Progress Report

March 5, 2007

PROPOSAL TO THE FEDERAL HIGHWAY ADMINISTRATION

TASK ORDER DTFH61-06-T-70006

**FOR THE DEVELOPMENT OF
GUIDE SPECIFICATIONS FOR BRIDGES VULNERABLE TO COASTAL STORMS
AND
HANDBOOK OF RETROFIT OPTIONS FOR BRIDGES VULNERABLE TO COASTAL STORMS**

by

Modjeski and Masters, Inc.

with

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Ocean Engineering Associates, Inc.
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Dr. Dennis R. Mertz

INTRODUCTION

We received Notice to Proceed on this Work Order on August 14, 2006.

This report covers work done in February, 2007. During this month the team held one conference call (Feb 16). Progress of the work on different tasks and internal team assignments were discussed during the meeting.

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TASK 1 – MEETINGS

No new meetings with the BWTF took place in this reporting period.

We are requesting that the Task Force call a meeting between our team and either the full Task Force or the Task Force members with hydraulics/waves interests and those from the FHWA in early to mid April. (we are proposing that those of the following whose schedules allow them to attend be invited: Rick Renna, David R. Henderson, Tom Everett, Joseph Krolak, Dr. Kornel Kerenyi, Dr. Firas Ibrahim, Dr. Robert A. (Tony) Dalrymple, Dr. David L. Kriebel). If the face-to-face meeting will include the full task force, we are suggesting canceling the telephone conference call between the research team and the task force (currently scheduled for March 20th). If the face to face meeting will not include the full task force, we are suggesting that the conference call be postponed and be held after the face-to-face meeting.

Our team's work to-date on determining and comparing different methods of calculating wave forces will be presented in the meeting. A recommendation of selecting the method of calculations to be incorporated in the specifications and discussing this recommendation may also be included in the meeting.

TASK 2 – REVIEW, SUMMARIZE, AND AUGMENT LITERATURE

A request for information was sent to all of the coastal states and to a variety of other agencies identified by the Project Team. We received a report on Oregon's efforts to prepare for Tsunamis. Other than this report, no new responses were received in the last two months. We are proceeding on the assumption that it is unlikely that additional significant information responses will be received.

Relevant information identified in the literature have been reviewed and summarized by the project team. The summary of the articles identified in the literature is included in Attachment A.

Bridge damage observed after costal storms was reviewed and the pattern of observed damage was catalogued. This information was delivered to the project BWTF during the December, 2006, kick-off meeting. After the December meeting, a minor additions to this document was incorporated. A copy of the updated catalogue of past observed damage is included in Attachment A.

The project team performed some calculations to determine the resistance of the components which failure may have caused the collapse of spans in both I-10 Escambia Bay and I-10 Lake Ponchartrian bridges. These resistances will be used to compare to the calculated wave forces in these bridges. A copy of these calculations is also included in Attachment A.

Attachment A represents the product of the work under Task 2. This document is a feeder document to other tasks of the project and will be included in the final deliverables of other tasks. **We regard the work on Task 2 as complete and we are requesting permission to invoice for this task**. The information in Attachment A may be updated as warranted as part of the work on later tasks (work on later tasks will require incorporating this information, or parts of it in the project final deliverables).

TASK 3 – REVIEW AND SUPPLEMENT ONGOING FORCE STUDIES

The work on wave force calculations and comparisons continued. Attachments B, C, and D include the following force calculations:

Attachment B (Prepared by OEA): The results of the wave force calculations for one span of Escambia Bay Bridge under different wave conditions are presented. The parameters considered include:

- Depth of water
- Significant wave height
- Maximum wave height
- Wave period
- Wave length
- Wave crest height above storm water level
- Distance from storm water level to bottom of girder

Plots for maximum forces are included for Kaplan, New Wallingford and Douglas methods. Kaplan's method consistently produced lower forces. Wallingford method produced higher forces than Douglas method in some cases and lower forces in other cases.

Also of interest are the observations regarding the dependence of the forces and moments on wave period (or more accurately on wave steepness). As the waves become more steep [increased (wave height)/(wave length)] the drag, inertia and change in added mass forces increase but the slamming forces decrease. The increase in the magnitude of the slamming force as the water surface becomes flatter is reasonable since it is contact of the water surface with the underside of the span that causes this impulsive force. This means that accurate predictions of not only the design wave heights but also the associated periods will be required to obtain accurate calculated wave forces.

Attachment C (Prepared by OEA):
The analysis results for one span of I-10 Escambia Bay Bridge under varying storm

water levels and different wave periods for both 8 ft. and 12 ft. wave heights are presented in this attachment. The calculations were made for Modified Kaplan, Wallingford and Douglas methods. As indicated above in the comments on Attachment B, Kaplan method gives significantly lower forces compared to the other two methods.

For Kaplan Method, the following observations are based on the information in the tables:

- The vertical force continues to increase when the bridge is inundated and the wave period is short (4 sec.). For wave periods of 6 and 8 sec., the vertical force increase when Y_c decreased from 3 ft to 0.0 ft., i.e. from water level 3 ft below the bottom of the beams to water level at the bottom of the beam. The force decrease as the water level rose from the bottom of the beam to 9 ft above the bottom of the beam(inundated condition)
- For all wave periods considered, the moment about the trailing and leading edges increased as the water rose from 3 ft below the bottom of beams to 9 ft above the bottom of beams. However, the increase was much higher for wave periods of 4 sec. than for wave periods of 6 and 8 sec.

These observations reinforce the earlier comments on the dependence of the forces and moments on wave period.

Attachment D (Prepared by OEA and Moffatt Nichol): Wave force calculations for all spans of I-10 Escambia Bay Bridge for both the Modified Kaplan and Wallingford methods are included. The bottom of the solid concrete diaphragms on this bridge is higher than the bottom of the girders. For slowly rising water, the volume of the entrapped air is 74% of that would be entrapped if the diaphragms were extended to the bottom of girders. The calculations were conducted for the cases of no air entrapment and maximum, i.e. 74%, air entrapment. The actual air entrapment is expected to be in between the two values and, thus, these two cases bound all possibilities.

The predicted wave forces were compared to the resistance provided by friction and the weight of the span. The calculation results were compared to the actual observed failure of the bridge spans to determine if the calculations can accurately predict the failure.

Intermittent failure was observed in the field (i.e. some spans did not collapse but were preceded and followed by collapsed spans). This intermittent damage is attributed to:

- Differences in the maximum waves that hit those spans (the calculations used the most likely maximum wave heights), and;
- Differences in the resistance (reduced strength due to corrosion, etc., bolt anchor strength being less than the specified bolt strength, etc.).

Wallingford Method produced higher forces than the Modified Kaplan Method. This means that the “coefficient” applied to the base pressure is higher than its optimum value for this bridge. Wallingford’s work was based on tests conducted using generally longer wave lengths compared to the structure width. This resulted in having uplift pressures along the entire width of structure; thus producing a relatively high uplift force. (note that the tests in the University of Florida where shorter wave lengths are used, there are some areas of uplift and some downward forces. Thus, the net force is much lower)

Due to the extensive amount of calculations involved, only the graphs showing the comparisons are included in the attachments.

Attachment E (Prepared by OEA) with review and input from M&M: A document entitled:" A Method for Estimating Bridge Span Resistance to Storm Surge and Wave Loading for Girder Type Bridge Superstructures" was prepared by OEA's members of our team. The calculations used to determine whether a span subjected to wave forces of a given magnitude will fail. A numerical example is included in the document.

TASK 4 – COMPILE AND CATALOG RETROFIT OPTIONS

A preliminary catalog was submitted to the BWTF during the kick off meeting. We have not received any comments on the submission beyond those received during the meeting.

More concepts were added to the catalog after the kick-off meeting and the proposed concepts were organized in groups based on the common features of the concepts. Concepts for substructure and foundation strengthening were also added. Attachment F includes the catalogue of retrofit option.

The catalog of retrofit options is a feeder document for later tasks. It will form the basis for the retrofit options to be included in the retrofit manual. As the work on the retrofit manual progresses, the most feasible retrofit concepts will be selected and, if needed, will be updated. **We regard the work on Task 4 as complete and are requesting permission to invoice for this task**.

TASK 5 – PERFORM ANALYTICAL STUDY OF RETROFIT OPTIONS

No progress to-date.

TASK 6 – DEVELOP A GUIDE SPECIFICATION AND A RETROFIT HANDBOOK FOR ADOPTION BY AASHTO

TASK 6A - GUIDE SPECIFICATION

A strawman of the proposed specifications has been updated. This document does not include all important wave force calculation procedures; which will be added later. This document will be attached to next month's progress report.

Mr. Shelden prepared a memorandum on the selection of the coefficient of variation (COV) to be used in probability-based specifications. This memorandum is attached as Attachment G.

TASK 6B - RETROFIT HANDBOOK

See above.

TASK 7 – DEVELOP FINAL REPORT AND RECOMMENDATIONS FOR FURTHER STUDIES

No progress

TASK 8 – PREPARE EXECUTIVE SUMMARY AND PRESENTATION MATERIALS

No progress

FUTURE WORK – NEXT MONTH

1. Formulate a recommendation on wave force calculation process.

3. Continue working on the strawman for design specifications to which was started in the month (January) as a way to organize thoughts and focus efforts.

4. Continue to research the reliability and recurrence issues.

SCHEDULE

The schedule previously agreed to is shown below as "Proposed Completion Dates".

Task 2 – We regard this task as complete

Task 3 – The comparative studies of four wave force prediction method was completed and appended hereto. The decision as to which method to recommend and development of any possible design aids will take longer than the February 28th completion date; probably until Late March.

Task 4 – We regard this task as complete

Task 5 – We need a decision on the wave force method before starting this task so we are proposing extending the deadline for this task from April 15 to April 30 as shown in the revised schedule below.

At the moment no other dates are in jeopardy.

TASK	Date shown in Work Plan	PROPOSED COMPLETION DATES
Notice to Proceed	September 1, 2006	
Kickoff Meeting	December 4,5,6, 2006	
Task 2	December 15, 2006	January 15, 2007
Task 3	December 15, 2006	February 28, 2007
Task 4	January 26, 2007	March 31, 2007
Task 5	March 2, 2007	April 30, 2007
Task 6 50% Draft Specification and Manual 90% Draft Specification and Manual 100% Draft Specification and Manual Interim Report Tasks 2 to 6	 February 15, 2007 May 31, 2007 August 15, 2007 July 15, 2007	 April 30, 2007 July 31, 2007 October 15, 2007 September 15, 2007
Task 7 Draft Final	 June 30, 2007 September 15, 2007	 August 31, 2007 November 15, 2007
Task 8 – Executive Summary Draft 4 to 6 page summary Final 4 to 6 page summary	 June 30, 2007 August 31, 2007	 August 31, 2007 October 31, 2007
Task 8 – 13 hour slides Draft Final	 November 30, 2007 January 31, 2008	 January 31, 2008 March 31, 2008

WAGDY G. WASSEF

Attachment A

Task 2 Submission

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TASK 2 – REVIEW, SUMMARIZE AND AUGMENT LITERATURE

LITERATURE SURVEY

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TASK 2 – REVIEW, SUMMARIZE AND AUGMENT LITERATURE

Literature Survey

Updated: March 1, 2007

ASCE

Title: "Hurricane Katrina, Performance of Transportation Systems"
Authors: Reginald DesRoches (ed.)
http://www.asce.org/bookstore/book.cfm?book=6281
Date: April 2006

Comments: Good summary of post Katrina damage observations. Lists 44 highway bridges with varying amounts of damage. The list includes 34 movable bridges or movable spans in a series of fixed spans. The response of several railroad bridges are also discussed as is nonstructural aspects such as roadways and traffic rerouting issues. The report contains numerous photographs and detailed narratives describing damage to bridges. In some cases the reconstruction process is described. The action of buoyancy and wave forces are presented as the apparent cause for the destruction of most of the bridges that were lost. For example, the report states that the weight of each span of the I-10 across Lake Pontchartrain Bridge weighed 500 kips and the hydrodynamic uplift was 900 kips. The report states that raising bridge grades to clear storm surges (and presumably waves) is a good approach when it can be accommodated. Otherwise, venting to reduce buoyancy and tie downs to restrain the superstructure are possibilities.

Title: "ASCE 7-02, Minimum Design Loads for Buildings and Other Structures" (2005 ed now available)
Authors: ASCE
Date: 2002

Comments: Provides minimum load requirements for the design of buildings and other structures that are subject to building code requirements. Loads and appropriate load combinations are set forth for strength design and allowable stress design. Load combinations include load factors for Flood Loads. Methods are provided for calculating flood loads for hydrostatic loads (caused by a depth of water to the level of the design flood); hydrodynamic loads (dynamic effects of moving water); wave loads (water waves propagating over the water surface and striking a building or other structure); and breaking wave loads on vertical pilings, columns and vertical and non-vertical walls.

ENR Construction Articles

Title: "A 24-Mi Bridge Across Louisiana's Lake Ponchartrain" Engineering News-Record Vol. 157 No. 9 Pages 30-33
Authors: Unknown
Date: August 30, 1956

Comments: This article summarizes several aspects of the construction of the Lake Ponchartrain Causeway. The economics, geometry, construction methods, construction time table, and traffic capacity of the causeway are discussed. The discussion of construction methods is of particular interest. Specifically addressed are the precasting and prestressing methods used in manufacturing the 54 inch diameter post-tensioned concrete cylinder piles and the monolithic pre-tensioned concrete bridge spans used to construct the causeway. Concrete strengths, wire/strand types and tensions, grouting methods, and construction sequences are discussed. It has been found that these details are not always given on the contract drawings for similar bridges built around this time. These items are critical for the forensic investigation of bridges impacted by coastal storms. Without these details, forensic analyses must use assumptions, which may lead to inaccurate estimates of failure loads. The builder of the bridge, Louisiana Bridge Co. (a joint venture of Brown & Root Inc. and T.L. James & Co.) constructed other bridges (including the 1960 Pensacola Bay Bridge and the 1963 I-10 Lake Ponchartrain Bridge) using the same or similar methods. Therefore, this article provides a good starting point when details of construction must be assumed during the forensic investigation of similar bridges.

Title: "Florida Spans: Made in Louisiana" Engineering News-Record Vol. 165 No. 3 Pages 45-47
Authors: William E. Dean
Date: July 21, 1960

Comments: This article summarizes several aspects of the construction of the Pensacola Bay Bridge in Florida. The economics, geometry, construction methods, construction time table, and design considerations of the bridge are discussed. The bridge was built by the same companies that constructed the Lake Ponchartrain Causeway using similar construction methods (the August 30, 1956 ENR article is referenced). The discussion of construction methods provides additional information beyond that provided in the 1956 article. Specifically addressed are the connection between the cylinder piles and the pile caps, camber considerations for the monolithic span units, and additional information on wire/strand types and tensions. Pictures and drawings of the pile to pier cap connection are provided. This (or a similar) detail, which uses a cast in place pile plug poured through a hole in the precast pile cap, was used on the Escambia Bay Bridge in Florida. The pile-to-pier cap connections of the Escambia Bay Bridge were damaged during Hurricane Ivan. Details of construction given in this article will provide a good starting point when details of construction must be assumed during the forensic investigation of similar bridges.

Title: “Slidell Bridge Nears Completion” Engineering News-Record Vol. 170 No. 14 Page 14
Authors: Unknown
Date: April 4, 1963

Comments: This brief article summarizes several aspects of the construction of the I-10 Lake Ponchartrain Bridge. A limited amount of information regarding construction is given. It is stated that the construction of the I-10 Slidell Bridge is similar to that of both the Ponchartrain Causeway and Pensacola Bay Bridge. The builder of the bridge is listed as Brown & Root Inc.

Florida Department of Transportation (William N. Nickas) – 8/30/06 – On CD

Title: White paper on the “Upcoming FDOT Modeling of Wave Forces on Bridges”
Authors: ????
Date: ???

Comments: A description of Florida Department of Transportation (FDOT) funded wave-loading research for analyzing bridge superstructures. The research is composed of three components. The first is a series of laboratory tests to measure wave loads on a model bridge span subjected to varying wave conditions. The objective of the first component is to relate wave forces to the structure configuration and sea conditions. The second component develops a numerical model to predict the wave induced forces using the laboratory tests as calibration data. The last component is the development of a risk-based bridge design procedure.

Title: Scope of Service – “Wave Loading on Bridge Decks”
Authors: D. Max Sheppard
Date: June 2005

Comments: As the title states, this document describes the scope of the University of Florida’s (UF) laboratory model tests of wave loading on bridge decks. The document details the forces that UF will measure as well as the model setup and the experiments. planned. The research will attempt, through a complex array of instruments, to delineate the wave and water induced forces into their components. The three components — hydrostatic, dynamic, and shear — are described in detail along with their relative importance. A listing of the tasks provides a sense of the range of experiments planned. The scope also includes a schedule.

Title: Progress Report on “Wave Loading on Bridge Decks”
Authors: D. Max Sheppard and Justin Marin
Date: March 2006

Comments: This document describes the progress of the FDOT/UF Wave Loading on Bridge Decks Study (described in Scope of Services – “Wave Loading on Bridge Decks”) up to March 2006. At that point in time, most of the bridge model and instrumentation were constructed. The researchers found that additional instrumentation was required to isolate all the components. Construction of the additional instrumentation may delay the project schedule.

Title: Draft – “Storm Surge and Wave Force Vulnerability Pilot Study in Tampa Area Phase I”
Authors: Ocean Engineering Associates, Inc.
Date: March 2006

Comments: The report provides a well thought out wave vulnerability screening process for bridges. It describes phase I of a two-phase pilot study to evaluate bridge vulnerability to storm surge and wave attack. This first phase describes a procedure to screen bridges using readily available data and simple empirical equations. The report includes a method to calculate a vulnerability index based on physical parameters. Those parameters include buoyancy, wind/fetch alignment probability, location of the bridge in the water column, structure type, and importance. The methodology is then applied to evaluate Florida DOT District 7 (Tampa area) bridges. The study identified 50 state and 2 county bridges in the District of which the screening algorithm eliminated 17. Appendices give data sources and example applications for determining wave conditions.

Title: Work Proposal – “Wave Loading Vulnerability of Florida’s Coastal Bridges Phase II”
Authors: Ocean Engineering Associates, Inc.
Date: April 2006

Comments: This document is a proposal for a second phase of the investigation on Wave Loading Vulnerability of Florida’s Coastal Bridge project. The first phase of the project lead to a screening algorithm. The primary object of this phase is to verify that the algorithm identifies the most vulnerable bridges. Florida DOT District 7, which contains Tampa Bay, was selected as the testbed. There are 50 states and 2 county bridges in the District of which only 17 were eliminated by the screening algorithm. One bridge in District 1 was also in the study group. This project is to evaluate the vulnerability of bridges by refining the variables used in the screening algorithm and also calculate wave forces on selected structures using a variation of the Kaplan method, and then apply those loads to structures and determine if they can survive. Improvements to fetch using actual hurricane paths in NOAA’s database, surge values from Florida DEP compared to FEMA, wave models from SWAN instead of the USCOE empirical equations will be incorporated. The results of these investigations will be used to refine the screening algorithm. This work is in progress as of this writing.

