 0.222 \text{ in}^2$

E = 14,000,000 psi (minimum specified before yielding)

E = 18,000,000 psi (after initial stretching)

Load Factor Design: Assume yield strength = 85% x 46 = 39.1 kips

High strength bars, galvanized: Use ASTM A7-22 with supplementary requirements (the minimum elongation is 7 percent in 10 bar diameters).

Diameter Inches	Cross Sections Area Inches	Ultimate Strength ksi	Yield Strength ksi	Yield Strength ksi
1"	0.85	150	120	102
1 1/4"	1.25	150	120	150
1 3/8"	1.58	150	120	190

Table A-2

E = 30,000,000 psi

Galvanizing may result in installation difficulties for high strength rods. Three types of rods are currently used - Dywidag rods, K&M smooth rods, and Mukosil rods. Dywidag rods are galvanized after being threaded. Therefore, the rod ends must be hot-brushed immediately after galvanizing. Even after this operation, placement of end nuts is difficult. K&M smooth rods are threaded after being galvanized. The ends are coated with zinc-rich paint after installation. If any damage to the galvanizing occurs, zinc-rich paint must be applied to the affected area.

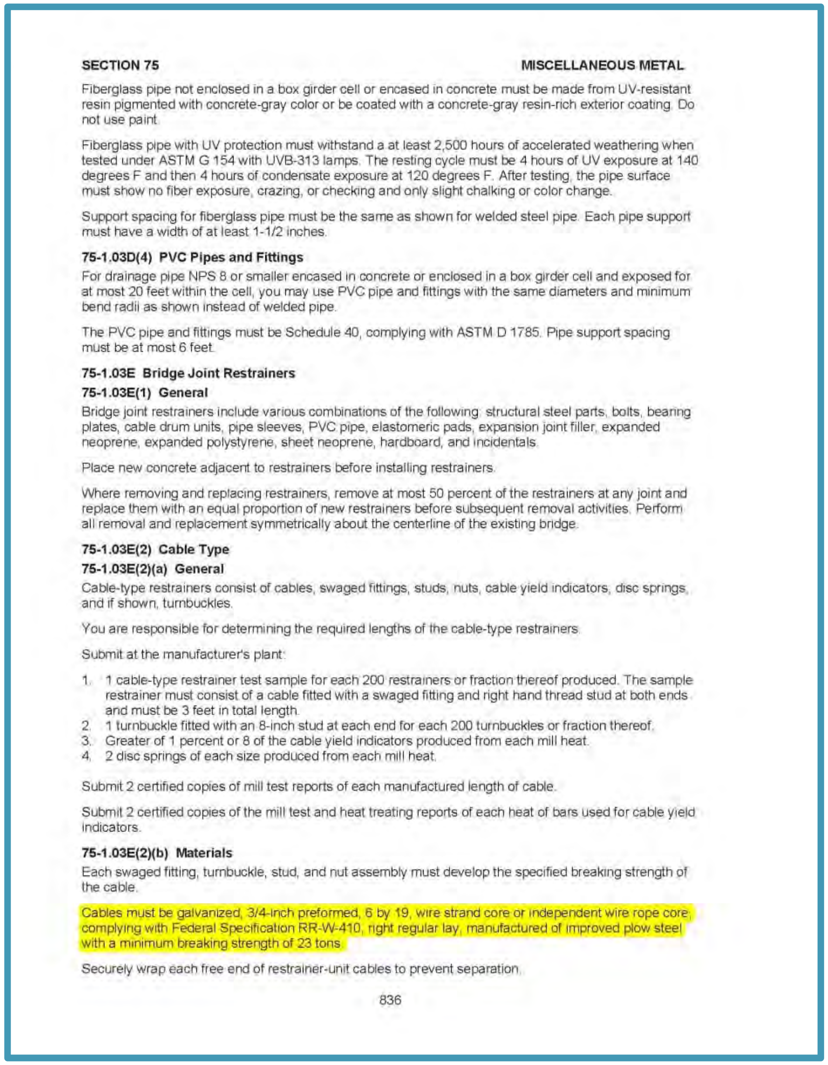
Standard locking devices may not be effective on Dywidag or Mukosil rods. Set bolts positioned properly must be applied to prevent lock nuts from vibrating off rods.

Rods should be no longer than 30 feet. This is the standard stock length and galvanizing tanks will not accommodate lengths greater than this.

RESTRAINER MATERIAL PROPERTIES AND DESIGN - ATTACHMENT A 14-1A.5

Cable Restrainers

- Cables are Galvanized
- ¾” Preformed, 6 by 19 Wire Strand Core



Fiberglass pipe not enclosed in a box girder cell or encased in concrete must be made from UV-resistant resin pigmented with concrete-gray color or be coated with a concrete-gray resin-rich exterior coating. Do not use paint.

Fiberglass pipe with UV protection must withstand a at least 2,500 hours of accelerated weathering when tested under ASTM G 154 with UVB-313 lamps. The resting cycle must be 4 hours of UV exposure at 140 degrees F and then 4 hours of condensate exposure at 120 degrees F. After testing, the pipe surface must show no fiber exposure, crazing, or checking and only slight chalking or color change.

Support spacing for fiberglass pipe must be the same as shown for welded steel pipe. Each pipe support must have a width of at least 1-1/2 inches.

75-1.03D(4) PVC Pipes and Fittings

For drainage pipe NPS 8 or smaller encased in concrete or enclosed in a box girder cell and exposed for at most 20 feet within the cell, you may use PVC pipe and fittings with the same diameters and minimum bend radii as shown instead of welded pipe.

The PVC pipe and fittings must be Schedule 40, complying with ASTM D 1785. Pipe support spacing must be at most 6 feet.

75-1.03E Bridge Joint Restrainers

75-1.03E(1) General

Bridge joint restrainers include various combinations of the following: structural steel parts, bolts, bearing plates, cable drum units, pipe sleeves, PVC pipe, elastomeric pads, expansion joint filler, expanded neoprene, expanded polystyrene, sheet neoprene, hardboard, and incidentals.

Place new concrete adjacent to restrainers before installing restrainers.

Where removing and replacing restrainers, remove at most 50 percent of the restrainers at any joint and replace them with an equal proportion of new restrainers before subsequent removal activities. Perform all removal and replacement symmetrically about the centerline of the existing bridge.

75-1.03E(2) Cable Type

75-1.03E(2)(a) General

Cable-type restrainers consist of cables, swaged fittings, studs, nuts, cable yield indicators, disc springs, and if shown, turnbuckles.

You are responsible for determining the required lengths of the cable-type restrainers.

Submit at the manufacturer's plant:

1. 1 cable-type restrainer test sample for each 200 restrainers or fraction thereof produced. The sample restrainer must consist of a cable fitted with a swaged fitting and right hand thread stud at both ends and must be 3 feet in total length.
2. 1 turnbuckle fitted with an 8-inch stud at each end for each 200 turnbuckles or fraction thereof.
3. Greater of 1 percent or 8 of the cable yield indicators produced from each mill heat.
4. 2 disc springs of each size produced from each mill heat.

Submit 2 certified copies of mill test reports of each manufactured length of cable.

Submit 2 certified copies of the mill test and heat treating reports of each heat of bars used for cable yield indicators.

75-1.03E(2)(b) Materials

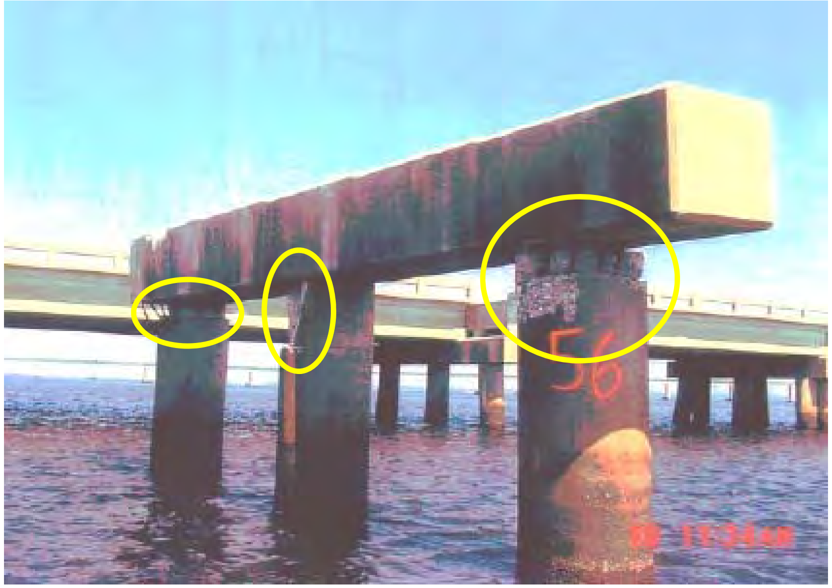
Each swaged fitting, turnbuckle, stud, and nut assembly must develop the specified breaking strength of the cable.

Cables must be galvanized, 3/4-inch preformed, 6 by 19, wire strand core or independent wire rope core, complying with Federal Specification RR-W-410, right regular lay, manufactured of improved plow steel with a minimum breaking strength of 23 tons.

Securely wrap each free end of restrainer-unit cables to prevent separation.

d. Strengthening Substructure

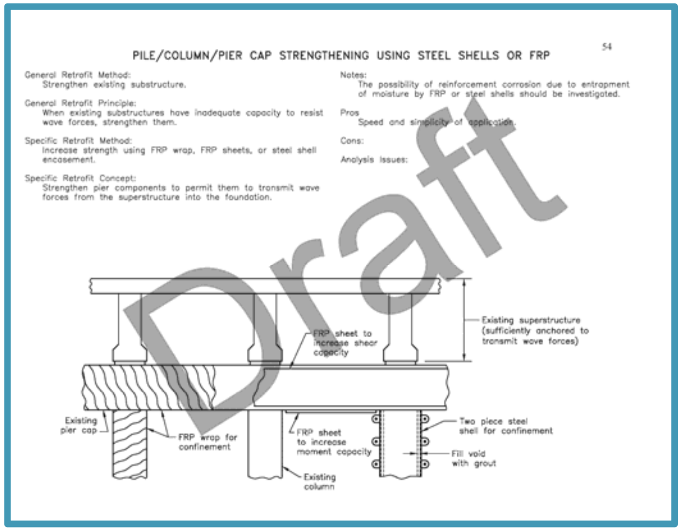
- The Lateral Loads and Vertical Uplift were Likely not Included in the Original design
- Aspects of the Substructure that Have not performed well in past Events include:
  - The Connection between the Pile and the Pile Cap
  - Bending Strength of Pile Immediately below the Pile Cap



Span Dislodged Before Bent Collapsed

d. Strengthening Substructure

- Substructure can be Strengthened using FRP, Steel Shells.
- Pier Cap to Pile Cable Restraints were not Considered as they would Require Drilling into the Piles



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PILE/COLUMN/PIER CAP STRENGTHENING USING STEEL SHELLS OR FRP

General Retrofit Method:
Strengthen existing substructure.

General Retrofit Principle:
When existing substructures have inadequate capacity to resist wave forces, strengthen them.

Specific Retrofit Method:
Increase strength using FRP wrap, FRP sheets, or steel shell encasement.

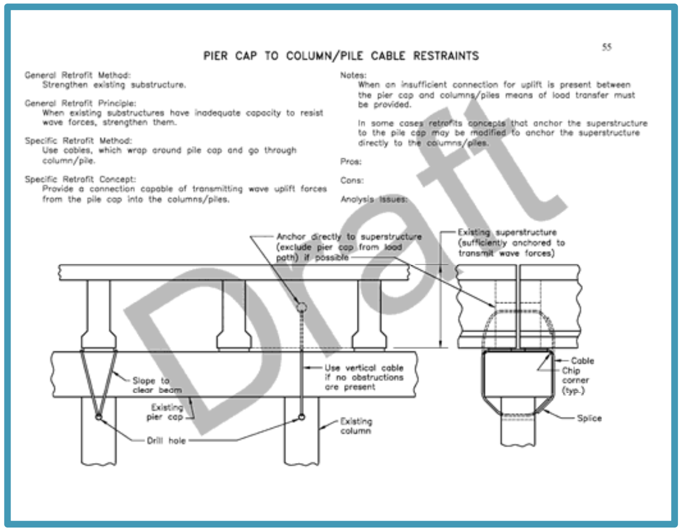
Specific Retrofit Concept:
Strengthen pier components to permit them to transmit wave forces from the superstructure into the foundation.

Notes:
The possibility of reinforcement corrosion due to entrapment of moisture by FRP or steel shells should be investigated.

Pros
Speed and simplicity of application.

Cons:

Analysis Issues:



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PIER CAP TO COLUMN/PILE CABLE RESTRAINTS

General Retrofit Method:
Strengthen existing substructure.

General Retrofit Principle:
When existing substructures have inadequate capacity to resist wave forces, strengthen them.

Specific Retrofit Method:
Use cables, which wrap around pile cap and go through column/pile.

Specific Retrofit Concept:
Provide a connection capable of transmitting wave uplift forces from the pile cap into the columns/piles.

Notes:
When an insufficient connection for uplift is present between the pier cap and columns/piles means of load transfer must be provided.

In some cases retrofits concepts that anchor the superstructure to the pile cap may be modified to anchor the superstructure directly to the columns/piles.

Pros:

Cons:

Analysis Issues:

FHWA Draft Details

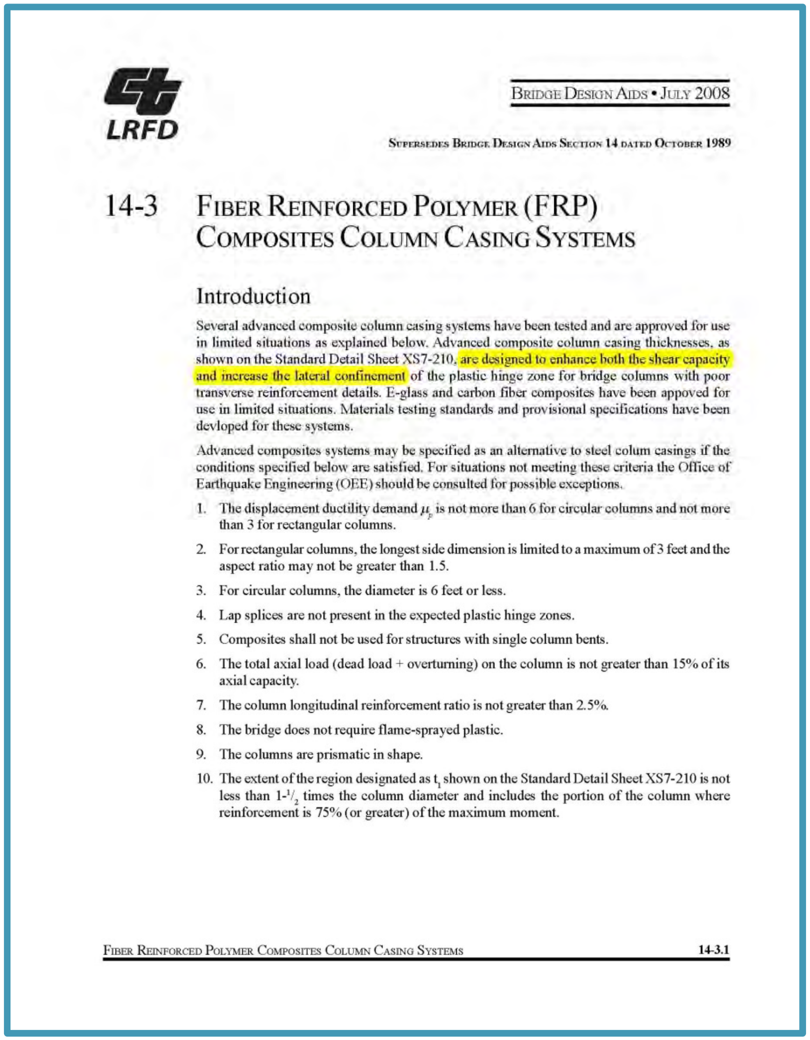
What is FRP Reinforcement

- FRP systems provide a very practical tool for strengthening and retrofit of concrete structures, and are appropriate for:
  - Flexural strengthening,
  - Shear strengthening, and
  - Column confinement and ductility improvement.
- Because of the resistance to corrosion, FRP composites can be utilized on interior and exterior structural members in all almost all types of environments

CFRP Reinforcement on Parking Garage Beams

FRP Column Casings

- Used by CALTRANS to retrofit Existing Bridges
- Designed to Enhance both the Shear Capacity and Increase the Lateral Confinement



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Bridge Design Aids • July 2008