Title: Scope of Service – "Development of Probabilistic Bridge Design Procedures for Wave Forces"
Authors: D. Max Sheppard
Date: July 2006

Comments: This reference contains the problem statement, objectives, tasks and deliverables for research proposed to the Florida DOT to develop procedures to assess the vulnerability of existing bridges to wave forces and to design new highway bridges for wave forces. Three objectives are listed: 1) Produce guidelines for addressing the joint probability of surge and wave in Florida coastal waters. 2) Produce plots of nondimensional parameters involving the horizontal and vertical forces, wave and surge parameters and bridge parameters using a combination of theoretical equations, wave task test results, and observations. 3) Produce Design Guidelines for Florida DOT structures staff to address wave loads. This work is underway at this writing.

Title: Final Report – "Design Storm Surge Hydrographs for the Florida Coast"
Authors: D. Max Sheppard and William Miller, Jr.
Date: September 2003

Comments: The objective of this study is to recommend hydrographs for 50, 100 and 500 year return period and storm surge peaks for the entire coastline of Florida. Peak storm surge results tabulated by the Florida Department of Environmental Protection (FDEP), the Federal Emergency Management Agency (FEMA), The National Atmospheric and Oceanic Administration (NOAA) and results obtained by the U.S. Army Corps of Engineers' ADCIRC model are compared for various sites for 50, 100 and 500 year return periods. The recommended agency whose peak surge data are thought to be best is shown for each location considered. The values presented by FDEP are recommended at sites covered in their study and are the most often recommended. Hydrographs prepared for low tide and high tide using the FDEP model and the results of using synthetic hydrographs developed by a pooled fund study entitled "Development of Hydraulic Computer Models to Analyze Tidal and Coastal Stream Hydraulic Conditions at Highway Structures" are compared for a 100 year event, presented graphically and discussed. Recommended hydrographs with rising and falling limbs are presented for various locations for 50, 100 and 500 year events. A companion plot shows the 100 year hydrograph along with the hyprographs which would have occurred if the time line was reduced to 70% or increased to 160% of the basic value.

Indian Institute of Technology

Title: "Storm Surge" presented at Fifth International Workshop on Tropical Cyclones, Cairns, Australia
Authors: S. K. Dube
Date: December, 2002

Comments: This report describes progress and improvements of the storm surge models (including inland inundation).

Louisiana Department of Transportation and Development (Artur D'Andrea) – 10/7/06

Title: "Design Criteria I-10 Bridge Over Lake Pontchartrain S.P. 450-17-0025"
Authors: Louisiana Department of Transportation and Development
Date: March 21, 2006

Comments: Presents a table of Load Combinations and Load Factors used by LADOTD for the Design of the I-10 Bridge over Lake Pontchartrain. Includes two storm surge combinations based on a strength case and an extreme case. Load factors are provided for quasi-static storm surge / wave forces and dynamic or impact storm surge / wave forces.

Louisiana Department of Transportation and Development

Title: "Hurricane Katrina Damage Assessment"
Authors: Volkert & Associates, Inc.
Date: September 2-4, 2005

Comments: This report was prepared for the Louisiana Department of Transportation and Development and constitutes a span-by-span summary of major findings on the I-10 Lake Pontchartrain Bridge. Tables are provided which contain areas for remarks, deck repair quantities, curb repair quantities, railing repair or retrofit, issues of beam slab misalignment, beam slab relocation, removal and disposal of slabs, removal and disposal of bents, and so on. A brief narrative describes a summary of findings indicating that there were 26 westbound and 38 eastbound spans in the water, 303 westbound, and 170 eastbound spans shifted, 13,910 lineal feet of barrier railing missing westbound, and 130 lineal feet eastbound, and one bent missing on the westbound structure. It was pointed out that shear studs used to connect bearings to girders had appeared to have been corroded prior to the storm, and this may have been a contributing factor in allowing spans to shift without associated damage to anchor bolts or bearing assemblies, i.e., the beams slid relative to the sole plates. It was observed that in many cases the misalignment of spans was limited to 5 feet, at which time Girder 1 made contact with the pedestal for Girder 2 preventing further dislodgement of the superstructure. Nine pages of photographs further document the types of damage noted.

Title: "Hurricane Storm Surge, Waves and Hydrodynamic Loads Design Report I-10 Bridge Replacement"
Authors: Moffatt & Nichol
Date: September 21, 2006

Comments: Report documents storm surge modeling, wave hindcasting, statistical analyses and wave force determination performed for the replacement of the I-10 Bridge over Lake Pontchartrain.

Mississippi Department of Transportation

Title: “Repair of Hurricane Camille Damage to U. S. 90 Bridges”
Authors: Julian R. Barksdale
Date: 1970

Comments: This document is an internal memorandum that describes the Mississippi Department of Transportation’s response to the August 17, 1969, storm known as Hurricane Camille. This storm resulted in winds reportedly exceeding 200 mph and tide 6.5 feet higher than any other recorded tide in the Biloxi Bay and Bay St. Louis areas. Both of these structures were severely damaged, and after initial inspection the Department took steps to have repair contracts underway within 11 days. At that time the Bay St. Louis Bridge has been reported to have been a toll facility.

The Bay St. Louis structure is reported as a series of monolithically cast reinforced concrete girders of 41-foot span, except for the bascule bridge which was longer and of steel construction. The 41 ft. spans had been cast as single units and each monolithic unit weighed approximately 130 tons. It had been raised sufficiently for rocker bearings to wash out after the spans to be displaced landward as much as 4 feet. Curb and barrier railings sections were missing or layed over in place held by reinforcing steel. Damage to the substructure was reported as fairly light consisting of spalled corners and surfaces where the superstructure spans had been raised and dropped on exposed concrete bent caps. There were some cases where spalling was evident around the piles which was seen as an indication of large bending moments present at the top of the piles during the storm. Many girder ends were heavily damaged with some crack and spall beyond repair. Some girders were cracked their entire length. Many of these spans were repaired and repositioned on the original substructure. Eight exterior girders on the south side of the bridge had to be replaced entirely.

The Biloxi Bay structure had prestressed concrete beam sections, many of which were displaced laterally to the north as much as 6 feet. Thirty-two of the sections were overlapped in the median indicating that the south structure had been lifted and deposited on the companion north structure. Mid-span diaphragm rods were broken and many diaphragms were cracked and spalled. These units are reported to have weighed 175 tons.

The report concludes that buoyancy was a major factor due to the entrapment of air between end diaphragms and the concrete girders. In the Bay St. Louis Bridge, it was estimated that 520 cubic feet of air resulted in 71,000 lbs. of buoyancy and that at Biloxi Bay there was 197,000 lbs. of buoyancy per span (these values were not verified). The report indicated that the tendency to float could have been negated by venting diaphragms below the deck level. A difference in response of the two structures due to the position of the fascia girder was also noted. At Bay St. Louis the fascia girder was set inward from the edge of the structure, i.e., there was an overhang, and it was believed that this allowed vertical wave forces to crack the overhangs. The fascia beam on the Biloxi Bay Bridge was located beneath the railing, and the curb and rail sections were reported as not having experienced the large upward forces that damaged those on the Bay St. Louis Bridge.

South Carolina Department of Transportation (M. R. Sanders) – 9/28/06

Title: "Development of Hydraulic Computer Models to Analyze Tidal and Coastal Stream Hydraulic Conditions at Highway Structures", Phases I, II, III
Authors: E. V. Richardson, et al (Phase I); L. W. Zevenbergen, et al (Phase II); L. W. Zevenbergen, et al (Phase III)
Date: September 1994 (Phase I); December 1997 (Phase II); March 2002 (Phase III)

Comments: This is a series of three papers intended to provide guidance to engineers performing hydraulic and scour analysis at bridges crossing tidally influenced waterways. The first report, Phase I, identifies through a literature review a list of 21 hydraulic models that are then reduced through a screening process to four (two 1-dimensional models and twp 2-dimensional models) that are tested, compared, and recommended for use for hydraulic analysis of bridges crossing tidally influenced waterways. The report also provides a discussion of storm surge boundary conditions and sources of storm surge data such as FEMA, NOAA, and USACE. Appendices provide a review of each of the 21 models including advantages and disadvantages for the application to hydraulic analysis of bridges crossing tidally influenced waterways. The second report, Phase II, tests two (UNET and FESWMS) of the four models recommended in Phase I, presents a method for computing storm surge hydrographs for model boundary conditions, and describes a users manual on tidal modeling for bridge applications to supplement existing model users manuals. Appendices present maps and tables of peak storm surge elevations along the east and gulf coasts and within Chesapeake Bay. The third report, Phase III, adds enhancements to the two selected models UNET and FESWMS, test the enhanced models using case studies, introduces an alternative to the synthetic surge hydrograph developed in Phase II, compares hurricane peak stage frequency with hurricane category, describes guidance on the inclusion of upland runoff into storm surge simulations, presents recommended methods for estimating wave heights at bridges, and guidance for incorporating wind into tidal simulations. Although the reports focus on tidal hydraulics for estimating scour, there are two topics in the third report Phase III that pertain to this project — estimating wave heights and wind.

University of Florida

Title: Progress Report - "Development of Probabilistic Bridge Design Procedures for Wave Forces"
Authors: D. Max Sheppard
Date: September 2006

Comments: **DRM**

Virginia Department of Transportation

Title: Hurricane Damage Assessment for Major Structures in Hampton Roads
Authors: Suffolk District Structure and Bridge Section
Date: July 30, 1997

Comments: The report was developed to assess the type of damage that might occur for hurricanes of Category 1 through 4 for bridge structures, approach roadways, and incidental structures in the Suffolk District of the Virginia Department of Transportation. Seven bridge structures were studied. The force effects resulting from wind, wave, stream flow, buoyancy and flooding, and collision damage were considered. It was determined based on numerous engineering assumptions that sign, signal and lighting structures were adequate for wind speeds from 86 to 120 mph, depending on the type of structure. Major structural failures of this type of engineered construction was not anticipated. The analysis of the bridge structures indicated that wave loads would far exceed forces exerted by winds.

A detailed analysis was made of Pier 23 of the I-64 High Rise Bridge over the southern branch of the Elizabeth River using wave impact forces based on a Category 4 hurricane. The results indicated that these loads would exceed the design stresses, but not to the extent that failure would be anticipated. However, resistance of bridge structures to wave impact are four of the most exposed structures indicated a probable failure for a Category 3 or 4 hurricanes for two of the four bridges and a probable failure in a Category 4 hurricane for a third bridge. These results are based on use of the COE Shore Protection Manual's empirical formulas for wave height and period, nondimensional plots for force coefficients for cylindrical piles, corrected for overtopping forces for walls and for stream flow in a channel below the superstructure. Figures were provided showing the expected surge elevation for Categories 1 through 4 related to mean high tide and low high tide for these four bridges. The source of the surge height did not appear to be listed.

The response of bridges to stream flow was also evaluated and it as determined that the four most exposed structures were adequate for stream flows far in excess of that which was projected for the James River Basin.

The subject of allision between a vessel and a structure was also evaluated, but since it was not possible to predict the joint probability of hurricane and vessel collision, only a general appraisal of damage was provided.

A bullet list of general types of damage to be expected from the four hurricane categories was also provided. Recommendations were made regarding the protection of facilities that concentrated on several movable bridges. These involve protection for diesel-generated oil storages and electrical transformers.

General

Title: "An Investigation of Wave Forces for Design of a Cruise Ship Pier Bridgetown, Barbados" presented at "Wave Kinematics, Dynamics and Loads on Structures", Symposium, Huston, Texas
Authors: Mark Mattila, et al
Date: 1998

Comments: Paper discusses the design of a pile supported pier in the Barbados. Through literature review, numerical analysis and physical modeling the effects and forces of waves were assessed for the design of the structure. Of particular significance for the design was the wave induced vertical pressure on the platform and access trestle decks. This pressure consisted of three components: (i) an initial large upward impact pressure of short duration acting over a small area followed by (ii) a slowly varying uplift pressure and (iii) a subsequent slowly varying suction pressure, the latter two which act over a broad area. The vertical wave forces on the deck were an order of magnitude larger than horizontal wav forces on the deck or supporting piles. It was also clearly apparent that the local wave pressures greatly exceeded those averaged over a larger area of the deck.

Title: "An Investigation of Load Factors for Flood and Combined Wind and Flood"
Authors: David L. Kriebel and Kishor C. Mehta
Date: June 1998

Comments: This report discusses the development of load factors for flood and combined wind and flood loads for incorporation into ASCE – 7. The project utilized numerical simulation of hurricane storms striking coastal areas due to the fact that recorded hurricane data at any coastal location are insufficient to yield good statistical results. Eleven sites were chosen from New England to the Texas-Mexico border to develop wind speed and hurricane storm tide data for statistical analysis. Monte Carlo simulation techniques were used to simulate 999 hurricanes at each of the eleven sites. Results of these simulations include the maximum wind speed and wind direction at the shoreline for each storm, with the associated storm tide levels (storm surge plus astronomical tide) as well as the peak storm tide and associated wind speed and direction. Combined load factors were determined for two scenarios, the structure subjected to: a) Maximum wind and an associated (correlated) storm tide; and b) Maximum flood and an associated (correlated) wind. A target reliability index of 2.5 was used throughout this work.

Title: “Analysis of Rubble Mound Breakwaters”, Supplement to Bulletin No. 78/79 (1992)
Authors: PIANC, Report of Working Group 12
Date: 1992

Comments: Working Group 12 was set up to consider the analysis of rubble mound breakwaters with a view to achieving a better understanding of safety aspects. The working group decided to develop the practical application of risk analysis in the design of rubble mound breakwaters by using partial coefficients. Six subgroups, A-F, were established to carry out different aspects of the study. This main report summarizes the subgroup reports and presents the overall view of the results of the working group.

Title: “Analysis of Rubble Mound Breakwaters”, Supplement to Bulletin No. 78/79
Authors: PIANC, Report of Working Group 12 – Sub-Group A – Formulae for Rubble Mound Breakwater Failure Modes
Date: 1993

Comments: The objective of this subgroup was to identify functional relationships between the main environmental / structural parameters, and the structure response for as many types of rubble mound breakwaters as possible.

Title: “Analysis of Rubble Mound Breakwaters”, Supplement to Bulletin No. 78/79
Authors: PIANC, Report of Working Group 12 – Sub-Group B – Uncertainty Related to Environmental Data and Estimated Extreme Events
Date: 1992

Comments: The objective of this subgroup was to evaluate uncertainty related to environmental data and uncertainty related to extreme events. This report discusses uncertainties related to parameters in short-term sea state statistics (extreme value distribution for individual wave heights in a record, sampling variability, variability due to different algorithms, instrument response and location of measurement, imperfection of numerical hindcast models and quality of wind input, shallow water wave propagation models). Estimates on over uncertainties are presented along with their coefficient of variation. Uncertainties related to the estimation of extreme events are also discussed with respect to return period, encounter probability, and extreme value analysis of wave heights (long term).

Title: “Analysis of Rubble Mound Breakwaters”, Supplement to Bulletin No. 78/79
Authors: PIANC, Report of Working Group 12 – Sub-Group C – Risk Analysis in Breakwater Design
Date: 1989

Comments: This report provides an overview of risk analysis calculation procedures at three different levels and an example of calculation and comparison of available methods at the three levels.

Title: “Analysis of Rubble Mound Breakwaters”, Supplement to Bulletin No. 78/79
Authors: PIANC, Report of Working Group 12 – Sub-Group E – Investigation of Selected Cases
Date: 1992

Comments: This subgroup was tasked to make a selection of existing breakwater structures of different types for which reliable data on environmental conditions, structural parameters and structural response would be readily available or could be made available. Using the findings of Subgroup F, in which a formulation for partial coefficients was developed, the selected cases were analyzed by applying partial coefficients and comparing the results with deterministic design and actual performance.

Title: “Analysis of Rubble Mound Breakwaters”, Supplement to Bulletin No. 78/79
Authors: PIANC, Report of Working Group 12 – Sub-Group F – Development of a Partial Coefficient System for the Design of Rubble Mound Breakwaters.
Date: 1991

Comments: This subgroup was tasked with proposing safety guidelines for rubble mound breakwaters including evaluation of the safety levels inherent in conventionally designed existing structures. The work of the other subgroups provides the basis for the development of guidelines presented in this report. These safety guidelines are based on the partial coefficient system which is simple to apply and well known from other fields of civil engineering. Examples of the use of the guidelines are provided and compared to conventional design practice.

Title: “Design of Jetty Decks for Extreme Vertical Wave Loads”
Authors: Jeroen Overbeek
Date: 2001

Comments: This paper discusses the use of published results of earlier investigations into the phenomenon of wave slamming and wave entrapment under decks and the development of a practical design approach that was used in the design of two jetty platforms. Both structures were subsequently hit by hurricane induced waves and survived with only minor structural damage. Two formulae are presented; one for the impact pressure assumed over the first meter of the wave front and the second for the slow varying pressure, assumed over the immersed part of the deck. Of note is that the jetty was designed with beams in only one direction to avoid air entrapment in a beam grid. Also, the deck has gaps in the transverse direction which are covered with timber, T-shaped strips that have no fixing and can blow-out during wave attack.

Title: "Design Wave Height Related to Structure Lifetime", Proceedings, International Conference Coastal Engineering, Orlando, Florida
Authors: Z. Liu and H. F. Burcharth
Date: 1996

Comments: This paper discusses the following sources of uncertainty that contribute to the uncertainty of the design wave height. 1) Statistical vagrancy of nature, i.e. the extreme wave height is a random variable; 2) Sample variability due to limited sample size, and 3) Error related to measurement, visual observation or hindcast.

Title: "Estimating Uncertainty in the Extreme Value Analysis of Data Generated by a Hurricane Simulation Model"
Authors: Stuart Coles
Date: 2003

Comments: This paper proposes a technique for uncertainty quantification in the inference of extreme return levels base on simulated hurricane series. Simulated series of 999 hurricane events were generated for each of around 50 mileposts along the eastern United States coastline. The aim of this paper is to calculate reasonable approximations to the uncertainty of extreme value estimates due to the hurricane model itself.

Title: "Hurricane Impact Analysis and Development of Design Criteria for the I-10 Bridges over Escambia Bay, Escambia County, Florida"
Authors: Ocean Engineering Associates, Inc.
Date: January 2005

Comments: This report describes Ocean Engineering Associates, Inc.'s investigation into the failure of I-10 over Escambia Bay during the passing of Hurricane Ivan. The study first presents hindcast wind and pressure fields for Hurricane Ivan. By comparing these data with a statistical analysis of wind speed and pressure from the NOAA HURDAT database for Hurricanes striking the coast near Escambia Bay, the investigators determined the landfall return period of Hurricane Ivan (approximately 200-years). The hindcast wind and pressure fields from Hurricane Ivan also provided the boundary conditions to drive two-dimensional wave (WAM and SWAN) and a tidal circulation (ADCIRC) models. High water marks recorded near Escambia Bay and NDBC wave buoy in the Gulf of Mexico provided calibration data for the ADCIRC and WAM models. The report presents a plot of the peak water surface elevation and wave crest envelope, low member elevations of the bridge, and damage reports along the bridge. In the figure most of the damage occurs when the wave crest envelope exceeds the low member elevation. This provides good evidence of the importance of raising the bridge low member elevation above the anticipated design wave crest elevation. With that in mind the authors develop design parameters for the replacement bridge. To do this they scaled the Hurricane Ivan wind and pressure fields to create the 100- and 500-year return period wind and pressure fields to drive the models for designing the replacement bridge.

Title: "Introduction to Coastal Engineering and Management"
Authors: J. William Kamphuis
Date: 2002

Comments: This textbook discusses the basic concepts of the coastal engineering and management field. It includes a chapter on the risk based analysis of coastal structures including how to determine the probability of failure for a design condition during the lifetime of a project.

Title: "Lessons Learned from Hurricane Katrina Storm Surge on Engineered Structures"
Authors: Ian N. Robertson, et al
Date: ???

Comments: The document is a report of a survey of damage to a variety civil engineering structures along the Gulf Coast primarily in Mississippi. It covers observed damage to residential construction, engineered buildings, bridges, and roadways. Uplift characteristics on bridges are divided into two pieces: hydrostatic uplift or buoyancy, including the effect of entrapped air below the structure, and hydrodynamic uplift which is a result of wave action. The report identifies hydrostatic uplift (buoyancy) as a major factor in the damage to and loss of many structures. The universal gas law is used in associated calculations to adjust the volume of the entrapped air for the pressure head above the entrapped air. Buoyancy calculations are provided for one structure and a table is provided for buoyancy calculations for a total of seven structures. That table is reproduced below.

Table 1: Buoyancy calculations for bridges investigated in this study

Bridge	Concrete volume m^3/m (ft^3/ft)	Self-weight* kN/m (lb/ft)	Air volume m^3/m (ft^3/ft)	Buoyancy** kN/m (lb/ft)	Net self-weight kN/m (lb/ft)	Percent of self-weight (%)
US 90-Biloxi Bay†	3.94 (42.4)	92.76 (6360)	5.27 (56.7)	91.55 (6277)	1.21 (83)	1.3
Old Bridge-Biloxi Bay†	2.42 (26.1)	56.98 (3915)	2.22 (23.9)	45.56 (3115)	11.52 (800)	20.2
Railroad Bridge-Biloxi Bay	3.60 (38.8)	84.76 (5820)	1.50 (16.1)	51.28 (3514)	33.48 (2306)	39.5
US 90-Bay St. Louis†	3.91 (42.1)	92.06 (6315)	3.21 (34.5)	70.57 (4836)	21.49 (1479)	23.3
Railroad Bridge- Bay St. Louis‡	3.05 (32.8)	71.81 (4920)	2.46 (26.5)	55.37 (3794)	16.44 (1126)	22.9
US-90 approach span Pass Christian†	7.55 (81.3)	177.8 (12195)	17.5 (188.2)	248.4 (17019)	-70.6 (-4824)	-39.7
I-10 Onramp-Mobile†	2.97 (32.0)	69.93 (4800)	2.44 (26.28)	50.46 (3458)	19.47 (1342)	27.8

*Self-weight based on 2400 kg/m^3 (150 lb/ft^3) for reinforced concrete
**Buoyancy based on 1025 kg/m^3 (64 lb/ft^3) density of seawater
†Assuming guardrails are not submerged
‡Assuming box girder stems and base slab are 12 inches thick

The data in the table above was said to not include the weight of bulk heads, diaphragms or other elements of the cross-section other than beams and decks. The general lack of vertical uplift capacity in bridge bearings is cited indicating that once the self-weight was overcome by the combination of buoyancy and wave forces, bridge superstructures were able to float free and move off their supports.

The railroad bridge from Biloxi to Ocean Springs is cited as an example of a bridge that had very good performance. As in the case of the Lake Pontchartrain Bridge, the tracks, ties and ballast were removed, but the superstructure remained intact. The cross-section for this bridge had very little room for entrapped air. There were four closely spaced prestressed concrete I-beams supporting the railway deck with substantial overhangs compared to many railroad bridges. Additionally, there were 15-inch high shear blocks on both ends of each pier cap which provided much greater restraint that was true in most highway bridges. This bridge is identified in the table reproduced above. It is reported to have a net dead weight when submerged equal to almost 40% of the in-air-weight.

Precast parking garage floor systems involving highly optimized double-T construction generally did poorly. Of significance is the combination of entrapped air and wave action, similar to that found in bridges. In some cases, the double-T floor systems were found to have broken in negative bending from the combination of these loads, particularly where they were somewhat restrained by spandrel beam connections.
The report indicates that extensive scour was observed around bridge abutments, piers and building foundations, as well as highway structures. It was not clear if the piers were wharf-type piers or bridge piers. Two mechanisms of scour were identified: 1) sediment transport due to flowing water and debris, and 2) liquefaction.
With respect of impact from floating debris, this report recommends use of the ASCE/SCI7-05 formula based on impulse momentum considerations. The calculation of the force requires a time period of the impulse and ASCE recommends .03 seconds, although the FEMA Coastal Construction Manual is reported as recommending .3 seconds. The formula is $F = \frac{\pi W V}{2 g \Delta t}$ where: W = weight, V = impact velocity, g = gravity, Δt = impact duration. In the conclusions and recommendations, it is suggested that bridge segments of low lying bridges be restrained against uplift and provided with shear keys to resist all lateral loads ignoring any friction due to gravity loads.

Title: "Marketing Uncertainty" in proceedings COPEDEC '99, Capetown
Authors: J. William Kamphuis
Date: 1997

Comments: This paper looks at some realities of coastal engineering and describes the uncertainties inherent in basic coastal data and in derived data such as sediment transport rates. Physical and numerical modeling are shown to contain their own uncertainties. When combined with uncertainties in the data, the paper states that models produce essentially qualitative results and interpretation of model results requires coastal engineering expertise and must be done very carefully.

Title: "Tsunami and Storm Surge Hydrodynamic Loads on Coastal Bridge Structures" presented at 21st US-Japan Bridge Engineering Workshop, Tsukuba, Japan
Authors: Solomon Yim
Date: 2005

Comments: This paper briefly describes the physical phenomena of tsunami and hurricane induced water elevation change and inundation at coastal areas; discusses tsunami and storm surge hydrodynamic loads on coastal bridge structures; and summarizes the physical experimental facilities at the Oregon State Wave Research Laboratory and selected models at OSU for coupled fluid-structure interaction modeling, testing and simulation; and provides a discussion on the development of comprehensive experimental studies and some challenges in experimental and numerical simulations of large-scale fluid-structure interaction with applications to coastal bridge structures.

Title: NCHRP Report 489, "Design of Highway Bridges for Extreme Events"
Authors: M. Ghosn, F. Moses and J. Wang
Date: 2003

Comments: This report contains the findings of a study to develop a design procedure for application of extreme event loads and extreme event loading combinations to highway bridges. The report describes the research effort leading to the recommended procedure and discusses the application of reliability analysis to bridge design.

Title: "New Guidance for Wave Forces on Jetties in Exposed Locations"
Authors: K. J. McConnell, et al
Date: 2003
Title: "Experimental Study of Wave-in-Deck Loads on Exposed Jetties"
Authors: Giovanni Cuomo, et al
Date: 2004

Title: "Physical Model Studies of Wave-Induced Loading on Exposed Jetties: Towards New Prediction Formulae"
Authors: Matteo Tirindelli, et al
Date: 2003
Title: "Exposed Jetties: Inconsistencies and Gaps in Design Methods for Wave-Induce Forces"
Authors: Tirindelli, et al
Date: 2002
Title: "Piers, Jetties and Related Structures Exposed to Waves – Guidelines for Hydraulic Loadings"
Authors: McConnell, et al (HR Wallingford)
Date: 2004

Comments: The first four titles are papers that present the background for the final title, which is a comprehensive report discussing guidelines for the design of nearshore structures exposed to waves. The project to develop these guidelines was undertaken by HR Wallingford with an extensive Steering Committee that directed the technical content of the project. The report discusses the various aspects of design such as the hydraulic and related loads; acceptable risk issues, approaches to design; and determining design wave conditions. The results of physical model tests are presented and compared to other previously published design equations. New design equations for horizontal and vertical quasi-static and impact forces are then presented based on the physical model test results.

Title: "New Prediction Method for Wave-in-Deck Loads on Exposed Piers/Jetties" – PowerPoint presentation
Authors: William Allsop, et al
Date: 2006

Comments: Introduces a new method of predicting forces using the Wallingford procedure but with a linear equation for the dimensionless force F^*_{qs}. This potentially supplants the exponential equation in earlier papers.

Title: "New Prediction Method for Wave-in-Deck Loads on Exposed Piers/Jetties" – Full document
Authors: William Allsop, et al
Date: 2006
Title: "Wave-in-Deck Loads on Exposed Jetties"
Authors: Giovanni Cuomo, Matteo Tirindelli, William Allsop
Date: 2006

Comments: These papers are further exposition of similarly titled Powerpoint presentation above and more fully document data analysis and contains work example of applying the new method. Presents revised tables of coefficients based on a new linear equation with different dimensionless parameters that presented in earlier papers. These new equations result from a re-analysis of the physical model tests based on a wavelet transform to filter out corruption in some of the recorded signals from dynamic responses of the measuring instruments.

Title: "Random Seas and Design in Maritime Structures"
Authors: Goda
Date: 2000

Comments: This textbook provides design tools to deal with random seas and discusses random wave theory. It includes a chapter on wave loads on vertical breakwaters and another chapter on the statistical analysis of extreme waves.

Title: "Recommended Practice for Planning, Design and Constructing Fixed Offshore Platforms – Load and Resistance Factor Design", First Edition, 224 pages
Authors: American Petroleum Institute
http://www.techstreet.com/cgi-bin/detail?product_id=1819
Date: July 1, 1993

Comments: This specification contains the engineering design principles and good practices that have been the basis of the API RP2A working strength design (WSD) recommended practice. These LRFD provisions have been developed from the WSD provisions using reliability based calibration. Load factors and load combinations are presented for extreme and operating wind, wave and current loads. Methods for calculating these loads are discussed extensively. Also, it contains an interesting commentary on three possibilities for defining the 100-year combined extreme wind, wave and current load. They are: 1) the 100-year return period wave height with “associated” wind and “associated” current; 2) Any “reasonable combination of wind speed, wave height, and current speed that results in the 100-year return period combined platform load; and 3) 100-year return period wave height combined with the 100-year return period wind speed and the 100-year return period current speed. After considering the technical and practical merits of each of the three definitions, Definition 1 was selected.

Title: “Synthesis of Wave Load Design Methods for Coastal Bridges” (???)
Authors: Francisco Aguiniga, et al
Date: August 2006

Comments: This report presents an introduction to weather and hurricanes, fundamental concepts of water waves, and a compilation of available source of information that contain information related to forces produced by waves acting on engineering structures such as sea walls, suspended walls and bridge decks. A section on the parameters most relevant to the design of bridge superstructures against hurricane waves is included. It also provides a synthesis of data found in several historical databases and databases maintained regularly by government organizations and research laboratories.

Title: “Theoretical Analysis of Wave Impact Forces on Platform Deck Structures”
Authors: P. Kaplan, et al
Date: 1995

Comments: This report presents an update to the mathematical wave force model presented by Kaplan in his 1992 “Wave Impact Forces on Offshore Structures: Re-examination and New Interpretations” paper. In this paper, Kaplan focuses on the flat plate deck elements and adds terms to account for different superstructure aspect ratios. The paper presents new equations for forces in both the vertical and horizontal plan, which include the aspect ratio terms. The equations, based on the classical Morison equation, include inertia, added mass, and drag terms. The author also discusses the importance of accurately predicting the wave kinematics used to drive the mathematical model. He includes a discussion of velocity blocking and shielding, which accounts for processes during the wave structure interaction such as the separation of the wave as it encounters the deck. There is a description of experimental data used for comparisons that includes a discussion of the difficulty of measuring impact forces. Comparisons between the theory presented in the paper and experimental data show good overall agreement. The authors attribute differences to the variability associated with the experimental measurements of waves and forces.

Title: "Unified Facilities Criteria – Design: Piers and Wharves"
Authors: Department of Defense
Date: July 2005

Comments: This contains descriptions and design criteria for pier and wharf construction, including subsidiary, contiguous and auxiliary structures. Load factors and load combinations are provided for such structures. They are primarily based on ASCE 7-02.

Title: "USCOE Coastal Engineering Manual – Chapter 6 – Reliability Based Design of Coastal Structures"
Authors: USCOE
Date: 2003

Comments: Advance probabilistic methods are presented where the uncertainties of the involved loading and strength variables for coastal structures are considered. The partial coefficient system is described which takes into account the stochastic properties of the variables and makes it possible to design a structure for a specific failure probability level.

Title: "USCOE TR CHL-97-9 Reliability Assessment of Breakwaters"
Authors: Jeffrey Melby
Date: 1997
Comments: This report is essentially the same as Chapter 6 of the CEM described above.

Title: "Wave Forces on Bridge Decks"
Authors: Scott L. Douglass, et al
Date: April 2006

Comments: This report is a synthesis of existing knowledge related to wave forces on highway bridge decks. The literature review includes research from the transportation engineering community as well as some research on related issues in the coastal and ocean engineering community. Also, included are results of some original, focused research on the topic including a new, recommended approach for estimating these forces. The study recommends FHWA apply the new method as interim guidance in this area. The study states that the new method does a good job of explaining damage to bridges in Hurricanes Katrina and Ivan. Estimated wave loads are sufficient to overcome the weight and connection resistance for the spans at lower elevations that failed (moved) at three bridges; the I-10 Bridge across Escambia Bay, FL; the I-10 on-ramp near Mobile, AL; and the U.S. 90 Bridge across Biloxi Bay, MS. Estimated wave loads are not sufficient, however, to overcome the weight and connection resistance for the spans at higher elevations that did not fail.

Title: "Wave Forces on Causeway-Type Coastal Bridges"
Authors: Keith H. Denson
Date: October 1978

Comments: This report presents the results of an experimental study on a model bridge, patterned very closely after the Bay St. Louis Bridge which was severely damaged by Hurricane Camille in 1969 as reported in another reference reviewed in this project. The effective wave forces were determined separately on the seaward and landward cross-sections. This bridge was a dual cross-section separated by a 1 inch construction joint. Data on lift, drag and rolling moments are reported in a dimensionless form. It is reported that damage was apparently caused by wave action overcoming the dead weight of the concrete structure resulting in failures of anchors between the girders and pile caps. It is stated that this type of damage was most likely produced by the rolling moments.

A 1:24 scale model made of wood and Plexiglas was used. The end diaphragms used in the prototype were not included in this model. It was attached to an instrumentation platform that used electrical resistance strain gages to separately measure lift, drag and rolling moments. Output from strain gages was recorded on an oscillograph as a function of time. It was not clear whether any effort was made to separate slamming forces from other types of forces nor was the number of readings per second reported.

The wave type was trochoidal with a period of 3 seconds. Wave direction was perpendicular to the bridge longitudinal axis. Lift, drag and rolling moment were measured for a minimum of five independent waves for each fixed value of bridge clearance, water depth and wave height. The following nomenclature is used: h = height of the top of the slab from the bay bottom, w = out-to-out width of the cross-section, D = still water depth, H = wave height, γ = specific weight of water, F_L = lift force per unit length, F_D = drag force per unit length, M = rolling moment. Each of the variables h/W, h/D and H/D were varied in five increments for both the seaward and landward cross-sections. Results are reported in terms of moment, lift and drag for h/W = 0.636, 0.572, 0.509, 0.445 and 0.381. Each graph contains five curves for different values of h/D. The figures are for the forces and moments produced only by the fluid motion. The effects of the dead weight of the structure in buoyancy at storm water elevations would still have to be accounted for separately.

The report concludes that the type of damage was similar to that caused by Hurricane Camille. It is concluded that relatively small anchorage forces would have prevented the types of failure observed at both Bay St. Louis and Biloxi Bay Bridge. The report contains caveats that significantly different geometries would give different results. The results of the study were scaled at prototype dimensions.

Title: "Wave Forces on Causeway-Type Coastal Bridges: Effects of Angle of Wave Incidence and Cross-Section Shape"
Authors: Keith H. Denson
Date: 1980

Comments: This report presents an extension of the study Professor Denson reported in 1978. In this study two different types of bridges are considered: 1-24 scale model of a beam-slab bridge patterned after the Bay St. Louis Bridge and a trapezoidal box girder bridge with 10% superelevation to seaward. Each of the bridges are dual roadway, but since the trapezoidal box girder bridge has significant overhangs, the distance between the beam elements are much different. This study involves waves impinging on the structure from different angles and, therefore, it is a three-dimensional presentation rather than the two-dimensional presentation used in the earlier study.

In the introduction the author indicates that in 1978 a questionnaire was sent to 22 states. Twenty states replied and six indicated that they had observed some damage on coastal bridges and 17 reported bridges located where they could be subject to wave action. Between 1978 when the questionnaire was sent and 1980 when the report was finished, there were two additional structures lost to wave action, the Hood Canal floating bridge in Washington and the Dolphin Island Causeway in Alabama. The Dolphin Island Causeway Bridge was destroyed by Hurricane Frederick in 1979. It was a two-lane bridge with simple spans and it was reported as being swept from the supporting pile bents.

In the current study, end diaphragms were added to the model of the Bay St. Louis Bridge used in the 1978 study. It is reported that results for normal incident waves were not exactly the same because of these additional diaphragms. The diaphragms did exist in the prototype.

The models were tested in a three-dimensional wave basin 40 ft. long and 16 ft. wide with waves generated at a constant period of three seconds. Waves were trochoidal as this was reported to be the shallow water wave. For such waves, the author indicated that the wave length and period are not significant and that the wave celerity is a function only of the water depth and the gravitational constant, i.e., $c = \sqrt{gd}$.

The construction of the Bay St. Louis Bridge was reported in the 1978 study; the horizontal box girder model was made of laminated wood planed to the shape of the prototype, scaled, and the piers were hexagonal, cut from solid wood. A photograph of the Bay St. Louis model indicates ten spans of the structure and the pile bents were actually modeled.