Supersedes Bridge Design Aids Section 14 dated October 1989

14-3 Fiber Reinforced Polymer (FRP) Composites Column Casing Systems

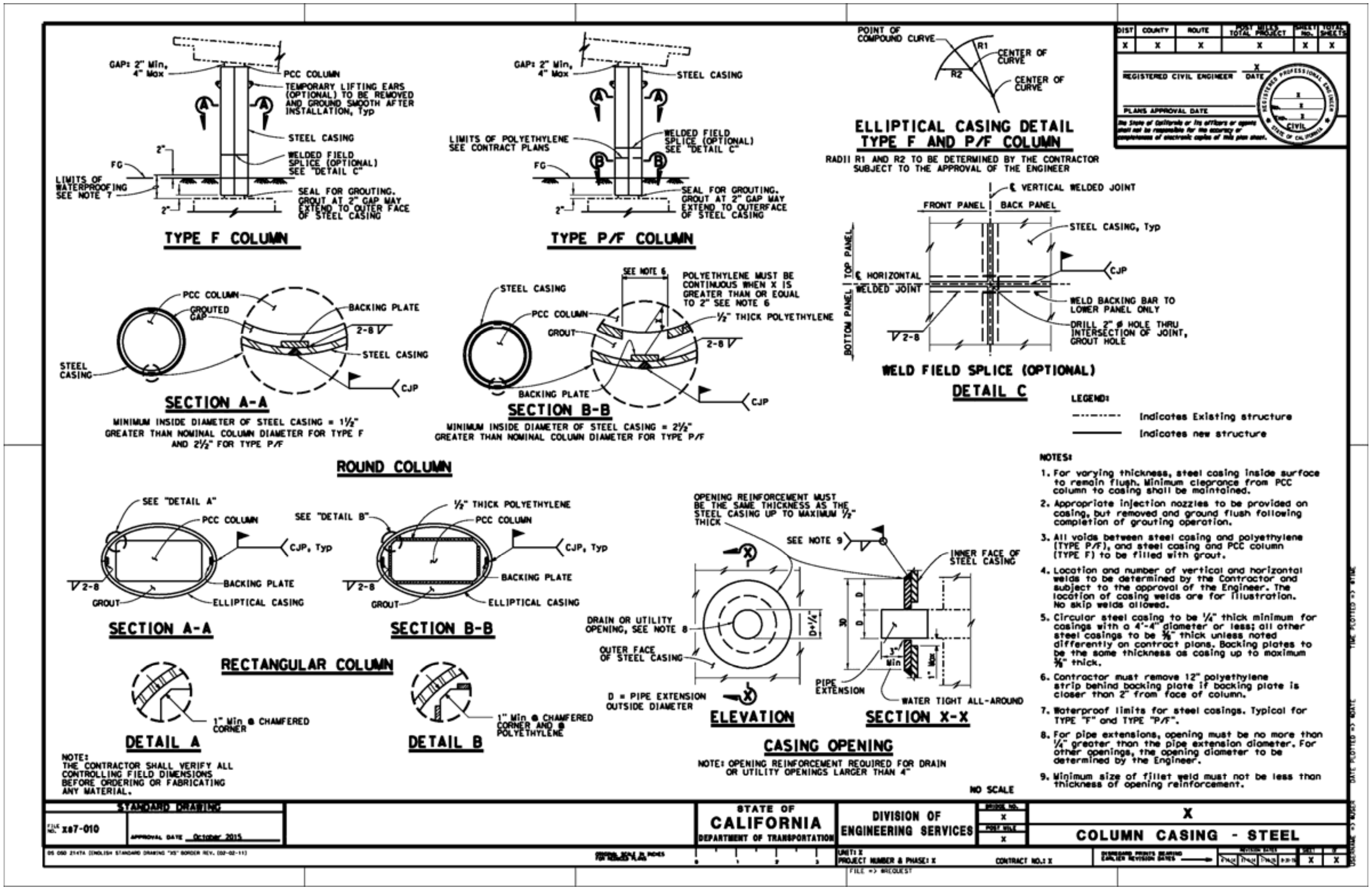
Introduction

Several advanced composite column casing systems have been tested and are approved for use in limited situations as explained below. Advanced composite column casing thicknesses, as shown on the Standard Detail Sheet XS7-210, are designed to enhance both the shear capacity and increase the lateral confinement of the plastic hinge zone for bridge columns with poor transverse reinforcement details. E-glass and carbon fiber composites have been appoved for use in limited situations. Materials testing standards and provisional specifications have been devloped for these systems.

Advanced composites systems may be specified as an alternative to steel colum casings if the conditions specified below are satisfied. For situations not meeting these criteria the Office of Earthquake Engineering (OEE) should be consulted for possible exceptions.

1. The displacement ductility demand μ_p is not more than 6 for circular columns and not more than 3 for rectangular columns.
2. For rectangular columns, the longest side dimension is limited to a maximum of 3 feet and the aspect ratio may not be greater than 1.5.
3. For circular columns, the diameter is 6 feet or less.
4. Lap splices are not present in the expected plastic hinge zones.
5. Composites shall not be used for structures with single column bents.
6. The total axial load (dead load + overturning) on the column is not greater than 15% of its axial capacity.
7. The column longitudinal reinforcement ratio is not greater than 2.5%.
8. The bridge does not require flame-sprayed plastic.
9. The columns are prismatic in shape.
10. The extent of the region designated as t_1 shown on the Standard Detail Sheet XS7-210 is not less than 1-$^{1}/_{2}$ times the column diameter and includes the portion of the column where reinforcement is 75% (or greater) of the maximum moment.

Fiber Reinforced Polymer Composites Column Casing Systems 14-3.1



MINIMUM INSIDE DIAMETER OF STEEL CASING = 1½" GREATER THAN NOMINAL COLUMN DIAMETER FOR TYPE F AND 2½" FOR TYPE P/F

MINIMUM INSIDE DIAMETER OF STEEL CASING = 2½" GREATER THAN NOMINAL COLUMN DIAMETER FOR TYPE P/F

RADII R1 AND R2 TO BE DETERMINED BY THE CONTRACTOR SUBJECT TO THE APPROVAL OF THE ENGINEER

OPENING REINFORCEMENT MUST BE THE SAME THICKNESS AS THE STEEL CASING UP TO MAXIMUM ½" THICK

D = PIPE EXTENSION OUTSIDE DIAMETER

NOTE: OPENING REINFORCEMENT REQUIRED FOR DRAIN OR UTILITY OPENINGS LARGER THAN 4"

NOTE:
THE CONTRACTOR SHALL VERIFY ALL CONTROLLING FIELD DIMENSIONS BEFORE ORDERING OR FABRICATING ANY MATERIAL.

The State of California or its officers or agents shall not be responsible for the accuracy or completeness of electronic copies of this plan sheet.

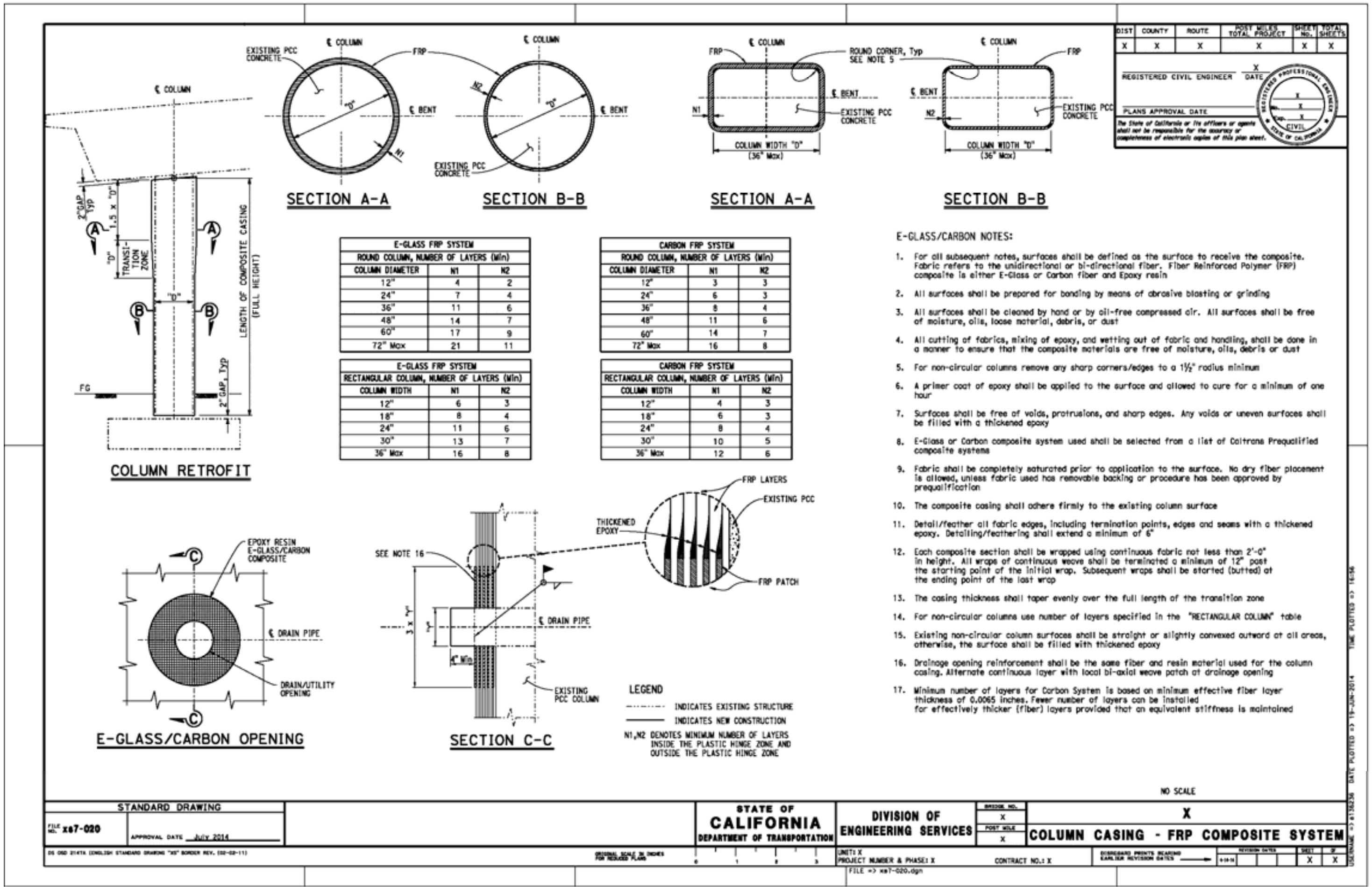
LEGEND:

-·-·-·-·	Indicates Existing structure
———	Indicates new structure

NOTES:

1. For varying thickness, steel casing inside surface to remain flush. Minimum clearance from PCC column to casing shall be maintained.
2. Appropriate injection nozzles to be provided on casing, but removed and ground flush following completion of grouting operation.
3. All voids between steel casing and polyethylene (TYPE P/F), and steel casing and PCC column (TYPE F) to be filled with grout.
4. Location and number of vertical and horizontal welds to be determined by the Contractor and subject to the approval of the Engineer. The location of casing welds are for illustration. No skip welds allowed.
5. Circular steel casing to be ¼" thick minimum for casings with a 4'-4" diameter or less; all other steel casings to be ⅜" thick unless noted differently on contract plans. Backing plates to be the same thickness as casing up to maximum ⅜" thick.
6. Contractor must remove 12" polyethylene strip behind backing plate if backing plate is closer than 2" from face of column.
7. Waterproof limits for steel casings. Typical for TYPE "F" and TYPE "P/F".
8. For pipe extensions, opening must be no more than ¼" greater than the pipe extension diameter. For other openings, the opening diameter to be determined by the Engineer.
9. Minimum size of fillet weld must not be less than thickness of opening reinforcement.