Measurements were taken at one section of each model, which consisted of a complete span between centerline of supporting bents or piers. The active span was suspended from a platform which measured three forces and three moments. There were two drag forces measured in the transverse and longitudinal direction, a lift force and yawing moments about the vertical access. Rolling and pitching moments were reported as not interacting with the other variables. As in the 1978 study, electric resistance strain gages were used and they were reported to have been foil backed epoxy gages.

Results were recorded using a six channel switch and balance unit and a two channel oscillograph. As in the case of the 1978 study, there is no indication of the number of readings per second that were taken, although a sample oscillograph trace is presented. It was reported that for the shallow water waves, wave period and wave length do not significantly effect wave forces in moments. (This may not be the same as reported in other references.) Wave lengths varied from 13.9 ft. at a water depth of 8 in. to 17.7 ft at a water depth of 13 in. The wave length scale is the same 1:24 as a structural model. Three forces and three moments were measured in the experimental studies. Forces were normalized by dividing by the unit weight of water and the overall width of the structure cubed; moments were normalized by dividing by the unit weight of water and the width raised to the fourth power. In the 1978 study, these parameters were normalized to the square of the width for force and the cube of the width for moment-cubed.

Water depth, wave height and angle of incident were all varied. This was reported as requiring 250 experimental setups with six sets of force and moment data obtained for each setup. The results are reported in 191 graphs with information for each of the two bridge cross-sections reported in terms of whether they were seaward or landward, the angle of incidence and whether a result was positive or negative. An example is presented showing the use of the graphs to generate prototype forces and moments. The example refers to the Bay St. Louis Bridge with a 45° wave incidence on the seaward pair of lanes. The height of the slab above the bottom of the bay is taken as 22 ft., the width of the structure is 31.46 ft., the depth of the water expected during the hurricane surge is 22 ft., and the maximum wave height anticipated is 8.8 ft. For this set of results in prototype scale, the longitudinal force is reported as +37.9 kips and -47.8 kips. The transverse force is reported as 61.8 kips and -69.8 kips. The vertical force is reported as +335 kips and -213 kips. The moment about the longitudinal axis is + 2510 kip/ft and zero, the moment about the vertical axis is reported as +1820 kip/ft and -1690 kip/ft and the moment about the vertical axis is reported as +207 kip/ft and -116 kip/ft. All these forces are for one complete span of 48 ft. Any additional forces due to dead weight or buoyancy have to be added to these numbers. The results with and without the end diaphragms are also reported for a 90° incident so that the results from the 1978 study can be compared to the 1980 study as shown below.

	Seaward Lanes		Landward Lanes	
	Without Diaphragms	With Diaphragms	Without Diaphragms	With Diaphragms
F_y	10×10^{-3}	70×10^{-3}	9×10^{-3}	35×10^{-3}
F_z	60×10^{-3}	250×10^{-3}	45×10^{-3}	105×10^{-3}
M_x	70×10^{-3}	50×10^{-3}	105×10^{-3}	40×10^{-3}

It is concluded that the end diaphragms increase the transverse drag and lift forces significantly whereas the moments decrease slightly or significantly. This is attributed to the entrapped air caused by the end diaphragms.

Some results are reported for the trapezoidal cross-section. It was reported that for the beam slab bridge a slight but definite trends with increasing angle of incident was evident for the upward vertical force while everything else remained essentially constant and the same was true generally for the box girder bridge. The general conclusion is that there is very little difference in the seaward lane force coefficients between the box girder and the beam slab bridges on the seaward structure. There was more difference in the landward structures and this was attributed to the wider spacing of the box girders.

Title: "Wave Forces on a Horizontal Plate"
International Journal of Offshore and Polar Engineering, Vol. 6, No. 1, March 1996
ISSN No. 1053-5381
Authors: Michael Isaacson and Shankar Bhat
Date: March 1996

Comments: This paper presents an experimental study of the non-breaking wave forces on a thin horizontal plate. The study focused on un-submerged horizontal plates subjected to intermittent submergence. The paper presents two similar theoretical equations based on the work of Paul Kaplan. In both equations terms accounting for, inertia, added mass, change in added mass, drag, and buoyancy are included. The paper describes the experimental setup, a free vibration test to determine the dynamic characteristics of the model and load cells, and the post-processing data filtering. In all the investigators ran 69 tests with varying wave heights, periods, and plate elevations above the still water level. The data was processed through a low pass filter (20 Hz) thus removing much of the slamming force component. The authors point out that their theoretical equations do not cover the impulsive slamming force. The paper examines the influence of plate clearance and wave height and period. Their calibrated equations do a good job of predicting the upward portion of the so called quasi-stationary force (force without slamming) but are not as good for predicting the downward component of the vertical force. The measured results are presented in terms of dimensionless groups as a function of plate clearance height over wave height.

Title: "Wave Impact Forces on Offshore Structures—Reexamination and New Interpretations"
24th Annual Offshore Technology Conference, Houston, TX, May 4-7, 1992 (6814)
Authors: P Kaplan
Date: 1992

Comments: This report presents a mathematical model for estimating the time histories of wave forces on horizontal structures. The mathematical model is an extension of the theory describing ship bottom slamming. The model is first applied to a horizontal cylinder followed by a flat plate deck. In both cases, the mathematical model takes the form of the Morison equation and includes buoyancy, drag, and inertia terms. Comparisons are presented between the mathematical model and measured data for both vertical and horizontal forces. Although the results show good agreement, the author focuses on the model's ability to reproduce the trends. This was due to the author's concerns regarding extraneous force effects in the data used for comparisons. As such, the report includes a long discussion on errors in measuring wave forces. Here the author discusses the need to account for inertial forces due to the acceleration of the structure, improper data sampling rates and data filtering, and errors introduced by the natural frequency of the model.

Bridge Plans Acquired

- U.S. 90 at Biloxi, MS
- U.S. 90 Bay St. Louis, MS
- I-10 across Escambia Bay, FL
- I-10 across Lake Pontchartrain, LA
- New I-10 across Lake Pontchartrain, LA, Excerpts

PowerPoint Presentations from FHWA Force Symposium, December 5-7, 2005

Summary of the Presentations of
The FHWA Wave Force Symposium Held in December 2005

<u>Coastal Storm Events Overview of FHWA Plan of Action</u>
Joe Korlak, P.E. Hydraulic Engineer, Federal highway Administration

This presentation gave a brief description of the FHWA's plan of action to minimize the susceptibility of bridges to damage from coastal storms. The goal of the action plan was defined as: "A proposed set of studies and technology transfer activities to fully achieve a rational approach that addresses wave force, storm surge, and scour vulnerabilities in existing and new structures"

Specific items and the FHWA's lead person for each item were included in the presentation.

<u>Coastal Storm Events, Coastal Bridges and Storm Frequency, Interim Criteria</u>
Joe Korlak, P.E. Hydraulic Engineer, Federal highway Administration

The presentation covered the interim criteria to be followed in determining the suitability of a bridge for the high water flow associated with coastal storms. It also covered the basics of coastal hydraulics and the definition of basic terms used in coastal hydraulics. The main sections of the presentation included:

- Background
  - Intent of Interim Criteria
  - Riverine & Coastal Hydraulics
- Bridge Design Frequencies
  - Typical State DOT practices
  - FHWA regulatory criteria
  - Applicability to coastal flood frequencies
- Bridge Hydraulics
  - Freeboard
  - Scour
  - Bridge approaches & touchdown locations
- Summary & Recommendations

FDOT Sponsored Wave Loading on Bridge Decks Research
Rick Renna, Max Sheppard and Don Slinn

An introduction to the wave parameters and wave forces was provided in this presentation. The presentation also covered the then-ongoing research on bridge decks in the University of Florida, Gainesville.

TXDOT Sponsored Wave Load Design Research - Background, Goals and Current Status
Jon Holt / Hector Estrada / Francisco Aguiniga

This presentation offered an overview of the research funded by Texas DOT. It also presented the provisions for calculating stream and/or wave forces in several design specifications (ASSHTO, Caltrans, ASCE-7) and a review of the required bridge parameters needed to determine the forces.

Design Problem - Extreme Event Design in Coastal Areas - Doyle Drive South of the Golden Gate Bridge
Steve Ng, CALTRANS

This presentation utilized the Doyle Drive South of the Golden Gate Bridge as a case study of the parameters to consider in design.

Hurricane Wave Forces on Bridge Decks
Nobu Kobayashi, University of Delaware
A brief description of contemplated research work was given.

Physical Modeling of Wave Forces
Steve Hughes, US Army COE

The capabilities of the US Army Corps of Engineers' laboratory are covered in this presentation. It also included coverage of similitude requirements and results of testing of a bridge model under different wave conditions. The model was not instrumented and did not have end diaphragms.

Investigations of Wave Forces on Bridge Decks
Billy Edge, Texas A&M

The results of testing an un-instrumented bridge model were included. Preliminary parameters for a contemplated, more comprehensive test were also presented.

How to Determine Surge and Wave Conditions near Coastal Bridges (30 min),
Qin Chen, University of South Alabama

This presentation covered analytical modeling results for surge and waves. It also included a comparison between the analytical results and actual conditions during past storms.

<u>Post Hurricane Katrina & Rita Bridge Issues,</u>
Ray A. Mumphrey, LA DOTD

A series of photos documenting bridge damage in Louisiana is included in this presentation.

<u>Estimating Wave Loads on Pile-Supported Bridges and Piers: Practical Aspects</u>
John Headland, Moffatt & Nichol Consultant

This presentation covered many of the basics of coastal engineering. The presentation was divided into the following areas:

- Introduction
- Design Conditions including design parameters and selection of design return period
- Lateral Loads (drag, pressure calculations)
- Uplift Loads (quasi-static, impulse)
- Importance of Wave Phase/Profile &Varying Water Level
- Structural Application of Loads
- Scour
- Conclusions and Research Needs

The presentation included some photos of models tested as part of studying new structures that were built recently.

<u>Pacific Northwest Experience with Unusual Hydraulic Events</u>
Matt Witecki, WA DOT

Photos of past bridge collapses from extreme hydraulic effects were presented.

<u>A Method for Screening Existing Bridges for Wave Loading Vulnerability</u>
Rick Renna, FL DOT, and Max Sheppard, OEA, Inc.

The outline of the screening process developed for Florida DOT was presented. The process outline is as follows:

- Identify Bridges Susceptible to Wave Impacts During a 100-Year Hurricane Event
- Determine Which Bridges Need Further Analysis
- Using USGS quad maps, NOAA charts, etc. identify bridges with possible problems (large fetch lengths)
- Obtain bathymetry over fetch lengths and in vicinity of bridges (quads, charts, other)
- Obtain 100 year wind speeds (or wind speed of maximum storm of record)
- Using best information available (FEMA, etc.) obtain 100 year storm surge elevations at bridge sites
- Using empirical equations for estimating Significant Wave Height, Hs, and Peak Period, Tp, in USACOE Shore Protection Manual, estimate these parameters at the bridge sites.

- Estimate peak wave crest height (adjust for uncertainty in prediction methods)
- Add crest height to storm surge elevation to obtain peak water elevation
- Obtain bride plans (member elevations)
- Determine bridge importance
    - Evacuation route, Interstate
    - Local (high traffic volume)
    - County (low traffic volume)
    - Rural (alternate route available)
- Determine the relative priority for the bridge based on its vulnerability and importance

Bridges that have high priority are then subjected to further analysis.

<u>Vulnerability Screening and Retrofitting -- Ideas from other Extreme Events</u>
Ian Friedland, FHWA / Jean Louis Briaud, Texas A&M

The results of an analytical study on bridge piles were presented. The study concluded that the piles will be subjected to ratcheting failure that will cause the surrounding soil to fail under the horizontal force acting on the piles. The study also concluded that, for the case studied, piles pull-out is unlikely. The authors recommended using few larger piles rather than many small piles and to install sheet pile wall around the pile group.

<u>Vulnerability Screening and Retrofitting – Ideas from Other Hazards</u>
Ian M. Friedland, FHWA

This presentation included several retrofit options suitable for improving the performance of coastal bridges under wave forces. Most of the ideas were originally developed for seismic retrofit.

<u>HR Wallingford Wave Force Prediction Methodology</u>
William Allsop, HR Wallingford / Giovanni Cuomo, University of Rome

This presentation covered test results of analytical and experimental wave force studies on vertical walls and jetties. The studies included measurements and predictions of the magnitude and duration of the impact forces. The experimental results were compared to analytical predictions made using an exponential formula.

<u>University of South Alabama's Wave Force Synthesis Study</u>
Scott Douglass, University of South Alabama (USA)

The outline of a then-contemplated study on the wave forces on bridge deck was presented. The study has been completed and published in 2006. The presentation also covered the research needed to determine wave forces and the design requirements for coastal transportation facilities.

<u>Field and Experimental Calibration of Wave Forces on Bridge Structures</u>
Solomon Yim, Oregon State University, Ian Robertson and Ron Riggs, University of Hawaii, and Julie Young, Princeton University

This presentation covered the following three areas:

- Photos from an extensive reconnaissance trip that included 110 sites. Several calculations of the buoyancy forces are included in the presentation. Most of these calculations indicated that the superstructure will "float" if air is entrapped between the girders.
- The three-dimensional and two-dimensional wave tank capabilities in Oregon State University.
- Results from finite element analysis of wave forces on a bridge structure.

<u>HR Wallingford UK Wave Force Prediction Methodology</u>
William Allsop, HR Wallingford / Giovanni Cuomo, University of Rome

Samples of static, impact and time history analysis were included in this presentation. The results were compared to experimental results.

<u>A Quick-and-Dirty Estimate of Wave Loads on Bridge Decks by Adapting an Existing Approach from the Oil Industry</u>
Scott Douglass, USA

The application of the approach used in the design of off-shore platforms to bridge is structures was presented. An overview of the wave parameters and a numerical example design were included in the presentation. The author concluded that better methods for estimating wave loads on bridge decks are needed. He also concluded that laboratory and prototype studies are also needed and are justified by cost considerations.

<u>Vertical Wave Forces on an I-10-Escambia Bay Bridge Span Using Kaplan's Method</u>
Max Sheppard, OEA, Inc.

A numerical example of using the Kaplan method to estimate the wave force was presented.

TASK SUBMISSION

TASK 2 – REVIEW, SUMMARIZE, AND AUGMENT LITERATURE

CATALOG OF OBSERVED DAMAGE FROM PAST STORMS

Table of Contents

Introduction

A portion of Task 2 of this project involves cataloging bridge damage experienced in past events. The purpose of cataloging damage is to determine failure modes that have actually occurred in bridges during storm events. Knowledge gained from cataloging damage will be used in later tasks, especially Task 4, which involves the development of retrofit concepts for bridges that are susceptible to coastal storms. The concepts developed will correspond to the observed failure patterns. The following information was gained by examining the sources listed at the beginning of each section. In many cases possible causes of observed damage are listed, these causes should be interpreted as ideas, not as facts.

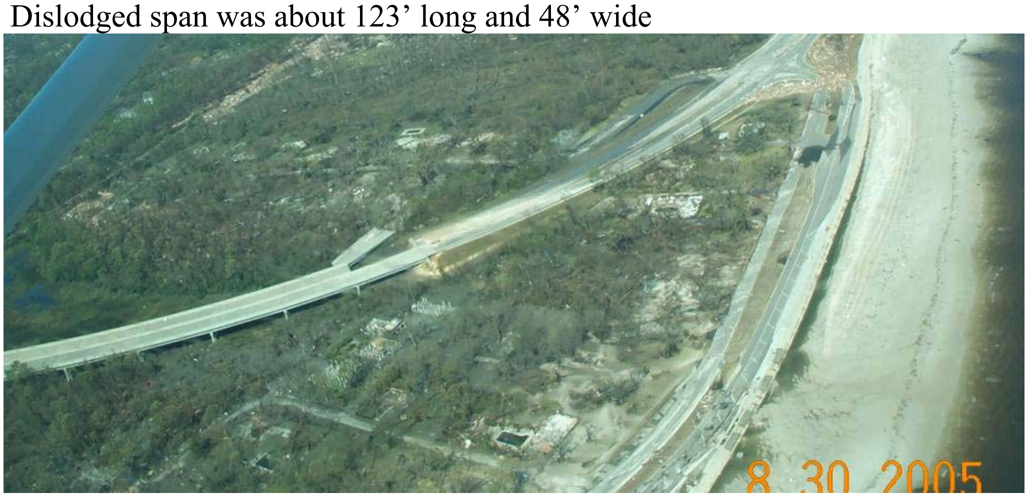
US 90 Henderson Point

No Vertical Restraint (if bond breaker used on dowels)

Information sources: Mississippi DOT pictures and bridge plans

General Information:

- Side by side chorded multi-span prestressed bulb-T girder bridges
- Bridge Drawings dated 1996
- Elastomeric bearing pads at free end
- Full depth doweled diaphragms at fixed ends
- One span of bridge dislodged completely, others shifted
- Dislodged span was where bridge met grade (lowest elevation/most submersion)
- Dislodged span was about 123' long and 48' wide



Information on full depth diaphragms with dowel bars:

- Dowel bars still intact (not sheared)
- Some dowels bent (from beams landing on them)
- Others still upright (at edge, where span moved away from them)
- Dowels in fixed diaphragm are double leg #5's at 12" (See bridge plan sheets 16 to 19, and 23)

- Some States use bond breakers between the steel dowel bars and the diaphragm concrete to enable jacking of the beams for bearing replacement and other maintenance
- No bond breaker details given in plans
- Did dowel bars provide any vertical restraint for this bridge?

- Concrete in end diaphragm is damaged (due to rotation of span about fixed end or dropping of the span?)

Information on entrapped air:

- At least some cavities had drains through the deck that would have allowed air to escape from between the beams (roughly 8” by 3” openings spaced at 8’ 10”)
- Not all spans have the same drain layout
- The dislodged span had deck drains that would vent air from only one of the 5 air-spaces between girders (six beam cross section)
- The dislodged span was sloped longitudinally, so the amount of air trapped between beams and diaphragms is less than would be trapped had the section not been sloped, assuming water rises uniformly
- Change in elevation along the length of the dislodged span was about 2.85’, or about 1’ of elevation change between successive diaphragms
- Higher end has about 5’6” average airspace height, two other air pockets have about 5’0” average air space height. (full depth end diaphragm allows more air to be trapped than intermediate diaphragms, which were not full depth)

Failure Considerations:

- Because there was no vertical restraint (assuming dowels ineffective) it is likely that the superstructure was lifted up and off of the dowels then was pushed laterally by waves, current, and/or wind.