CALTRANS – Steel Columns Casing



COLUMN RETROFIT

SECTION A-A

SECTION B-B

SECTION A-A

SECTION B-B

E-GLASS/CARBON OPENING

SECTION C-C

E-GLASS FRP SYSTEM		
ROUND COLUMN, NUMBER OF LAYERS (Min)		
COLUMN DIAMETER	N1	N2
12"	4	2
24"	7	4
36"	11	6
48"	14	7
60"	17	9
72" Max	21	11

CARBON FRP SYSTEM		
ROUND COLUMN, NUMBER OF LAYERS (Min)		
COLUMN DIAMETER	N1	N2
12"	3	3
24"	6	3
36"	8	4
48"	11	6
60"	14	7
72" Max	16	8

E-GLASS FRP SYSTEM		
RECTANGULAR COLUMN, NUMBER OF LAYERS (Min)		
COLUMN WIDTH	N1	N2
12"	6	3
18"	8	4
24"	11	6
30"	13	7
36" Max	16	8

CARBON FRP SYSTEM		
RECTANGULAR COLUMN, NUMBER OF LAYERS (Min)		
COLUMN WIDTH	N1	N2
12"	4	3
18"	6	3
24"	8	4
30"	10	5
36" Max	12	6

LEGEND

- ·-·-·-· INDICATES EXISTING STRUCTURE
- ——— INDICATES NEW CONSTRUCTION

N1,N2 DENOTES MINIMUM NUMBER OF LAYERS INSIDE THE PLASTIC HINGE ZONE AND OUTSIDE THE PLASTIC HINGE ZONE

E-GLASS/CARBON NOTES:

1. For all subsequent notes, surfaces shall be defined as the surface to receive the composite. Fabric refers to the unidirectional or bi-directional fiber. Fiber Reinforced Polymer (FRP) composite is either E-Glass or Carbon fiber and Epoxy resin
2. All surfaces shall be prepared for bonding by means of abrasive blasting or grinding
3. All surfaces shall be cleaned by hand or by oil-free compressed air. All surfaces shall be free of moisture, oils, loose material, debris, or dust
4. All cutting of fabrics, mixing of epoxy, and wetting out of fabric and handling, shall be done in a manner to ensure that the composite materials are free of moisture, oils, debris or dust
5. For non-circular columns remove any sharp corners/edges to a 1½" radius minimum
6. A primer coat of epoxy shall be applied to the surface and allowed to cure for a minimum of one hour
7. Surfaces shall be free of voids, protrusions, and sharp edges. Any voids or uneven surfaces shall be filled with a thickened epoxy
8. E-Glass or Carbon composite system used shall be selected from a list of Caltrans Prequalified composite systems
9. Fabric shall be completely saturated prior to application to the surface. No dry fiber placement is allowed, unless fabric used has removable backing or procedure has been approved by prequalification
10. The composite casing shall adhere firmly to the existing column surface
11. Detail/feather all fabric edges, including termination points, edges and seams with a thickened epoxy. Detailing/feathering shall extend a minimum of 6"
12. Each composite section shall be wrapped using continuous fabric not less than 2'-0" in height. All wraps of continuous weave shall be terminated a minimum of 12" past the starting point of the initial wrap. Subsequent wraps shall be started (butted) at the ending point of the last wrap
13. The casing thickness shall taper evenly over the full length of the transition zone
14. For non-circular columns use number of layers specified in the "RECTANGULAR COLUMN" table
15. Existing non-circular column surfaces shall be straight or slightly convexed outward at all areas, otherwise, the surface shall be filled with thickened epoxy
16. Drainage opening reinforcement shall be the same fiber and resin material used for the column casing. Alternate continuous layer with local bi-axial weave patch at drainage opening
17. Minimum number of layers for Carbon System is based on minimum effective fiber layer thickness of 0.0065 inches. Fewer number of layers can be installed for effectively thicker (fiber) layers provided that an equivalent stiffness is maintained

NO SCALE

DIST	COUNTY	ROUTE	POST MILES TOTAL PROJECT	SHEET No.	TOTAL SHEETS
X	X	X	X	X	X

REGISTERED CIVIL ENGINEER X DATE

PLANS APPROVAL DATE

The State of California or its officers or agents shall not be responsible for the accuracy or completeness of electronic copies of this plan sheet.

STANDARD DRAWING
FILE NO. xs7-020
APPROVAL DATE July 2014

STATE OF CALIFORNIA
DEPARTMENT OF TRANSPORTATION

DIVISION OF ENGINEERING SERVICES

BRIDGE NO. X
POST MILE X

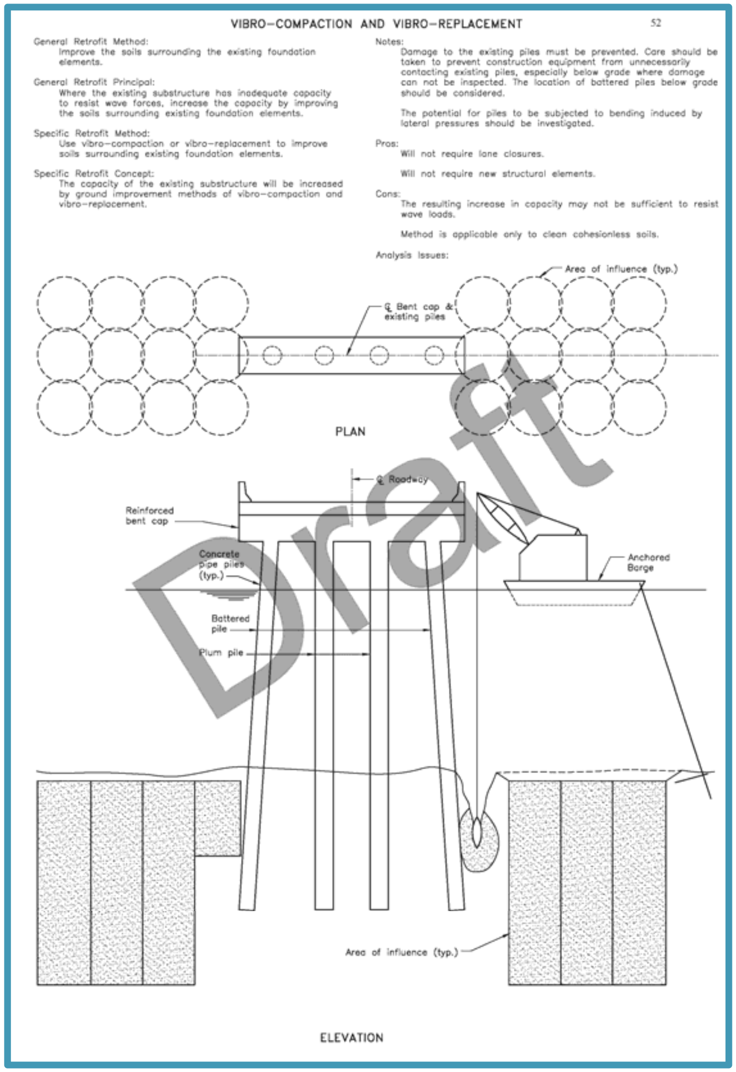
X
COLUMN CASING - FRP COMPOSITE SYSTEM

UNIT: X
PROJECT NUMBER & PHASE: X
CONTRACT NO.: X
FILE => xs7-020.dgn

CALTRANS – FRP Columns Casing

e. Strengthening Geotechnical Capacity of Foundation

- This Approach is Useful if Surge/Wave Loads Overload the Existing Foundations
- Two General Groups:
  - Auxiliary Foundations
  - Soil Improvement



VIBRO–COMPACTION AND VIBRO–REPLACEMENT 52

General Retrofit Method:
Improve the soils surrounding the existing foundation elements.

General Retrofit Principal:
Where the existing substructure has inadequate capacity to resist wave forces, increase the capacity by improving the soils surrounding existing foundation elements.

Specific Retrofit Method:
Use vibro–compaction or vibro–replacement to improve soils surrounding existing foundation elements.

Specific Retrofit Concept:
The capacity of the existing substructure will be increased by ground improvement methods of vibro–compaction and vibro–replacement.

Notes:
Damage to the existing piles must be prevented. Care should be taken to prevent construction equipment from unnecessarily contacting existing piles, especially below grade where damage can not be inspected. The location of battered piles below grade should be considered.

The potential for piles to be subjected to bending induced by lateral pressures should be investigated.

Pros:
Will not require lane closures.

Will not require new structural elements.

Cons:
The resulting increase in capacity may not be sufficient to resist wave loads.

Method is applicable only to clean cohesionless soils.

Analysis Issues:

FHWA Draft Details

f. Accepting Loss of Superstructure to Protect Substructure

- Depending on the Cost Assessment Results, in some Cases the best Approach to Retrofit a Coastal Bridge may be to Allow the Superstructure to be lost during a Storm Event
- The Presence of an Intact Substructure has Greatly reduced the Time and Cost required to put the Bridge Back in service
- <u>Due to the Critical Nature of the Bayway, this approach is considered Unacceptable</u>

Retrofit Measures to Be Considered

1. Superstructure to Substructure Uplift – Cable Restrainers
2. Superstructure to Substructure Transverse Restraint – Earwalls
3. Prestressed Beam Shear Capacity – FRP Wraps at Ends of Beams
4. Bent Cap Strengthening – Cap Retrofit
5. Foundation Strengthening – Auxiliary Piles
6. Pile Confinement – FRP or Concrete Jacket

07

Retrofits

Substructure Results – 2 Pile Bents

- Bent 282 is Representative Bent
- Total of 434 2-Pile Bents

Storm / SLR	Total Bents Modified	% Bents Modified	Total Bents Unmodified	% Bents Unmodified
25yr, SLR 2017	0	0%	434	100%
25yr, SLR 2067	0	0%	434	100%
50yr, SLR 2017	41	9%	393	91%
50yr, SLR 2067	147	34%	287	66%
100yr, SLR 2017	225	52%	209	48%

Substructure Results – 3 Pile Bents

- Bent 499 is Representative Bent
- Total of 67 3-Pile Bents

Storm / SLR	Total Bents Modified	% Bents Modified	Total Bents Unmodified	% Bents Unmodified
25yr, SLR 2017	0	0%	67	100%
25yr, SLR 2067	0	0%	67	100%
50yr, SLR 2017	16	24%	51	76%
50yr, SLR 2067	25	37%	42	63%
100yr, SLR 2017	42	63%	25	37%

Substructure Retrofits

1. Superstructure to Substructure Transverse Restraint – Earwalls
2. Bent Cap Strengthening – Cap Retrofit
3. Foundation Strengthening – Auxiliary Piles
4. Existing Pile Confinement – Concrete Jacket

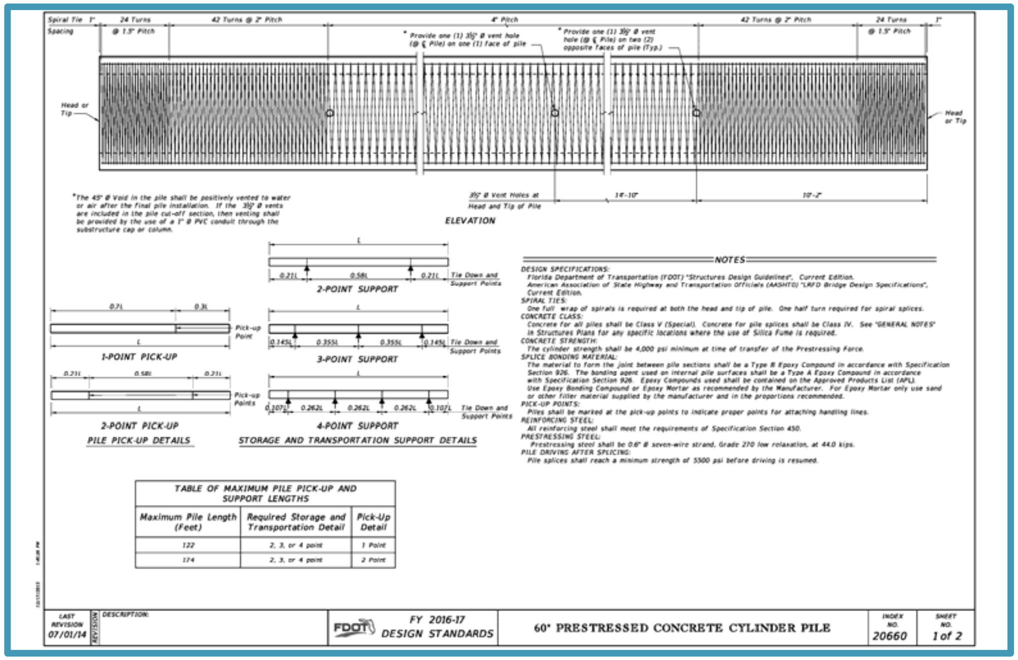
Depending on Load Intensity Along the Bridge, Various Retrofit Configurations were Developed

Foundation Strengthening - Pile Types Studied

- Foundations with High Capacities to Resist Lateral Loads
    - Drilled Shafts
    - Precast Cylinder Piles
- Consider:
    - Environmental Impacts
    - Constructibility
    - Corrosion Resistance
- Existing Bridge Pile Options were all Precast piles
- Due to Highly Corrosive Environment Steel Piles were Not Considered

Foundation Strengthening - Pile Types Studied

- Drilled Shafts were Eliminated due to:
    - Environmental Impact During Construction
    - Constructibility
    - Risk
    - Cost
- For this Study Used 60” Diameter Precast Cylinder Piles



*The 45" Ø Void in the pile shall be positively vented to water or air after the final pile installation. If the 3½" Ø vents are included in the pile cut-off section, then venting shall be provided by the use of a 1" Ø PVC conduit through the substructure cap or column.

NOTES

DESIGN SPECIFICATIONS:
Florida Department of Transportation (FDOT) "Structures Design Guidelines", Current Edition.
American Association of State Highway and Transportation Officials (AASHTO) "LRFD Bridge Design Specifications", Current Edition.

SPIRAL TIES:
One full wrap of spirals is required at both the head and tip of pile. One half turn required for spiral splices.

CONCRETE CLASS:
Concrete for all piles shall be Class V (Special). Concrete for pile splices shall be Class IV. See "GENERAL NOTES" in Structures Plans for any specific locations where the use of Silica Fume is required.

CONCRETE STRENGTH:
The cylinder strength shall be 4,000 psi minimum at time of transfer of the Prestressing Force.

SPLICE BONDING MATERIAL:
The material to form the joint between pile sections shall be a Type B Epoxy Compound in accordance with Specification Section 926. The bonding agent used on internal pile surfaces shall be a Type A Epoxy Compound in accordance with Specification Section 926. Epoxy Compounds used shall be contained on the Approved Products List (APL). Use Epoxy Bonding Compound or Epoxy Mortar as recommended by the Manufacturer. For Epoxy Mortar only use sand or other filler material supplied by the manufacturer and in the proportions recommended.

PICK-UP POINTS:
Piles shall be marked at the pick-up points to indicate proper points for attaching handling lines.

REINFORCING STEEL:
All reinforcing steel shall meet the requirements of Specification Section 450.

PRESTRESSING STEEL:
Prestressing steel shall be 0.6" Ø seven-wire strand, Grade 270 low relaxation, at 44.0 kips.

PILE DRIVING AFTER SPLICING:
Pile splices shall reach a minimum strength of 5500 psi before driving is resumed.

TABLE OF MAXIMUM PILE PICK-UP AND SUPPORT LENGTHS

Maximum Pile Length (Feet)	Required Storage and Transportation Detail	Pick-Up Detail
122	2, 3, or 4 point	1 Point
174	2, 3, or 4 point	2 Point