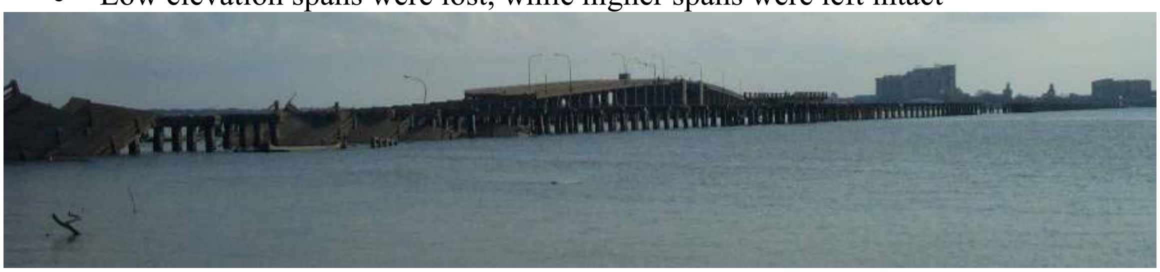
US 90 Biloxi Bay / Biloxi-Ocean Springs

No Vertical Restraint

Information sources: Mississippi DOT Pictures and Bridge Plans

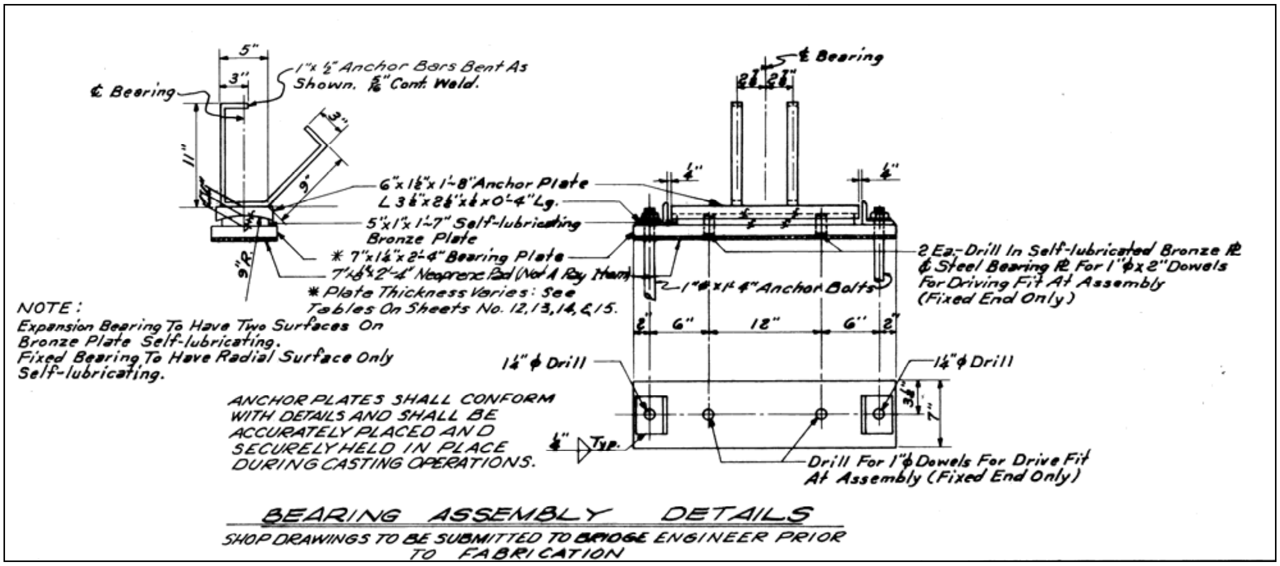
General information:

- Bridge contained concrete girder spans, steel girder spans, and a bascule span
- Bridge plans dated 1959
- Concrete girder spans
    - Prestressed girders
    - 52’ typical span length
    - 33’ 5” typical span width
    - Used on lower elevation spans
    - Majority of bridge was concrete spans
- Steel girder spans
    - Wide flange sections
    - 76’ typical span length
    - Used on higher elevation spans adjacent to bascule
- Low elevation spans were lost, while higher spans were left intact



Information on bearings for 52’ prestressed girder spans:

- Bronze bearings used (Bridge plans, sheet 19)
- No positive connection between beams and pier cap (uplift not resisted)
- Small angles used to restrain transverse movement of bronze bearings (2 ½” high)
- Stepped beam seats, but no shear blocks



Information on entrapped air for 52' concrete spans:

- Deck had 4" inside diameter drains through the deck spaced at 11' center to center.
- These drains would have allowed air to escape from only one of the 5 air-spaces between girders (six girder cross section)
- Solid end and intermediate diaphragms were present (diaphragms terminated 6" from bottom of beam)
- Neglecting weight of pipe railings, light standards, etc, but accounting for diaphragms and concrete rails, assuming air escaped from one airspace due to vents, and assuming the water level to be at the top of the sidewalk, the weight of a typical 52' prestressed span was
  - Un-submerged 347.2k
  - Submerged, air not compressed (ignore ideal gas law) 31.9k
  - Submerged, air compressed using ideal gas law, 1st iteration on h 49.4k
  - Submerged, air compressed using ideal gas law, iterated h 48.4k
  - Where h is the height of the compressed air
- Sections nearly buoyant when submerged in static water

Failure Considerations:

- Conditions that failed the bridge obviously did not involve static water
- Is it possible that the sections were displaced prior to complete submersion? (i.e. wave forces failed the bridge before the above-mentioned buoyancy calculations materialized)
- Likely superstructure was lifted (above the small angles or other features, such as stepped beam seats, that might have provided lateral or longitudinal restraint), then lateral and or longitudinal forces due to waves, tides, and/or wind pushed sections off of piers

US 90 Bay St. Louis

No Vertical Restraint (for concrete spans)

Information sources: Mississippi DOT Pictures, bridge plans, AASHTO slideshow, and pictures taken from paper by Robertson et al

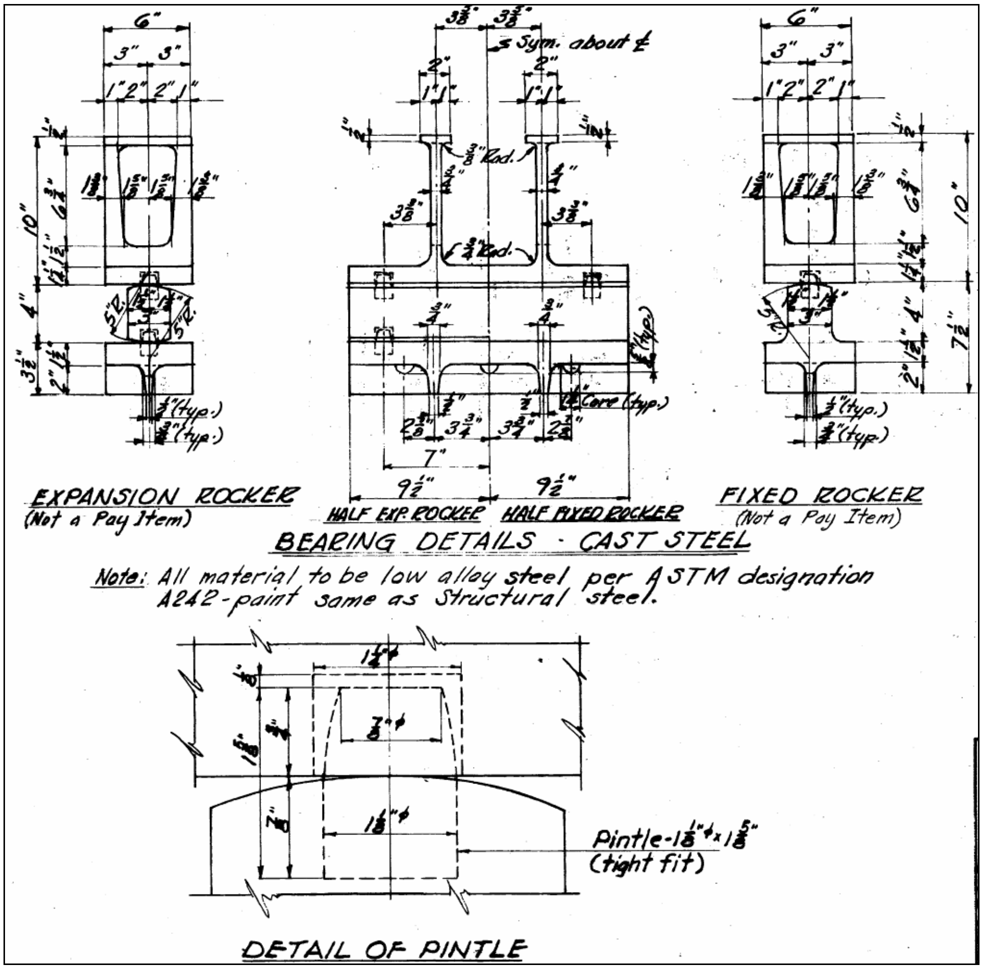
General Information:

- Bridge contained concrete girder spans, steel girder spans, and a bascule span
- Concrete girder spans
  - Reinforced concrete T-Beams
  - 41’ typical span length
  - Majority of bridge was concrete spans
  - Almost all dislodged
- Steel girder spans (two)
  - Wide flange sections
  - 75’ span length
  - Used as approach span on both sides of bascule
  - Concrete deck appears to have been non-composite (no shear studs)
  - One steel span dislodged
  - One steel span remained on piers, but the deck was missing
- Bascule span damaged, but not examined herein
  - Movement of bascule may provide an estimate of wave loads
- Storm surge elevation 24.9’, storm surge plus wave 40.2’ (AASHTO slideshow)
- Dislodged spans found as far as 220’ from their original location (AASHTO slideshow)
- Bridge plans dated 1951

Information on bearings for 41' concrete spans

- Bearings provide no positive connection that would resist uplift
- Bearing details shown below are from sheet 12 of the bridge plans
- Pintles should project ¾" from the rocker or sole plate

- The picture below from Robertson et al shows Pintles that appear to be undamaged



Information on entrapped air for 41’ concrete spans

- 3” diameter drains through deck at 10’ increments
- Drains would permit air to escape from one of the three air-spaces between girders
- Diaphragms were only slightly more than half of the beam depth below the deck
- Comparatively small volume of trapped air

Information on steel spans:

- Bearings of steel spans provided vertical restraint in the form of bolts at both the expansion and fixed ends.
- It appears that the deck was not composite with the girders

Failure Considerations:

- Likely the concrete spans were lifted (above the Pintles or other features that might provide lateral or longitudinal restraint), then lateral and or longitudinal forces due to waves, tides, and/or wind pushed sections off of piers
- Steel spans had connections capable of resisting at least some uplift
- For the steel span that remained on the piers, it is likely that the non-composite slab was lifted away from the beams by wave forces while the beams remained in place because of the provided connections between the beams and pier
- It is unclear if the steel span that was dislodged was knocked off of the pier before or after the non-composite deck was separated from the girders.

Popps Ferry Bridge

Information Source: Mississippi DOT Pictures (no plans available)

General Information:

- Prestressed bulb T girder spans
- Four beam cross section, bents have 4 columns with a cap beam
- It appears that the bridge was designed so that the beams would sit directly over the bent columns
- The spans were shifted, but not lost
- Details of bearings unavailable
- Shifting of the spans caused damage to the pier cap
- Prestressed beams show cracking in the bottom flange near an intermediate external diaphragm (it is unknown how many beams exhibited this behavior or if is was an isolated occurrence)
- Damage to end of beams in area of sole plate
- Failure of deck/sidewalk

Damage resulting from shifted spans

- Cracked pier cap ends
    - Beams came to rest at the end of pier caps
    - The pier cap of pier 52 looks like it sheared where reinforcement started (i.e. beam came to rest with 2” or 3” of bearing on end of pier cap and the cap failed in direct vertical shear along the plane where the first tie/stirrup was)
    - Shallow member beam theory is not applicable to the pier cap overhang due to the large depth to length ratio.

- Pier cap beam cracked between columns
  - Superstructure beams came to rest midway between columns
  - Pier cap beam shows inclined shear crack from face of column
  - Flexure cracks observed directly below some of the shifted superstructure beams.
  - Spalling under superstructure beam, insufficient bearing / bearing on corner???
  - Pier cap beam likely not designed to take superstructure reactions at these locations – beams supposed to sit directly over the columns


51


43

Cracked p/s beam in span / away from end -- Bottom flange at external diaphragm

- The overall context of the picture is not clear
- Other pictures indicate that there are intermediate diaphragms at midspan only
- Positive bending (tension cracks) when span picked up and dropped (or slammed)???
- Does not appear to be crushing from compression (negative bending from uplift or removal of the selfweight that counteracts the prestress force)
- Cracks appear to be located only in close proximity to the diaphragm, this is likely not a coincidence. The diaphragms may create a stiffer section of the structure which attracts load.

Deck/sidewalk failed

- Context of the picture is unclear

- Cantilevered section?
- Failed due to uplift on cantilever?

- Cracked beam end in area of sole plate
    - Due to shortened bearing length after span shifted?
    - Due to dropping of the span?

Failure Considerations:

- Damage to pier cap is due to movement of spans - prevent spans from moving to solve this problem
- Cracked beam near midspan diaphragm – more information needed, may be potential future specification issue

- Failed sidewalk – more information needed, may be a potential future specification issue (see also I-10 Twin Spans)
- Damage to beam near sole plate – prevent spans from moving to solve this problem

I-10 Twin Spans

Broken Connections

Information sources: "Hurricane Katrina, Performance of Transportation Systems" by ASCE, Bridge Plans, Survey of damage performed by Volkert and Associates

General Information:

- Bridge consisted of concrete spans and steels spans
- Concrete spans
    - Precast prestressed monolithic concrete girder/deck units
    - Typical span length 65 ft
    - Used steel and bronze bearings
- Steel Spans
    - Used at higher elevation, not impacted by surge
- Low lying spans impacted by surge while higher elevation spans largely undamaged
- Some spans completely dislodged while others shifted laterally

Information on bearings for concrete spans:

- Steel and bronze bearings used (details from bridge plans not included due to poor image quality)
- Only fixed end exterior girders had vertical restraint (two connections per span)
- Uplift load path from beam to pier is as follows
    - Two straps transfer force from the beam into the bronze plate (photos of damage do not agree with this detail which is shown in the bridge plans)
        - The straps each have two legs, which are hooked and embedded into the beam
        - The straps are attached to the bronze plate through an unknown connection (shop drawings not available)
    - Two bolts transfer the force from the bronze plate into to the pier
        - The anchor bolts are embedded in the pier cap

- Area of steel for bolts and straps are about the same
    - Two 1” diameter bolts = 1.57 square inches
    - Four legs of 1” x 3/8” bar = 1.50 square inches
- Bearings not having vertical restraint had lateral restraint in the form of small angles

Fixed bearing at an exterior beam - Bronze plate still attached to pier
Straps holding the bronze plate to the beam failed while the anchor bolts remain intact

Fixed bearing at an exterior beam - (Bronze plate is still attached to pier, not shown)
Straps holding the bronze plate to the beam failed while the anchor bolts remain intact

Fixed bearing at an exterior beam - (Bronze plate is still attached to pier, not shown)
Straps holding the bronze plate to the beam failed while the anchor bolts remain intact

Fixed bearing at an exterior beam - Bronze plate is still attached to the beam
Straps holding the bronze plate to the beam are intact while the anchor bolts failed

Free end any beam or fixed end interior beam (locations where plate not bolted to pier)
Bronze plates remain attached to the beam through the straps (expected behavior)

Cracks and spalls at beam ends

- Could some of this damage be repairs to damage from previous events?
- Beams banging on pier?
- Shifted beams being supported with insufficient bearing area/surface?
- At locations where the bronze plates were anchored to both the beam and the pier, the beam to plate connection may have caused damage to the concrete?

Cracking/spalling/missing concrete in top of girders/deck

- Context of pictures unclear
- Negative bending--tension?
- Positive bending—compression—picking up and dropping (or slamming) of span?
- Due to railing breaking away (note deck drains in photos, these are present at the edge of the deck)

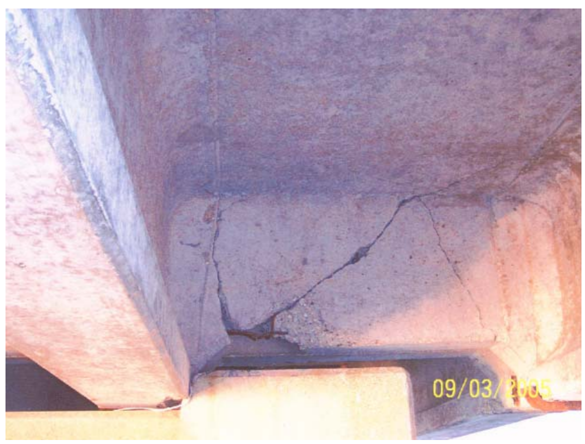
Damage to end of decks, ends of parapets, and ends of curbs

- Longitudinal movement caused banging?
- Spans become misaligned as they move sideways and get pinched between spans on either end (free end moves first)?

45 degree cracks at ends of decks

- Initiated through end diaphragms

- Beams gets caught on a stepped bearing seat as it moves sideways, halts sideways movement, tension transferred through diaphragm, cracks initiate and propagate into deck??
- Span shifts so that fascia beam is no longer supported by pier, weight of unsupported beam is now transferred through diaphragm which develops an inclined shear crack that propagates through the deck??



Spalls on pier, cracking at ends

- Insufficient bearing area when girder shifted??
- Dropping or banging of spans??
- Bending or direct shear on cantilevered bent cap

Missing barrier rail

- Cause of top of girder damage listed above?

Failure considerations:

- Connections inadequate to prevent uplift of spans
    - Vertical restraint provided only at one end of each span
    - In some cases bolts connecting bronze plate to pier broke while in other cases the straps connecting the bronze plate to the beam broke
    - Area of bolts and straps about the same (steel strengths unknown)
    - About 3 square inches of steel to anchor each span
    - Provided area of steel reduced by corrosion?
    - It is unclear if the connections failed in shear, tension, or a combination of shear or tension
- Once spans were lifted (above any existing features that might provide lateral or longitudinal restraint), they moved laterally or longitudinally causing most of the damage pictured above, if spans could be anchored most damage would be avoided.
- 45 degree cracks in ends of decks which were initiated through the diaphragms are a potential future specification issue
    - If superstructure is restrained vertically by some adequate connection and shear blocks are used to restrain the superstructure laterally, will the diaphragms crack and lead to damage of the deck?
    - Are diaphragms an adequate way to anchor the superstructure?
    - Would different reinforcing details solve this problem?
- Damage to barrier rail is a potential future specification issue
    - Use of open rails
    - Use of more robust reinforcement
    - Consideration of uplift on cantilevered section of deck for quantity and development of reinforcement
    - How does the reinforcement used in this bridge compare to current design standards?
    - Are modern designs still susceptible to this type of failure?
    - See also Popps Ferry Bridge
- Cracking/spalling/missing concrete in top of girders/deck is a potential future specification issue
    - What caused this?