FY 2016-17
DESIGN STANDARDS

60" PRESTRESSED CONCRETE CYLINDER PILE

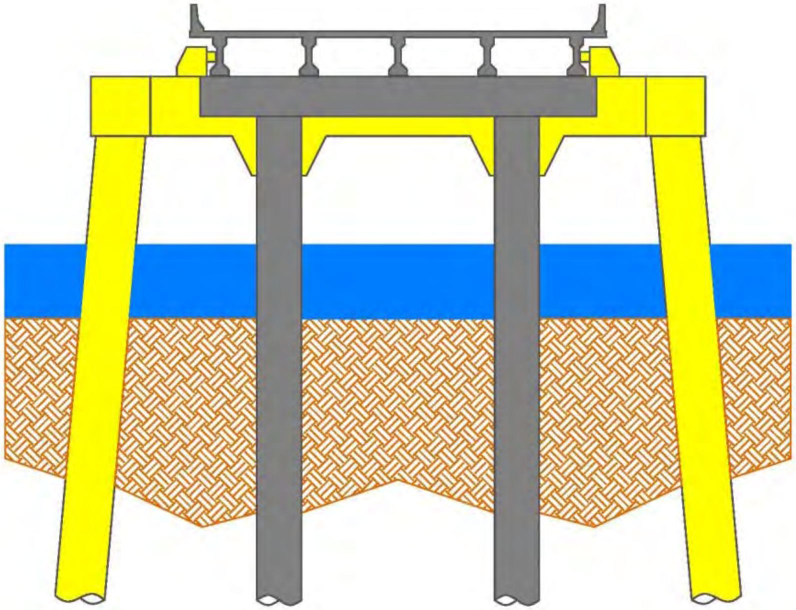
20660

1 of 2

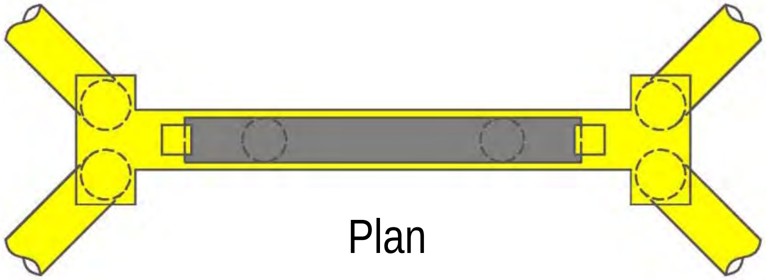
FDOT Standard Drawing

Bayway Analysis

Bent Retrofit Type A



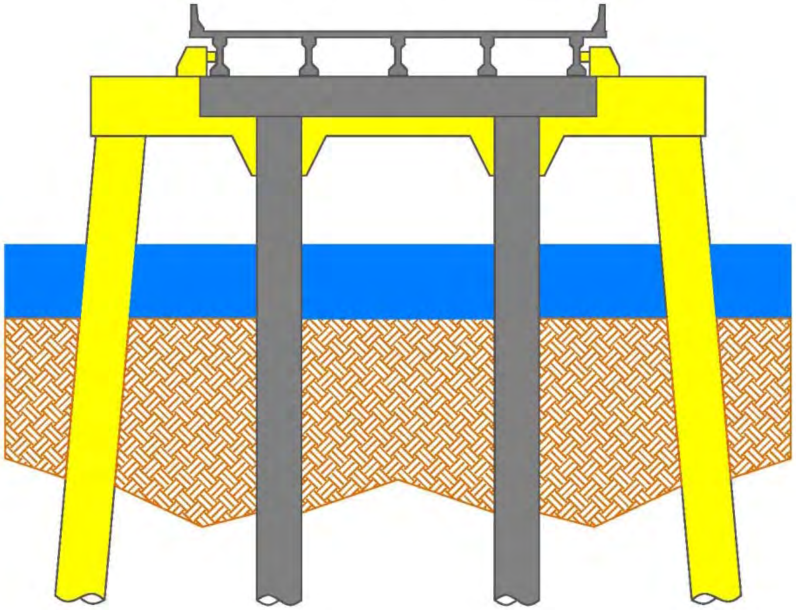
Elevation



Plan

Bayway Analysis

Bent Retrofit Type B



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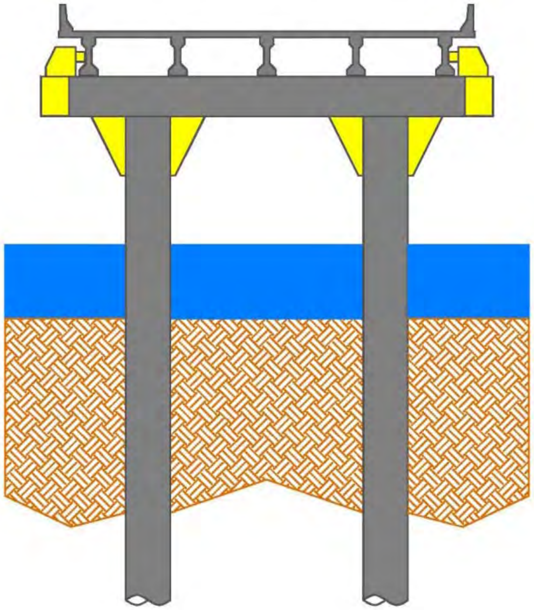
Plan



Elevation

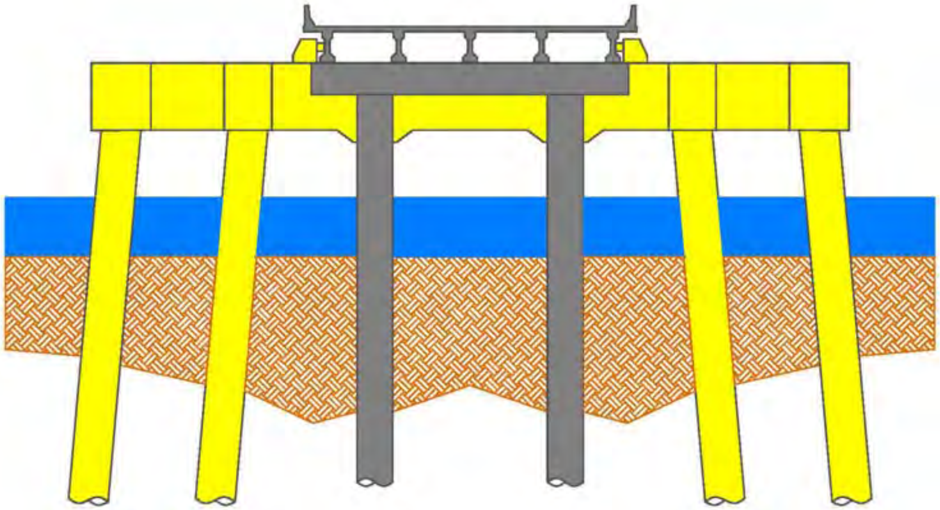
Plan

Bayway Analysis

Bent Retrofit Type C

Bayway Analysis

Bent Retrofit Type D



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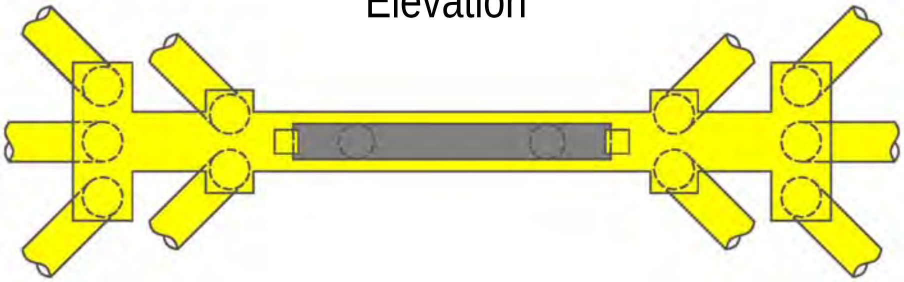

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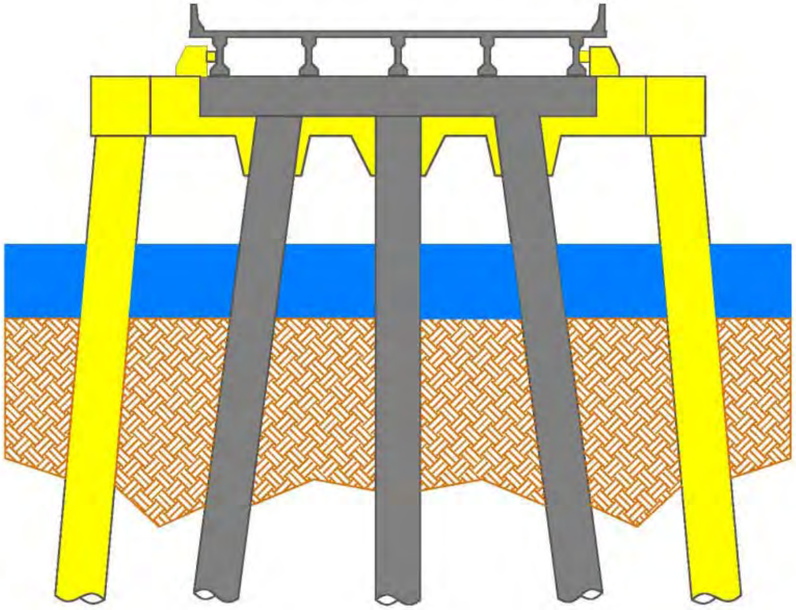

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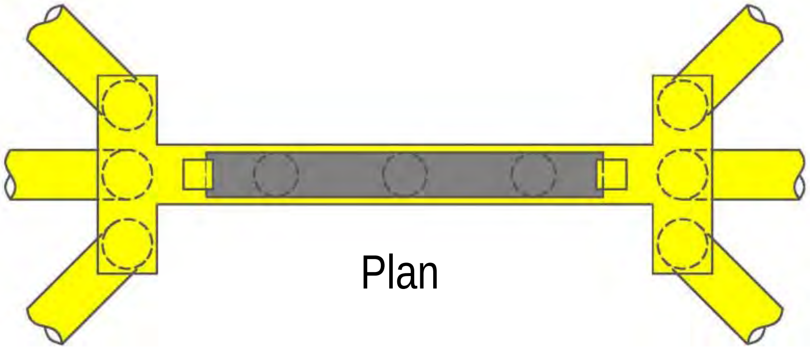