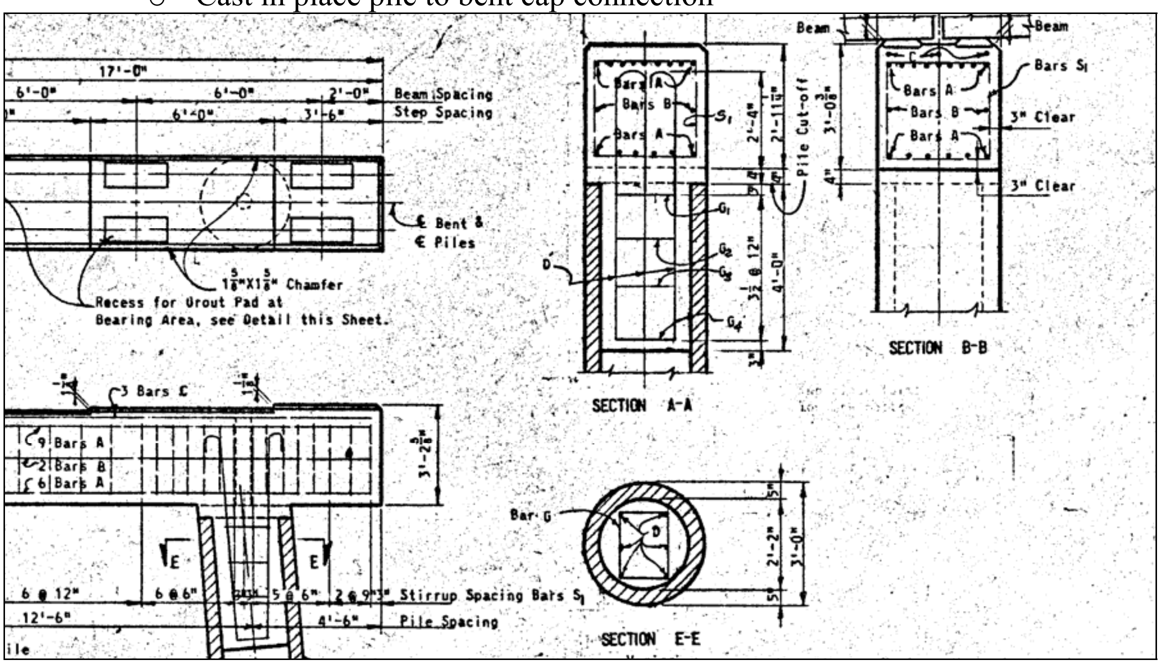
I-10 Escambia Bay

Broken connections, damaged piers

Information sources: Bridge Plans, Hurricane Ivan Damage Inspection by FDOT

General Information:

- Bridge consisted of concrete spans and steels spans
- Concrete spans
    - Precast prestressed monolithic concrete girder/deck units
    - Typical span length 60 ft
    - Used neoprene bearings
- Steel Spans
    - Used over main channel
    - The higher elevation of these spans prevented them from being impacted by waves
- Typical "intermediate" bents were constructed using:
    - Three 36" diameter post-tensioned concrete cylinder piles.
    - Precast bent cap
    - Cast in place pile to bent cap connection



Details of pile to bent cap connection from bridge plans

-
    - Reinforcing shown in pile to bent cap connection was cast into the precast pile cap
    - Pile cap was placed in position above the piles, which were cut at a specified elevation after driving
        - Reinforcing extending into the hollow cylinder pile

- There was a space several inches high between the bottom of the cap and the top of the pile
  - Grout was pumped into the pile so that the top 4' of the pile was filled
  - The short distance between the pile cutoff and the bottom of the bent cap was somehow formed so that grout would fill the space
  - Grout was pumped through a 6" diameter hole in the precast bent cap
  - See pictures below

- Some spans completely dislodged while others shifted laterally and/or longitudinally
- Bents damage included
  - Pile to bent cap connection damaged
  - Piles broken at splices (according to FDOT report)
  - Bents pushed over longitudinally
  - Completely and partially missing bents

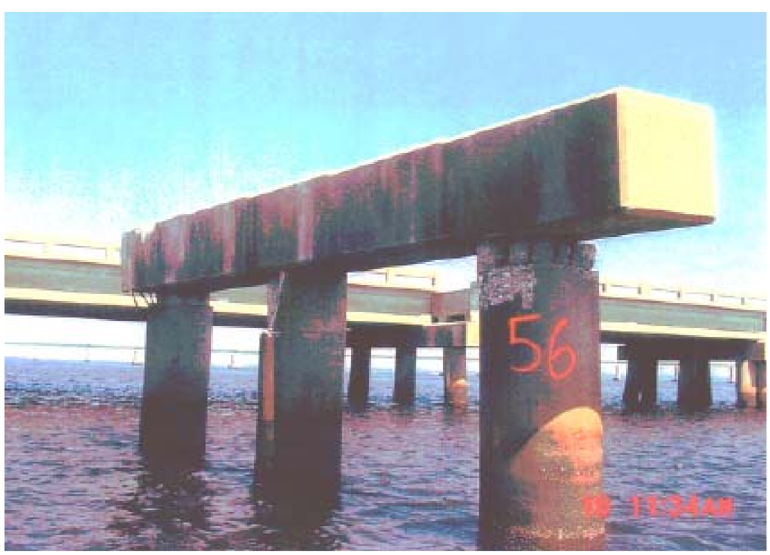
Information on bearings for concrete spans:

- Bridge plans show that spans on flat grade used two bearing configurations
- Interior girders used beams resting directly on neoprene pads
  - No positive restraint at interior bearing locations
- Fascia girders had a steel sole plate that was fastened to the beam and the bent
  - Fascia girder bearings had restraint against uplift, lateral, and longitudinal shifting.
- Although many spans were displaced, spans were anchored sufficiently to cause bent damage at some locations.

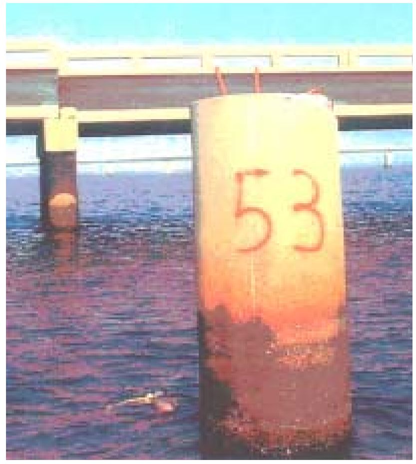
Span displaced laterally and longitudinally

Bent deformed longitudinally

Bent failed longitudinally



Bent with damaged pile to bent cap connection



As built reinforcement?

Failure considerations:

- Bent damage indicates that securing the superstructure to the substructure may lead to destruction of the substructure. This is a serious limitation on retrofits.
- It is unknown if the as-built conditions of the pile to bent cap connections were in agreement with the bridge plans shown above.
- Pile to bent cap connection used on this bridge was also used on other bridges of this era including I-10 Lake Ponchartrain, the Ponchartrain Causeway, and possibly the Pensacola Bay Bridge. These bridges were constructed using precast elements such as monolithic superstructure units and post tensioned Raymond piles. The I-10 Lake Ponchartrain Bridge, the Ponchartrain Causeway, and the Pensacola Bridge are known to have been manufactured at the Mandeville plant on Lake Ponchartrain. It is suspected, but has not been verified, that the Escambia Bay Bridge was also manufactured at this plant.
- I-10 Lake Ponchartrain did not exhibit this type of damage, possibly because:
    - Larger diameter piles were used
    - Bearings provided positive restraint only at one end, so forces actually reacted by the bents may not have been large enough to cause damage
- Broken pile splices that were reported in the FDOT damage inspection are another potential problem with respect to substructure capacity. Pictures of broken splices were not included in the report, possibly because the breaks were underwater. A brochure on Raymond piles from this era (i.e. the 1960's) indicates that piles were cast in segments, then placed end to end and post tensioned. The brochure states that:

    > "A plastic compound having a greater ultimate strength than the concrete is placed on the face of each section. This provides a perfect joint when stressing is complete."

  It is believed, but not verified, that these joints are the "splices" referred to in the FDOT damage inspection.

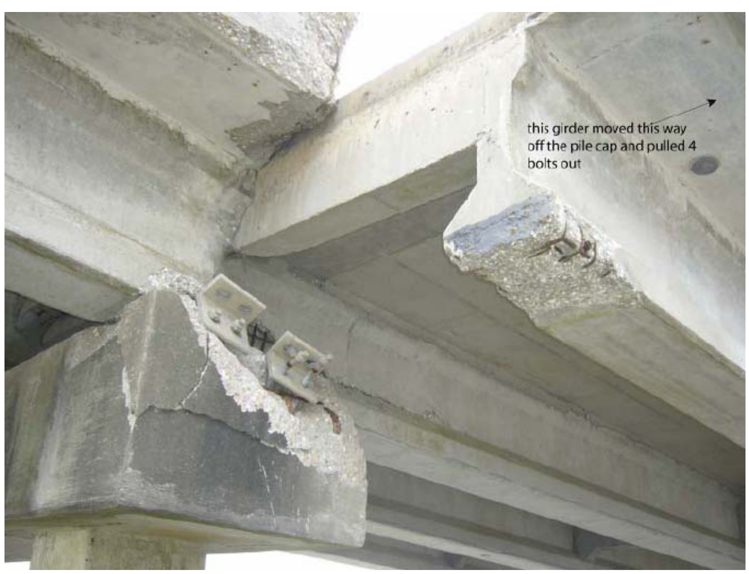
I-10 On-ramp at Midbay Crossing of US-90/98

Broken Connections

Information source: "Hurricane Katrina, Performance of Transportation Systems" by ASCE and pictures in "Wave forces on Bridge Decks" by Douglass

General information

- Prestressed concrete simple spans 45' in length
- Built in 1970's
- Beams anchored using 1" diameter anchor bolts and 6" x 9" x 1" steel angles (ASCE)
- Concrete around bolts broke (Douglass et al.)
- Broken anchor bolts (ASCE)
- Some spans on curve were pushed toward center of the curve, so this may have prevented them from being fully dislodged



Pontchartrain Causeway

Limited Information

Information source: “Hurricane Katrina, Performance of Transportation Systems” by ASCE

- Construction generally similar to the I-10 Twin Spans
- Most of spans above surge level, thus undamaged
- The “turnaround” spans were at a lower elevation and 17 were lost, no specifics given

US-11 over Lake Pontchartrain

Largely Undamaged—Limited Information

Information source: “Hurricane Katrina, Performance of Transportation Systems” by ASCE and Bridge Plans

- Largely undamaged
- Haunched continuous girders (according to ASCE report)
- Bridge plans appear to show that the spans were not continuous. Reinforcement was not continued from span to span at expansion or articulated joints.
- Air vents in diaphragms
  - Bridge plans indicate that these were provided for utilities (bridge lighting etc)
  - Where did these go, to an expansion joint??
  - How far did air have to travel to escape??
- Bridge plans show the following with respect to bearings and fixity:
  - Articulated joints were doweled
  - Expansion joints???

LA-1 over Camanda Bay

Limited Information

Information source: “Hurricane Katrina, Performance of Transportation Systems” by ASCE

- Simply supported reinforced concrete
- 13 shifted spans
- Spalling and exposed rebar from debris
  - Would a sacrificial barrier to protect the structure be practical?
- No mention of bearing types

David V. LaRosa Bridge

Limited information

Information source: “Hurricane Katrina, Performance of Transportation Systems” by ASCE

- No specifics given
- A few shifted spans

Precast Bridge at Bayou La Batre

Banging of Adjacent Boxes

Information source: “Hurricane Katrina, Performance of Transportation Systems” by ASCE

- Single span precast girder bridge
- Prestressed adjacent box beams
- No cast in place deck or diaphragms
- Spalling between beams caused by them banging into one another
- No cast in place shear keys, or cast in place deck, or post tensioning of the beams transversely to prevent rattling or banging of beams??

Dauphin Island Bridge

New Bridge Undamaged – Limited information

Information source: “Hurricane Katrina, Performance of Transportation Systems” by ASCE

- First bridge destroyed by hurricane Fredrick in 1979
- New bridge built using precast segmental construction
- Minimal damage to bridge (was this bridge above the surge?)
- Damage to approaches and fenders

Cochrane-Africatown USA Bridge

Vessel Impact

Information source: “Hurricane Katrina, Performance of Transportation Systems” by ASCE

- Cable stayed bridge
- Damaged cable when bridge was struck by oil platform

Biloxi Back Bay Bridge

Vessel Impact

Information source: “Hurricane Katrina, Performance of Transportation Systems” by ASCE

- High-span bridge (remained above surge?)
- Impacted by barge, which damaged a pile/bent
- Superstructure intact

Ocean Springs Pascagula

Vessel Impact

Information source: Mississippi DOT pictures and “Hurricane Katrina, Performance of Transportation Systems” by ASCE

- 13 to 18 feet of storm surge
- Repeatedly impacted by barges
    - One barge carried large cranes
    - One or more smaller barges
    - Also a tugboat
- Damaged piers (lateral forces failed rigid frame)
- A continuous span section (six spans) shifted 45 inches
- Damaged fascia girders
    - Spalling
    - Exposed strands
    - One completely destroyed

Seems to be undamaged by storm surge, possibly because of continuity of girders

TASK SUBMISSION

TASK 2 – REVIEW, SUMMARIZE, AND AUGMENT LITERATURE

Resistance Calculations of Failed Components

Table of Contents

1.0 Introduction

A portion of Task 2 of this project involves back calculating forces experienced in past events. The purpose is to determine the resistance of the components that failed during past hurricanes. This resistance will be compared to the wave forces calculated in later tasks of the project. The comparison will be used to determine whether the wave force calculation methods are yielding force magnitudes that reflect the observed damage. This will be accomplished by answering two questions:

(1) Is the estimated wave force sufficient to have done the observed damage?

(2) Should more damage have been evident?

An investigation of two bridges that experienced damage during coastal storms was performed. Estimates of the loads required to cause several different failure mechanisms were determined. The actual damage sustained was then examined and compared to the predicted failure mechanisms. Based on the occurrence or non-occurrence of failure mechanisms, maximum and minimum wave loads were estimated when possible. It should be emphasized that all forces contained herein represent loads developed from structural inference, not wave force equations or analysis of the seastate.

Typical 65' spans of the I-10 Lake Ponchartrain Bridge and typical 60' spans on flat grade of the I-10 Escambia Bay Bridge were investigated by considering the following failure modes:

- Displacement of spans due to insufficient anchorage to the substructure. This considered the strength of individual bearing components, friction and selfweight. Resistance to uplift, lateral, and longitudinal loads was investigated.
- Resistance of spans to negative bending due to uplift assuming the spans were sufficiently anchored at each end. This considered the cracking and ultimate capacity of the beams.
- Structural capacity of typical 3 pile bents to resist lateral and longitudinal loads applied at the top of the piles. This investigation focused on the formation of plastic hinges in the piles. Geotechnical capacity was not considered.

2.0 I-10 Lake Ponchartrain Bridge

The first structure examined was the I-10 Bridge over Lake Ponchartrain. The 5.5 mile bridge, which was built in 1963, consisted of two side by side spans each carrying three lanes of traffic. The majority of the bridge was constructed using 65' precast spans supported on three-pile bents. The exception to this was around the shipping channel where longer spans and higher bents with more piles were used. The 65' spans consisted of six girders and a deck which were cast monolithically at a casting yard on the lake. The three-pile bents were constructed using 54" diameter post-tensioned cylinder piles and reinforced concrete bent caps. The majority of the bridge was at a profile grade elevation of 15.25' above mean sea level. The design drawings give the mean high water elevation at the bridge site as 3.52' above mean sea level. According to a damage report by Volkert and Associates, 537 spans were displaced during Hurricane Katrina: 473

remained on the bents while 64 ended up in the lake. Figure 2.1 shows spans of the I-10 Lake Ponchartrain Bridge that were displaced longitudinally.

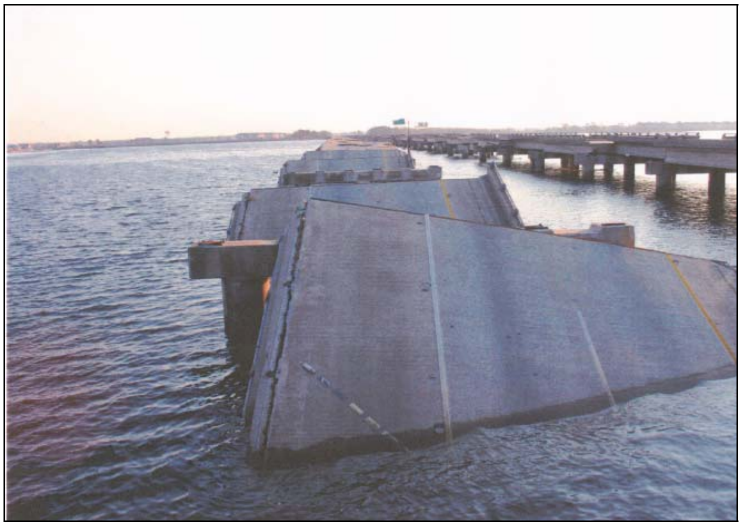


Figure 2.1 – I-10 Lake Ponchartrain Bridge after Hurricane Katrina
Source: Volkert and Associates Damage Assessment

2.1 Strength of Bearings

Typical 65' spans of the I-10 Lake Ponchartrain Bridge used bronze bearings. Three different configurations were used. Each configuration provided different resistances to wave loads and is discussed separately. The figures shown below were redrawn from the design drawings. However, the design drawings do not necessarily represent the as-built structure. Pictures of damage sustained by the I-10 Lake Ponchartrain Bridge indicate that end welded studs were used to connect the beam to the corresponding plate, not straps as shown in the plans. In bearings that used a strap connection, calculations suggest that the bolts would control. However, in bearings that used studs, pictures of the damaged bridge show that failure of both the studs and the bolts occurred. Figure 2.2 shows a beam with the bronze plate missing, indicating the plate to beam connection failed. This picture also indicates that end welded studs were used because the strap or a void left by the strap is not present in the bottom of the beam. Figure 2.3 shows a beam with the bronze plate intact, indicating bolt failure. Observed failure of both connections suggests that either the bolt and stud capacity were similar, or that the controlling component depended on which failure mode occurred (i.e. failure under lateral load or uplift load). No information on the type of studs that were used was available. Because accurate information about the plate to beam connections was unavailable and because the capacity of the plate to beam connection was not expected to be dramatically different than the bolt capacity, bolt capacities were used in the evaluation of the resistance of spans to displacement. Resistances given below assume no over/under strength of fasteners and no interaction of shear and tension on bolt strength. The resistances shown do not include friction, which will be added to the structural resistance in Section 2.2.