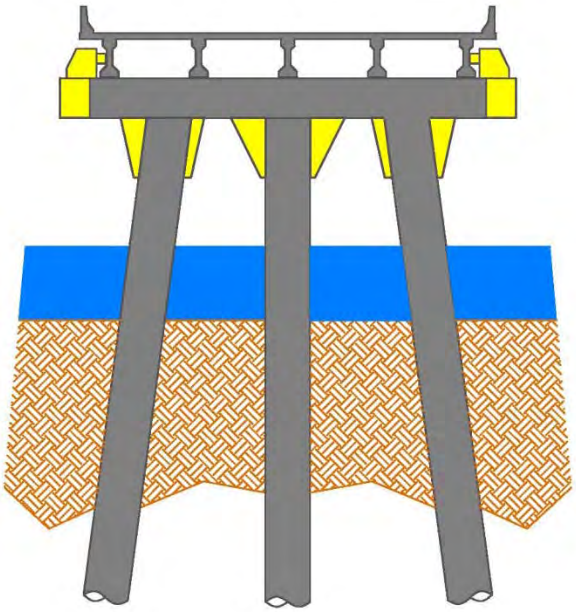
Plan

Bayway Analysis

Bent Retrofit Type E

Bayway Analysis

Bent Retrofit Type F



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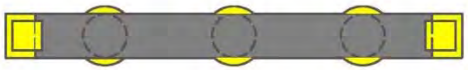
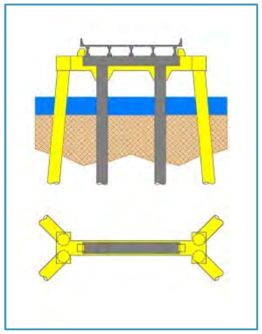
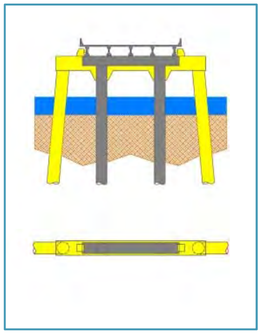
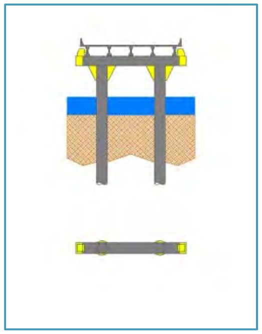


Plan

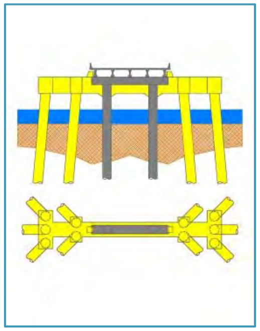
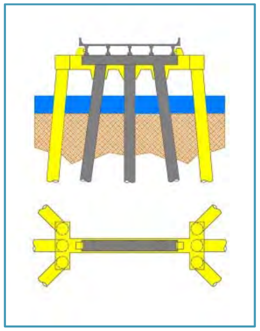
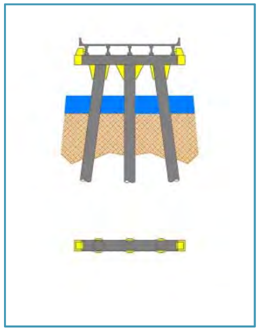
Summary of Substructure Retrofits – 2 Pile Bents

	Retrofit Type		
Storm / SLR	**A**	**B**	**C**
25yr, SLR 2017	0	0	0
25yr, SLR 2067	0	0	0
50yr, SLR 2017	0	34	7
50yr, SLR 2067	62	57	28
100yr, SLR 2017	124	101	0

Summary of Substructure Retrofits – 3 Pile Bents

	Retrofit Type		
Storm / SLR	**D**	**E**	**F**
25yr, SLR 2017	0	0	0
25yr, SLR 2067	0	0	0
50yr, SLR 2017	0	16	1
50yr, SLR 2067	0	25	9
100yr, SLR 2017	18	24	0

Superstructure Retrofits Measures

1. Superstructure to Substructure Uplift – Cable Restrainers
2. Prestressed Beam Shear Capacity – FRP Wraps at Ends of Beams
3. Span Replacement

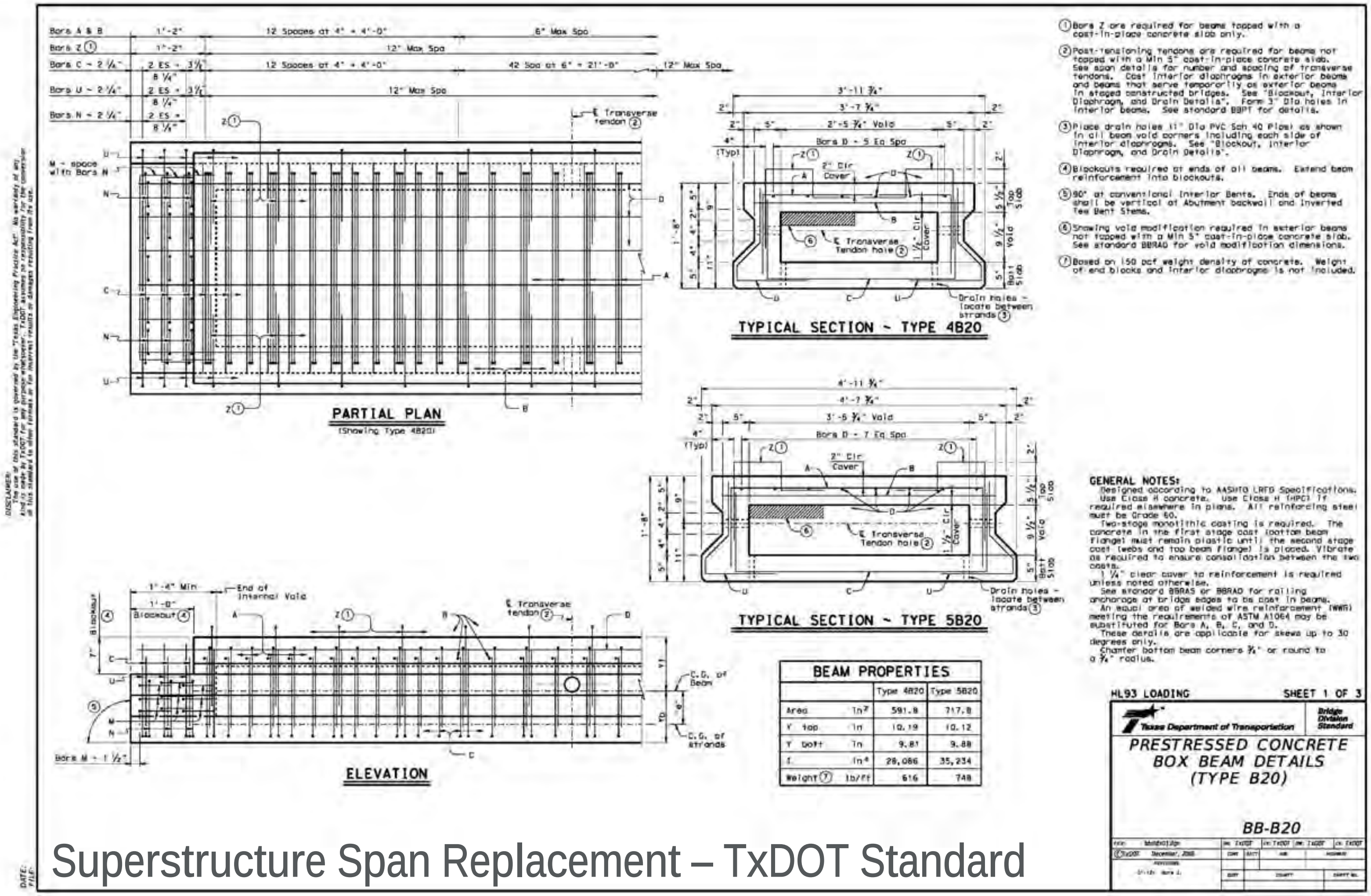
Depending on Load Intensity Along the Bridge, Various Retrofit Configurations were Developed

Negative Bending of Superstructure

- Failure of the Span Under Negative Bending is the Dominant Failure Mode of the Superstructure
- For Purposes of This Study it is Assumed that These Spans Need to be Replaced including their Connections to the Bent Caps

Storm / SLR	**Spans Lost in Negative Bending**
25yr, SLR 2017	0
25yr, SLR 2067	6
50yr, SLR 2017	51
50yr, SLR 2067	262
100yr, SLR 2017	421

Superstructure Span Replacement – TxDOT Standard



① Bars Z are required for beams topped with a cast-in-place concrete slab only.

② Post-tensioning tendons are required for beams not topped with a Min 5" cast-in-place concrete slab. See span details for number and spacing of transverse tendons. Cast interior diaphragms in exterior beams and beams that serve temporarily as exterior beams in staged constructed bridges. See "Blockout, Interior Diaphragm, and Drain Details". Form 3" Dia holes in interior beams. See standard BBPT for details.

③ Place drain holes (1" Dia PVC Sch 40 Plas) as shown in all beam void corners including each side of interior diaphragms. See "Blockout, Interior Diaphragm, and Drain Details".

④ Blockouts required at ends of all beams. Extend beam reinforcement into blockouts.

⑤ 90° at conventional Interior Bents. Ends of beams shall be vertical at Abutment backwall and Inverted Tee Bent Stems.

⑥ Showing void modification required in exterior beams not topped with a Min 5" cast-in-place concrete slab. See standard BBRAD for void modification dimensions.

⑦ Based on 150 pcf weight density of concrete. Weight of end blocks and interior diaphragms is not included.

GENERAL NOTES:

Designed according to AASHTO LRFD Specifications.

Use Class H concrete. Use Class H (HPC) if required elsewhere in plans. All reinforcing steel must be Grade 60.

Two-stage monolithic casting is required. The concrete in the first stage cast (bottom beam flange) must remain plastic until the second stage cast (webs and top beam flange) is placed. Vibrate as required to ensure consolidation between the two casts.

1 ¼" clear cover to reinforcement is required unless noted otherwise.

See standard BBRAS or BBRAD for railing anchorage at bridge edges to be cast in beams.

An equal area of welded wire reinforcement (WWR) meeting the requirements of ASTM A1064 may be substituted for Bars A, B, C, and D.

These details are applicable for skews up to 30 degrees only.

Chamfer bottom beam corners ¾" or round to a ¾" radius.