Figure 2.2 – Beams with Missing Bearing Plates on I-10 Lake Ponchartrain Bridge
Source: Volkert and Associates Damage Assessment

Figure 2.3 – Beam with Intact Bearing Plate on I-10 Lake Ponchartrain Bridge
Source: Volkert and Associates Damage Assessment

2.1.1 Fixed End Exterior Bearings

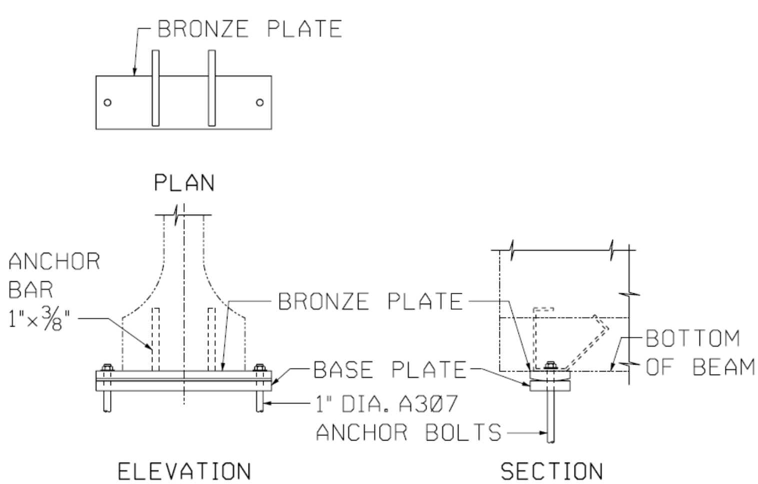


Figure 2.4 – Bearings at Fixed End Exterior Girders

Bearings at fixed end exterior girder (2)

- Resistance to uplift provided between beam and pier cap
  - Uplift capacity controlled by tension in bolts
  - Uplift capacity = 33.5 k per bolt
  - Two bolts per bearing
- Resistance to lateral loads provided between beam and pier cap
  - Lateral capacity controlled by shear in bolts (assume threads excluded)
  - Lateral capacity = 20.7 k per bolt
  - Two bolts per bearing
- Resistance to longitudinal loads provided between beam and pier cap
  - Longitudinal capacity controlled by shear in bolts (assume threads excluded)
  - Longitudinal capacity = 20.7 k per bolt
  - Two bolts per bearing

2.1.2 Fixed End Interior Bearings

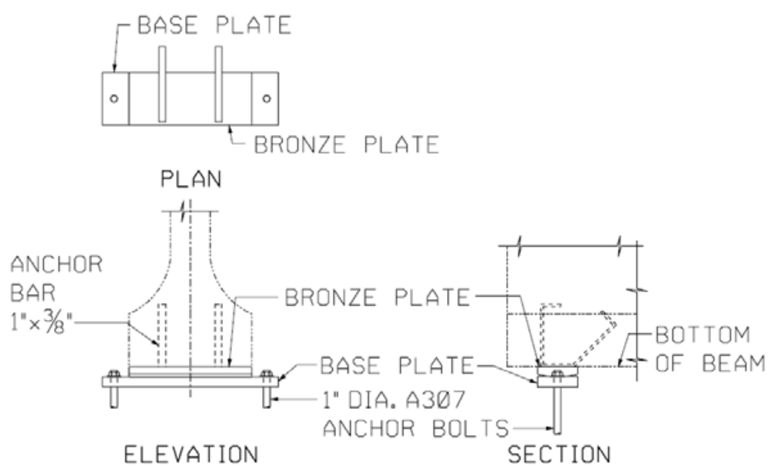


Figure 2.5 – Bearings at Fixed End Interior Girders

Bearings at fixed end interior girder (4)

- No resistance to uplift provided between beam and pier cap
  - Uplift capacity = 0 k
- No resistance to lateral loads provided between beam and pier cap
  - Lateral capacity = 0 k
- No resistance to longitudinal loads provided between beam and pier cap
  - Longitudinal capacity = 0 k

2.1.3 Expansion End Bearings

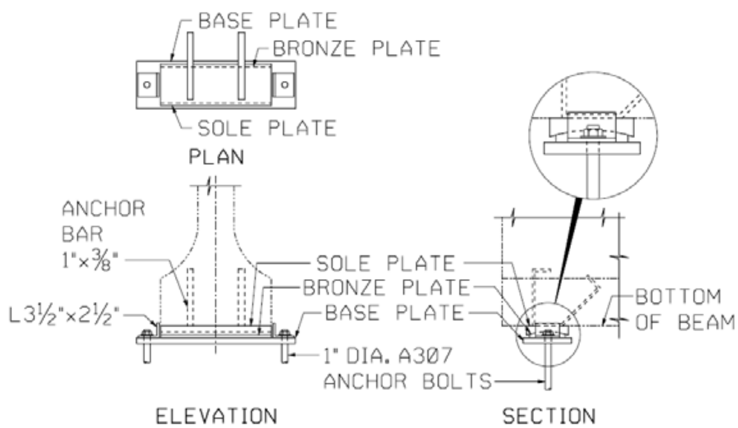


Figure 2.6 – Bearings at Expansion End All Girders

Bearings at expansion end (6)

- No resistance to uplift provided between beam and pier cap
  - Uplift capacity = 0 k
- Resistance to lateral loads provided between beam and pier cap
  - Angles resist lateral load, but under uplift section may lift above the angles
  - Lateral capacity controlled by shear in bolts (assume threads excluded)
  - Lateral capacity = 20.7 k per bolt (if section not lifted)
  - One bolt per bearing
  - Lateral capacity = 0 k (if section lifted)
- No resistance to longitudinal loads provided between beam and pier cap
  - Longitudinal capacity = 0 k

2.2 Resistance of Spans to Displacement

The spans on the I-10 Lake Ponchartrain Bridge were restrained by the bearings described above. Uplift was resisted at the fixed end by four bolts. Lateral movement was restrained by four bolts at the fixed end and by six bolts at the expansion end if the span was not lifted. If a span was lifted, lateral restraint would only be provided by four bolts at the fixed end. Longitudinal movement was restrained at the fixed end by four bolts. The extent to which the fasteners were simultaneously engaged is not known. The calculations that follow assume that the fasteners acted simultaneously, allowing their capacities to be additive. In addition to mechanical fasteners, friction and selfweight were considered.

Friction was assumed to contribute to the lateral and longitudinal capacity of the spans. The frictional resistance is a function of the normal force and coefficient of friction between contact surfaces. Because the normal force affects the frictional resistance, the lateral resistance of the span is dependant on the applied uplift load. The bearings used on the I-10 Lake Ponchartrain Bridge had more than one interface where sliding could have occurred. Sliding between steel and concrete, steel and bronze, and bronze and concrete was possible. Unrestrained bearings were designed to slide on the bronze to steel interface, and this behavior would be expected under wave loading. The sliding interface of restrained bearings depends on where failure occurs. Assuming bolt failure, as was done in the analysis of bearing capacities, sliding of bronze on steel would be expected at restrained bearings. Because steel on bronze sliding was expected, the coefficient of friction between these two materials was considered appropriate for use in resistance calculations. However, an appropriate coefficient was not able to be determined. The AASHTO LRFD specification indicates that a coefficient of friction between 0.07 and 0.10 may be expected during the life of lubricated bronze bearings, and that a coefficient of friction of 0.4 may be conservatively used for the design of unlubricated bronze bearings. The bearings of the I-10 Lake Ponchartrain Bridge were lubricated bronze, however, after more than 40 years in service the bearings exhibited signs of corrosion as seen in Figure 2.3. Therefore, the frictional resistance was bracketed by using coefficient of friction values of 0.10 and 0.40.

The self weight of the span provided the majority of the resistance to uplift and contributed to the lateral resistance of the span through friction when uplift loads were small. The self weight of the span was calculated to be 535 kips.

Binding or pinching of adjacent spans was not considered, although it is possible that this occurred.

2.2.1 Uplift Resistance of Spans

- The maximum uplift resistance would be obtained if the fixed end exterior bearings were assumed effective. However, restraining the span vertically at one end only could lead to rigid body rotation about the restrained end.
    - Maximum uplift resistance = selfweight + fixed end exterior bearings
    - Maximum uplift resistance = 535 k + 4 * 33.5 k = 669 k
- The lower bound on uplift resistance would be obtained by accounting for only the dead load of the span. Because the span is restrained vertically only at one end, the provided bolts may be ineffective, and are not accounted for in this calculation.
    - Minimum uplift resistance = selfweight
    - Minimum uplift resistance = 535k

2.2.2 Lateral Resistance of Spans

- Lateral resistance of spans with no simultaneous uplift loading:
    - Lateral resistance provided by fasteners was obtained by assuming that the total reactions at each bent were equal, i.e. the expansion bearings are

capable of providing a larger reaction than the fixed bearings, so the capacity of the fixed bearings was assumed at each bent.
  - Frictional resistance was based on the full dead load of the span
  - Lateral resistance = fastener capacity + frictional resistance.
  - Lateral resistance = 8 * 20.7k + (0.10 to 0.40) * 535k
  - Lateral resistance = 166k + friction = 219 k to 380 k
- Lateral resistance of spans with simultaneous uplift loading less than the dead load:
  - Lateral resistance provided by fasteners is the same as when no uplift loads are present
  - Frictional resistance is based on the net vertical load of the span
  - Lateral resistance = fastener capacity + frictional resistance.
  - Lateral resistance = 8 * 20.7 k + (0.10 to 0.40) * (535k – uplift)
  - Lateral resistance = 166k + friction
- Lateral resistance of spans with simultaneous uplift loading greater than the dead load:
  - If the uplift load exceeds the dead load, there is no frictional resistance
  - If the uplift load exceeds the dead load of the span, the beams may be lifted above the angles in the expansion bearings, and lateral restraint at the expansion end will be lost.
  - Once frictional restraint and lateral restraint at the expansion bearings is lost, the bearings at the fixed end exterior beams will provide the only lateral restraint. Because the span will be restrained only at one end, the bearings will be subjected to forces in both the longitudinal and transverse directions as shown in Figure 2.7.
  - Lateral resistance = fastener capacity
  - Lateral resistance = 41 k

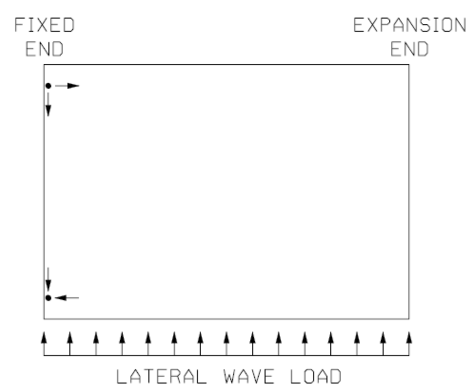


Figure 2.7 - Unsymmetric Restraint of Span Subjected to Lateral Load

Figure 2.8 shows the expected lateral resistance of the span as a function of the uplift load on the span. The coefficients of friction used represent the expected upper and lower bounds for the bearings on the I-10 Lake Ponchartrain Bridge.

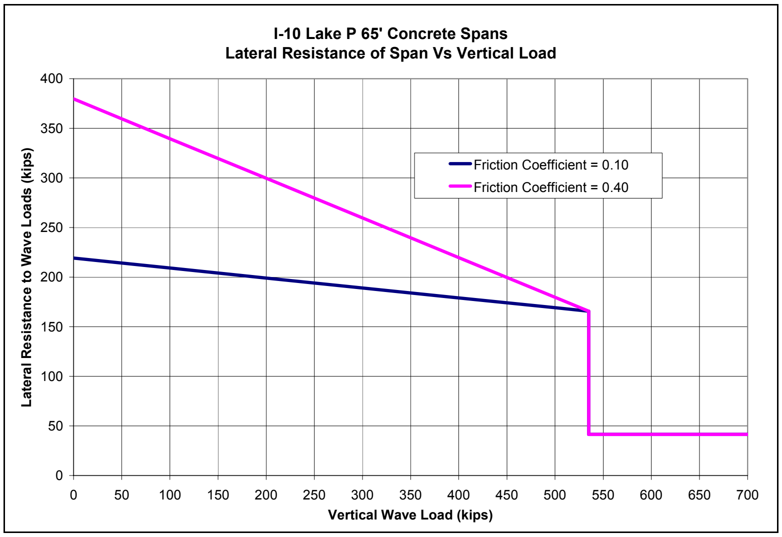


Figure 2.8 – Lateral Resistance of I-10 Lake Ponchartrain Spans vs. Uplift Loads

2.2.3 Longitudinal Resistance of Spans

- Longitudinal resistance of spans with no simultaneous uplift loading:
  - Longitudinal fastener resistance was due to four bolts at the fixed end
  - Frictional resistance was based on the full dead load of the span
  - Longitudinal resistance = fastener capacity + frictional resistance
  - Longitudinal resistance = 4 * 20.7k + (0.10 to 0.40) * 535k = 136k to 297k
- Longitudinal resistance of spans with simultaneous uplift loading less than the dead load:
  - Longitudinal resistance provided by fasteners is the same as when no uplift loads are present
  - Frictional resistance is based on the net vertical load of the span
  - Longitudinal resistance = fastener capacity + frictional resistance
  - Longitudinal resistance = 4 * 20.7 k + (0.10 to 0.40) * (535k – uplift)
- Longitudinal resistance of spans with simultaneous uplift loading greater than the dead load:
  - Longitudinal resistance provided by fasteners is the same as when no uplift loads are present
  - If the uplift load exceeds the dead load, there is no frictional resistance
  - Lateral resistance = fastener capacity
  - Lateral resistance = 4 * 20.7 k = 83 k

Figure 2.9 shows the expected longitudinal resistance of the span as a function of the uplift load on the span. The coefficients of friction used represent the expected upper and lower bounds for the bearings on the I-10 Lake Ponchartrain Bridge.

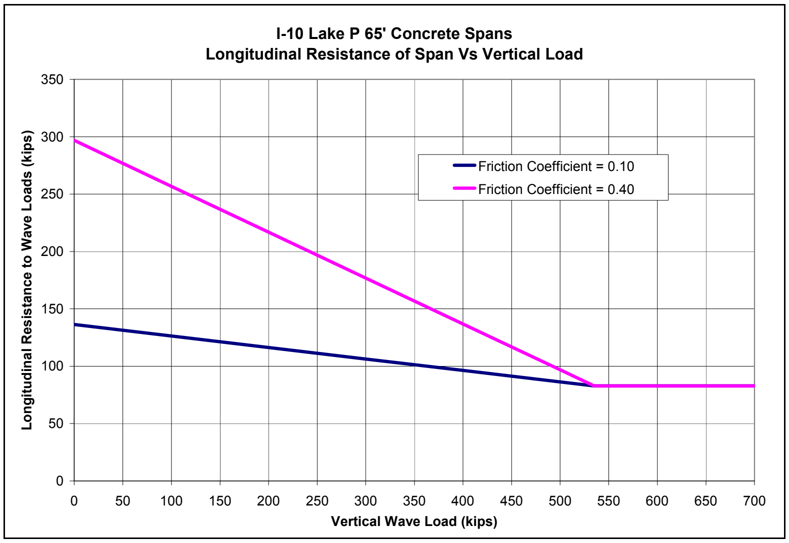


Figure 2.9 – Longitudinal Resistance of I-10 Lake Ponchartrain Spans vs. Uplift Loads

2.3 Resistance of Girders

Uplift loads due to waves, surge, and buoyancy act in a direction that was likely not considered in design of the girders. Damage to the girders due to uplift loads was investigated to determine if these loads were a factor in the failure of the bridge, or if they could have been a factor if the spans were sufficiently anchored to the substructure. Specifically, negative bending of the girders was investigated. The spans of the I-10 Lake Ponchartrain Bridge were monolithic prestressed beam/deck segments. Thus, the distribution of stress in the section due to prestress and bending was continuous. A typical interior girder was analyzed assuming the effective deck width to be one half of the girder spacing. Transformed section properties were used in stress calculations and strain compatibility was used in ultimate strength calculations.

The cracking moment (M_{cr}) of the beam was determined using stress calculations. The calculated cracking moment was the moment required to change the stress in the top of the section from the stress under dead load to a tensile stress of 7.5 sqrt(f'c). Because the cracking moment of an interior girder is not readily comparable to an uplift force on a span, a conversion from moments to forces was made. This was done by assuming that all girders in the cross section had the same moment capacity and that the uplift load was uniformly distributed both transversely and longitudinally. The length of the span was taken as the center to center distance between the bearings. The uplift force (P_{cr}) is equivalent to the applied force required to cause the cracking moment at midspan in all girders in the cross section.

- M_{cr} = 1,132 k-ft for a typical interior girder
- P_{cr} = 858 k per span (uniformly distributed)

The ultimate moment capacity (M_n) of a typical interior girder was calculated using strain compatibility. Unlike stress calculations, where dead load is already accounted for, the dead load of the span must be overcome in addition to M_n to cause failure. Therefore, the total applied moment (M_a) to cause an ultimate type failure is the sum of M_n and the selfweight moment (M_{sw}). The uplift force (P_a) is equivalent to the total force from a uniformly distributed load that would cause ultimate failure in negative bending at midspan in all girders in the cross section.

- M_n = 492 k-ft for a typical interior girder
- M_{sw}= 551k-ft for a typical interior girder
- M_a = 1,043 k-ft for a typical interior girder
- Note: Ma = 1,043 k-ft < M_{cr} = 1,132 k-ft
- P_a = 790 k per span assuming a uniformly distributed load.

Assuming that the deck has not been cracked transversely in service, or by previous events, an interior girder may be loaded to 1,132 k-ft at midspan. At this loading, the beam/deck will crack. Once the beam is cracked the section will only be capable of resisting a moment of 1,043 k-ft.

2.4 Structural Resistance of Bents

The typical bent supporting 65' spans on the I-10 Lake Ponchartrain bridge consisted of three 54" diameter post-tensioned cylinder piles and a 3' deep reinforced concrete bent cap. The top 4' of the piles were filled with concrete and reinforced to facilitate continuity between the piles and the pier cap. The top of the piles were about 18' above the mud line for typical bents. The plastic moment capacity of each pile was determined to be about 1,825 k-ft using strain compatibility.

An investigation of the horizontal load required to cause plastic moments to form in the piles was conducted using COM624P. Investigations were conducted with two different assumed soils. The first soil was soft clay with a soil modulus of 30 lb/in^3, unit weight of 100 lb/ft^3, cohesion of 110 lb/ft^2, and strain at 50 percent stress of 0.02. The second soil was sand with a soil modulus of 60 lb/in^3, unit weight of 120 lb/ft^3, and an angle of internal friction of 32°. The assumed soils were based on soil properties from the site of the I-10 Escambia Bay Bridge because specific information about soils at the Lake Ponchartrain Bridge was not available. Single piles were modeled in COM624P and the forces required to form plastic moments were multiplied by three to determine the bent capacity. The uncracked moment of inertia was used in COM624P calculations.