BEAM PROPERTIES

		Type 4B20	Type 5B20
Area	In²	591.8	717.8
Y top	In	10.19	10.12
Y bott	In	9.81	9.88
I	In⁴	28,086	35,234
Weight ⑦	lb/ft	616	748

HL93 LOADING SHEET 1 OF 3

Texas Department of Transportation

Bridge Division Standard

PRESTRESSED CONCRETE
BOX BEAM DETAILS
(TYPE B20)

BB-B20

08 High-Level Cost Estimates

High-Level Cost Estimating Approach

- Based on Historical Bid data of projects with similar scope
- Life Cycle and Maintenance Costs not Included
- Foundation Load tests Costs not Included
- 2017 Costs Used – no Escalation Costs Applied
- 20% Contingency and 20% Overhead (Mobilization, E&I, etc.)
- For construction over open water, floodplains that flood frequently or other similar areas, increase cost by 3 % (FDOT) Does ALDOT apply similar factors?

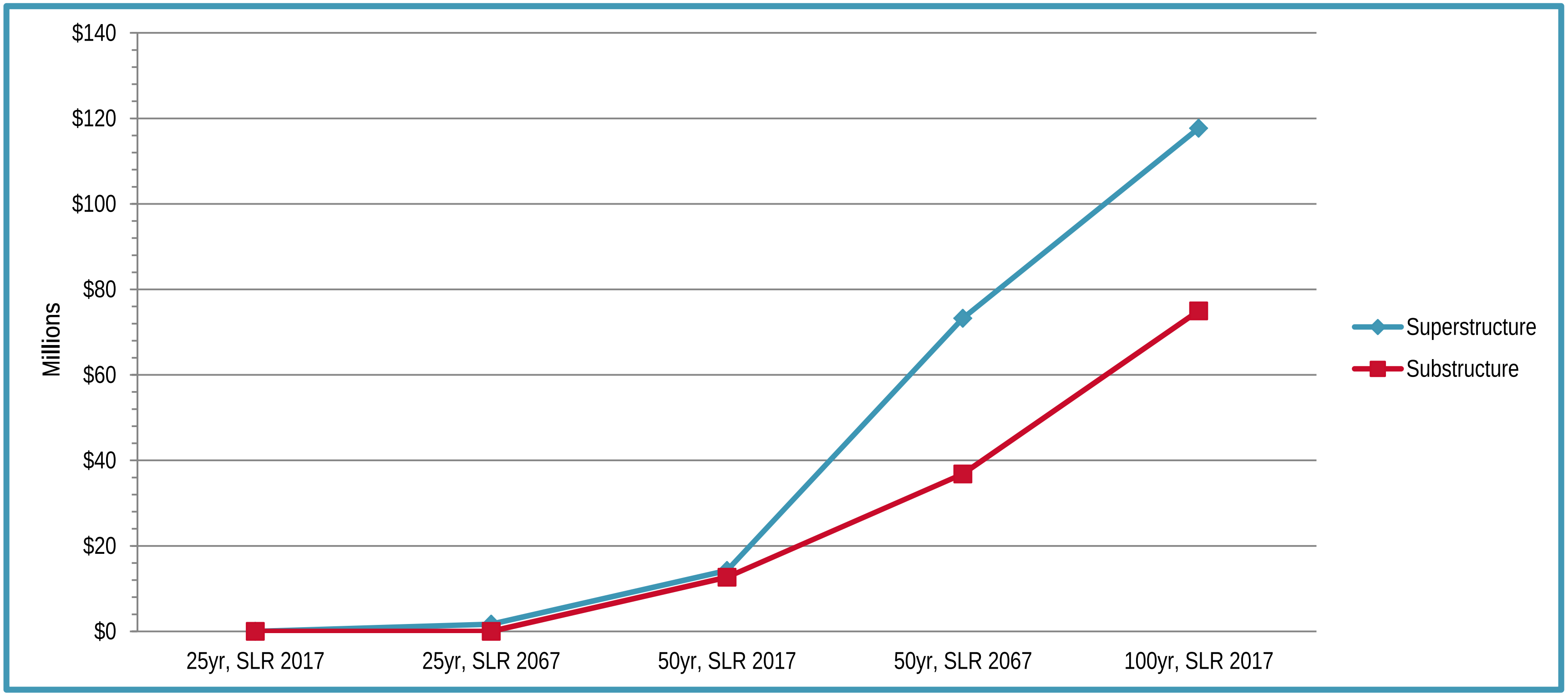
High-Level Cost Estimating

- Assumed Unit Costs (Would Like to Request ALDOT's input on These Values):

Items	Measurement	Unit Cost
Concrete	CY	$700
Reinforcing Steel	LBS	$0.90
60" Cylinder Pile	LF	$425
Replacement Span	EA	$279,500

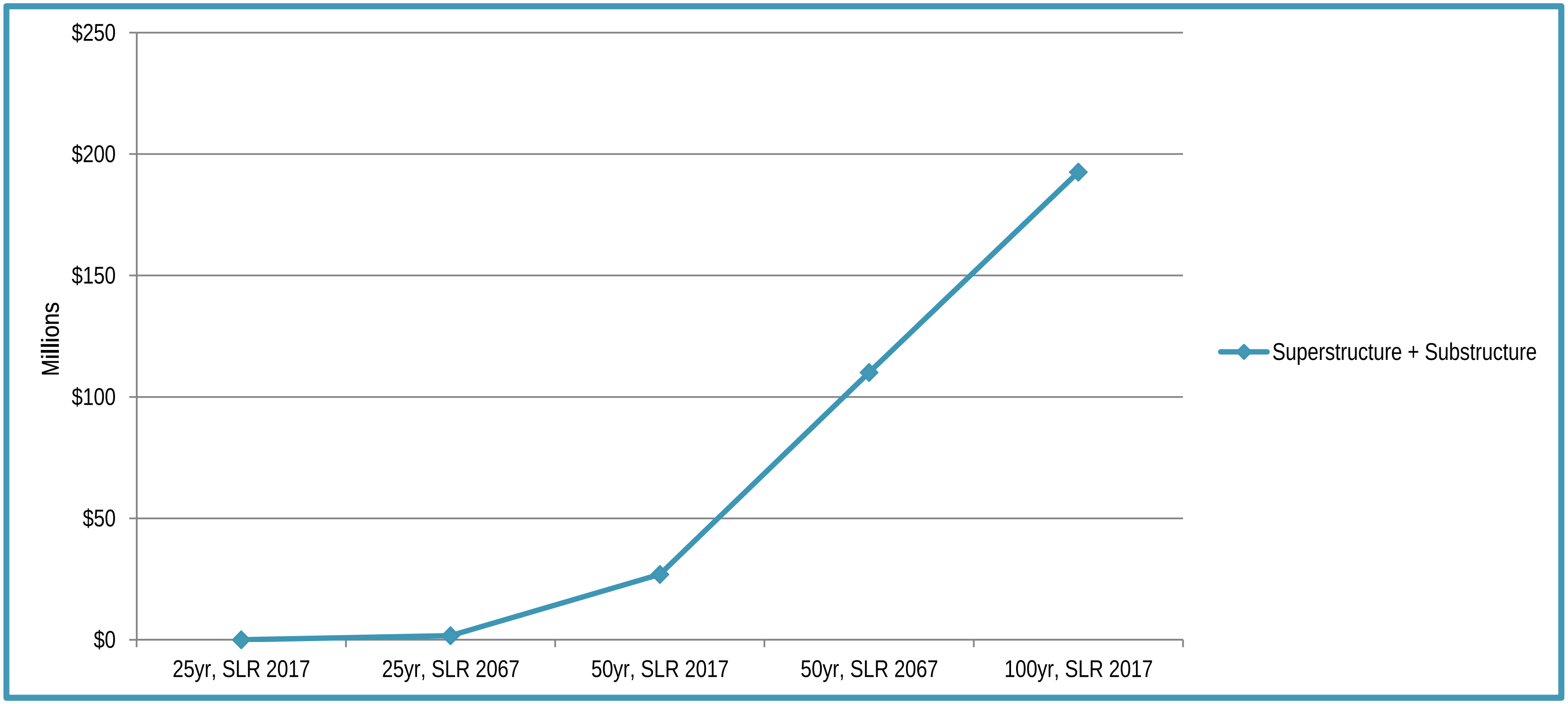
High-Level Cost Estimating

- Substructure and Superstructure Costs (Bridge Construction Only):



High-Level Cost Estimating

- Substructure and Superstructure Costs (Bridge Construction Only):



High-Level Cost Estimating

- Total Costs:

Storm / SLR	Bridge Construction	Contingency 20%	Overhead 20%	Total
25yr, SLR 2017	$0	$0	$0	$0
25yr, SLR 2067	$2M	$.4M	$.4M	$3M
50yr, SLR 2017	$27M	$6M	$6M	$39M
50yr, SLR 2067	$110M	$22M	$22M	$154M
100yr, SLR 2017	$193M	$39M	$39M	$271M

09

Next Steps

Next Steps

- Get ALDOT’s Input on Unit Costs and Update Estimates
- Get Thompson’s input on Pile Lengths and Update Estimates
- Prepare Design Memorandum
- Discuss Level III Analysis
- Incorporate Level III Analysis (if needed)
- Prepare Final Report

Thank you for your time

QUESTIONS?