2.4.1 Lateral Resistance of Bents

The maximum lateral resistance of the bents was determined by applying a horizontal load to a fixed head pile until a plastic moment formed. The first plastic moment formed at the head of the pile for both clay and sand. At this point the model was changed to a free head pile with an applied moment equal to the plastic moment. The new model was then loaded until a plastic moment formed below the mud line. This treatment of the bent neglects the possibility of plastic moments forming in the bent cap as well as the possibility that the pile to bent cap connection will fail. Both of these would reduce the capacity of the bent. However, the lateral capacity would still be greater than the longitudinal capacity discussed below. The possible increase in strength due to the 4' plug at the top of the pile was also ignored.

- Lateral capacity in clay
  - Mp forms at pile head at a horizontal load of 61 k
  - Mp forms 27' below the mud line at a horizontal load of 113 k
  - Bent capacity = 3 * 113 k = 339 k per bent
- Lateral capacity in sand
  - Mp forms at pile head at a horizontal load of 106 k
  - Mp forms 10' below the mud line at a horizontal load of 150 k
  - Bent capacity = 3 * 150 k = 450 k per bent

Because bents support two spans, the maximum load that may be applied to a span before bent failure occurs is a function of both the bent resistance and the loading applied to adjacent spans. For example, assume a span were subjected to a symmetrically applied lateral load of 100k, the reaction at each bent would be 50k. If adjacent spans are not loaded simultaneously, each bent resists a total lateral load of 50k. However, if an adjacent span is loaded simultaneously with a similar load, the bent between two loaded spans will be required to support 100 k of lateral load (50k/span * 2 spans). The degree to which adjacent spans are simultaneously laterally loaded by waves is not clear. If a bridge were impacted by a wide wave traveling perpendicular to the bridge, it seems possible that simultaneous lateral loading of adjacent spans could occur. However, detailed evaluation of the seastate would likely be required to determine the likelihood of such an occurrence. The possibility of simultaneous lateral loading of adjacent spans will be considered in addition to lateral loading of a single span.

- Lateral capacity in clay
  - One span loaded = 678 k per span
  - Two spans loaded = 339 k per span
- Lateral capacity in sand
  - One span loaded = 900 k per span
  - Two spans loaded = 450 k per span

2.4.2 Longitudinal Resistance of Bents

The maximum longitudinal resistance of the bents was determined by applying a horizontal load to a free head pile until a plastic moment formed. The plastic moment formed below the mud line as would be expected for a cantilevered structure.

- Longitudinal capacity in clay
  - Mp forms 19' below the mud line at a horizontal load of 65 k
  - Bent capacity = 3 * 65 k = 195 k per bent
- Longitudinal capacity in sand
  - Mp forms 7' below the mud line at a horizontal load of 80 k

- Bent capacity = 3 * 80 k = 240 k per bent

The simultaneous loading of adjacent spans may also be relevant to longitudinal loading. On the I-10 Lake Ponchartrain Bridge, mechanical resistance to longitudinal loading was only provided at fixed bearings, thus longitudinal restraint provided by bolts is not shared by the two bents supporting a span. However, the frictional restraint at each bent supporting a span may be assumed to be equal. When one span is loaded, the expansion end bent will be subjected to friction and the fixed end bent will be subjected to friction and the restraint provided by the bolts. For simplicity, when one span is loaded longitudinally, it will be assumed that each bent takes one half of the load applied to the span. This is a reasonable, practical expediency because the maximum force transferred through the bolts is small compared to the force that may be transferred through friction. The bearings on the I-10 Lake Ponchartrain Bridge appear to have been laid out such that a bent would support one fixed and one expansion bearing. If simultaneous and equal loading of adjacent spans occurred, the bents would be subjected to a load equal to the load on one span.

- Lateral capacity in clay
    - Two spans loaded = 195 k per span
    - One span loaded = 390 k per span
- Lateral capacity in sand
    - Two span loaded = 240 k per span
    - One span loaded = 480 k per span

2.5 Summary of Force Estimates for I-10 Lake Ponchartrain Bridge

The uplift resistance of the spans was:

535 k to 669 k based on span dislodgment

790 k to 858 k based on negative bending of the beams

Using the predicted capacities above, it would be expected that the spans would dislodge before they were failed in negative bending.

Lateral resistance of the bridge was:

41 k to 380 k based on span dislodgment

339 k to 900 k per span based on bent resistance

The wide range of resistance for span dislodgment under lateral loads is due to frictional resistance and disengagement of expansion end bearings under uplift. The wide range of values for bent resistance is due to uncertainty regarding the simultaneous loading of adjacent spans and soil conditions at the site.

Longitudinal resistance of the bridge was:

83 k to 297 k based on dislodgment

195 k to 480 k based on bent resistance

The wide range of resistance for span dislodgment under lateral loads is due to friction. The range in bent resistance is due to uncertainty regarding the simultaneous loading of adjacent spans and soil conditions at the site.

With respect to lateral and longitudinal loading, strictly comparing the capacity values above does not allow a failure mode to be easily determined. This is due to the dependency of span dislodgment resistance on vertical load, uncertainty relating to simultaneous loading of adjacent spans, and uncertainty in soil conditions at the site. To better asses the ranges above, a linear programming approach was selected. The approach seeks to examine known boundary conditions to find a range of acceptable solutions. Figure 2.10 is a plot of the examined lateral and longitudinal failure mechanisms. Horizontal lines represent lateral load resistances that are relatively independent of uplift loads, vertical lines represent vertical resistances that are relatively independent of lateral loads, and sloped lines represent lateral resistances that are dependent on vertical loads.

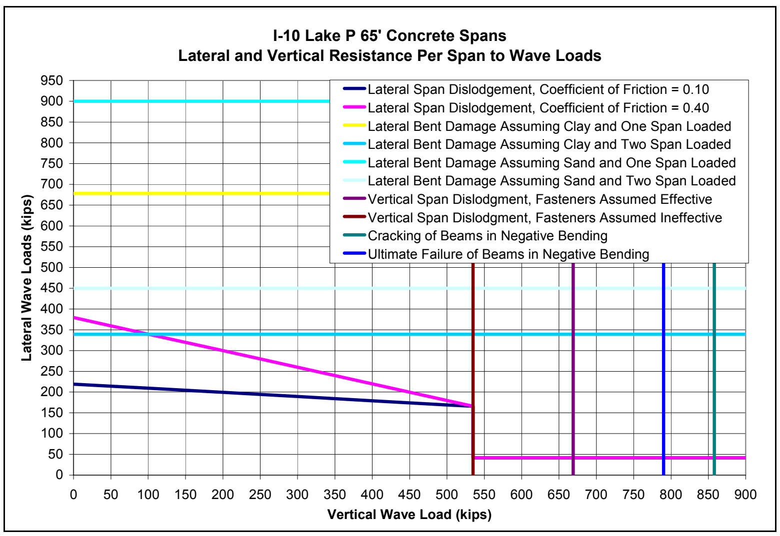


Figure 2.10 – Plot of Lateral and Vertical Resistances for I-10 Lake Ponchartrain Bridge

Examination of the chart reveals that several failure modes may be removed completely from consideration because other failure modes would occur first. For example, negative bending damage to the beams would be preceded by span dislodgment. Three of the four bent damage scenarios would also be preceded by span dislodgment. Other failure modes may be seen to control over only a small range of loads. For example, “Lateral Bent Damage Assuming Clay and Two Spans Loaded” will control over “Lateral Span Dislodgement, Coefficient of Friction = 0.40” when vertical loads are less than about 100 kips. Removal of failure modes that will not occur (because other failure modes occur first) in the above graph will lead to Figure 2.11.

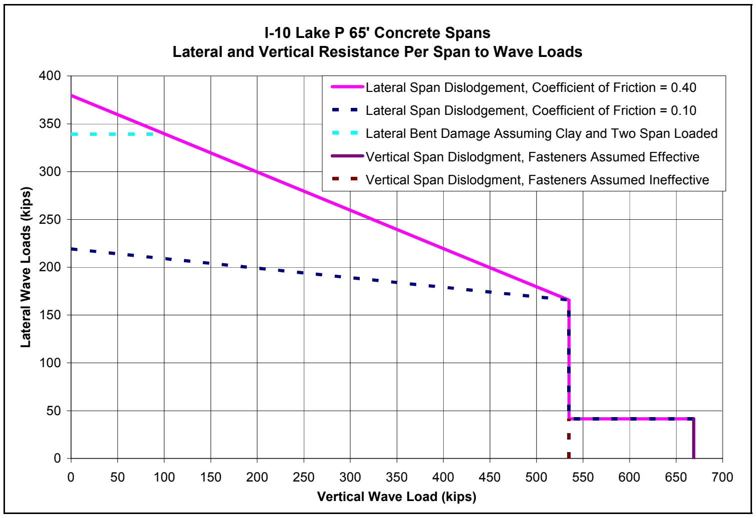


Figure 2.11 – Failure Envelope for Lateral and Vertical Loads

In Figure 2.11, it may be noted that "Lateral Span Dislodgement, Coefficient of Friction = 0.10" will control over "Lateral Span Dislodgement, Coefficient of Friction = 0.40." This is true, however, the actual coefficient of friction is not known, 0.40 represents an upper bound while the 0.10 represents a lower bound. Both lines were retained to represent this uncertainty. A similar situation exists with vertical span dislodgment. Because of the unsymmetric restraint of the span, it is unclear how effective provided fasteners would be at restraining the span vertically. Thus, both lines were retained. The line showing "Lateral Bent Damage Assuming Clay and Two Spans Loaded" was not selected as the cut off point for lateral span dislodgment because of uncertainty relating to soil conditions and the simultaneous loading of the spans. Solid lines in Figure 2.11 represent values that are relatively certain. Points outside of the solid lines will cause failure of the bridge. Dashed lines represent failure boundaries with more uncertainty. Points outside of dashed lines represent possible failure or possible non-failure, depending on conditions. For example, consider a vertical load of 25 k in combination with a lateral load of 350 k. This load will not cause span dislodgment if the true coefficient of friction is 0.40. However, if the true coefficient of friction is 0.10, span dislodgment would occur. The load also has the potential to cause pier damage. If the actual soil conditions at the site are represented by the assumed sand, the piers should not fail. If the actual soils at the site are represented by the clay, the condition of simultaneous loading will determine failure: if adjacent spans are simultaneously loaded failure may be expected, but if only one span were loaded, failure would not be expected.

A similar linear programming process was conducted for lateral and vertical loads. The results are shown in Figure 2.12. In this figure horizontal lines are longitudinal load resistances that are relatively independent of uplift loads, vertical lines are vertical resistances that are relatively independent of longitudinal loads, and sloped lines are longitudinal resistances that are dependent on vertical loads

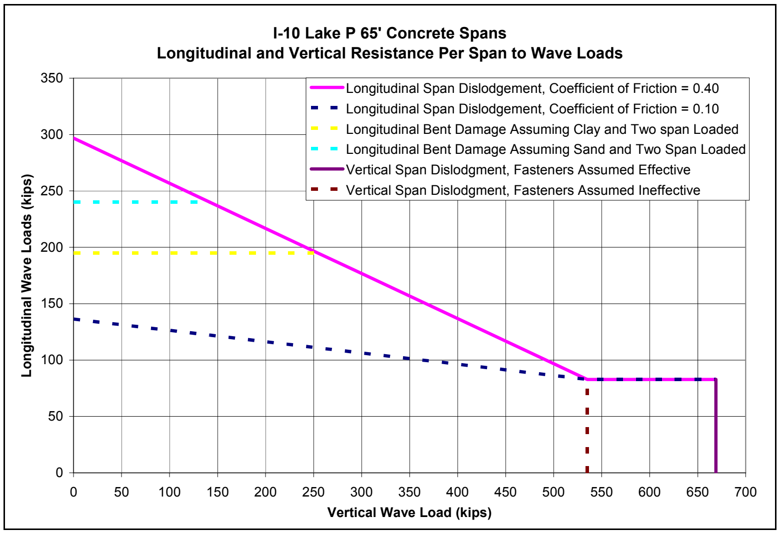


Figure 2.12 - Failure Envelope for Longitudinal and Vertical Loads

With respect to wave loads actually reacted by the spans, estimation of horizontal wave forces is not practical because the effect of vertical load on horizontal resistance of the spans creates a significant amount of uncertainty. Failure of bents, which have a capacity that is relatively independent of vertical loads, was not seen. The non-occurrence of bent failure does not exclude the possibility that the seastate could cause lateral wave loads in excess of the bent capacity because, if/when these loads occurred, the spans were no longer in place. It is possible that the spans were completely lifted by large vertical forces. If this occurred, there would be no resistance to horizontal loads and the spans could be displaced by small lateral or longitudinal forces.

The minimum magnitude of vertical wave loads may be estimated as 535 k to 669 k. This estimate is based on the displacement of spans. Although the spans could have been displaced by lateral loads, several bents appeared to have had damage from beams being dropped or slammed onto the bent cap, suggesting the girders were lifted. Also, as shown in Figure 2.13, some bents had obstacles such as stepped beam seats or anchor bolts, which would exhibit damage if the spans were pushed across the bent cap without being lifted. Evidence indicates that many spans were lifted by vertical forces. The non-occurrence of negative bending failures does not exclude the possibility that the seastate could cause vertical loads in excess of the negative bending capacity of the beams. This is because the beams were not vertically restrained at both ends, thus span dislodgment would occur before sufficient negative bending could not have occurred. When comparing the linear programming results to bound a solution, it should be recognized that the direction of the waves that caused the damage is not entirely known. It is likely that the actual direction of waves was not perpendicular to or parallel to the bridge, but somewhere in between. This may be confirmed by the fact that both longitudinal and lateral span displacement occurred.

Figure 2.13 – Undamaged Bolt where Beam was Displaced Laterally
Source: Volkert and Associates Damage Assessment

3.0 I-10 Escambia Bay Bridge

The second structure examined was the I-10 Bridge over Escambia Bay. The 2.6 mile bridge, which was built in the mid 1960's, consisted of two side by side spans, each carrying two lanes of traffic. The majority of the bridge was constructed using 60' precast spans supported on intermediate or tower bents. Intermediate bents used a single row of three piles, while tower bents used two rows of three piles. Tower bents were used every seventh bent. The exception to this was around the shipping channel where longer spans and higher bents with more or larger piles were used. The 60' spans consisted of six girders and a deck which were cast monolithically. The intermediate and tower bents were constructed using 36" diameter post-tensioned cylinder piles and reinforced concrete bent caps. The majority of the bridge was at a profile grade elevation of 16.0'. The FDOT damage report performed after Hurricane Ivan lists 46 spans as "gone" and photographs contained in the report showed that numerous spans were shifted. The damage report also lists 8 bents as "gone", "destroyed", or "1 pile remaining." Less severe damage to 11 other bents is also listed. Figure 3.1 shows lost and shifted spans as well as damaged and missing bents of the I-10 Escambia Bay Bridge.

Figure 3.1 – I-10 Escambia Bay Bridge after Hurricane Ivan
Source: FDOT

3.1 Strength of Bearings

Typical 60’ spans of the I-10 Escambia Bay Bridge used neoprene bearings. The design drawings indicate that spans on flat grade would utilize one bearing configuration on the fascia girders and a second bearing configuration on all interior girders. The two configurations provided different resistances to wave loads and are discussed separately below. The figures showing each bearing type were redrawn from the design drawings. Design drawings for the exterior girder bearings indicate that either end welded studs or welded straps were permissible for attaching the bearing plates to the beams. It is not known which detail was used. Because of this uncertainty, the bolts were assumed to control the capacity of the bearings. Information was also obtained that suggested 7/8” diameter bolts were used instead of the 1” diameter bolts shown in the plans. The following calculations assume that 7/8” diameter bolts were used. Pictures of the damaged Escambia Bay Bridge show that failure of both the bearing-plate-to-beam and bearing-plate-to-bent-cap connections occurred. Figure 3.2 shows displaced girders with the bearing plates still attached, suggesting bolt failure occurred. Figure 3.3 shows displaced girders where the bearing plate is still partially attached to the bent cap, suggesting stud/strap failure. The fact that failures of both connections were observed suggests that either the bolt and stud/strap capacities were similar, or that the controlling component depended on which failure mode occurred (i.e. failure under lateral load or uplift load). Resistances given below assume no over/under strength of fasteners and no interaction of shear and tension on bolt strength. Because of the factors listed above, the calculated capacities are likely an upper bound for the actual capacities of the bearings. The resistances shown do not include friction, which will be added to the structural resistance in Section 3.2.

Figure 3.2 – Girders with Bearing Plates Intact on Escambia Bay Bridge
Source: FDOT Damage Report

Figure 3.3 – Bent Cap with Bearing Plate Partially Attached on Escambia Bay Bridge
Source: FDOT Damage Report

3.1.1 Fascia Girder Bearings

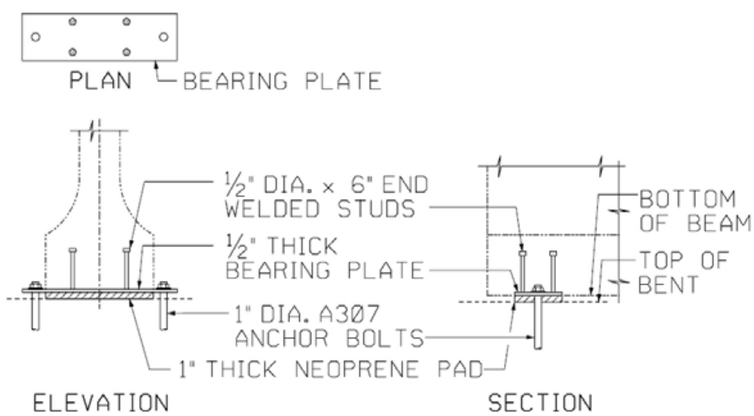


Figure 3.4 – Fascia Girder Bearings

Fascia girder bearings (4)

- Resistance to uplift provided between beam and pier cap
    - Uplift capacity assumed to be controlled by tension in bolts
    - Uplift capacity = 25.4 k per bolt
    - Two bolts per bearing
- Resistance to lateral loads provided between beam and pier cap
    - Lateral capacity assumed to be controlled by shear in bolts (assume threads excluded)
    - Absence of a well defined shear plane in the bearing was not considered in the calculation of shear capacity
    - Lateral capacity = 15.9 k per bolt
    - Two bolts per bearing
- Resistance to longitudinal loads provided between beam and pier cap
    - Longitudinal capacity assumed to be controlled by shear in bolts (assume threads excluded)

- Absence of a well defined shear plane in the bearing was not considered in the calculation of shear capacity
- Longitudinal capacity = 15.9 k per bolt
- Two bolts per bearing

3.1.2 Interior Girder Bearings

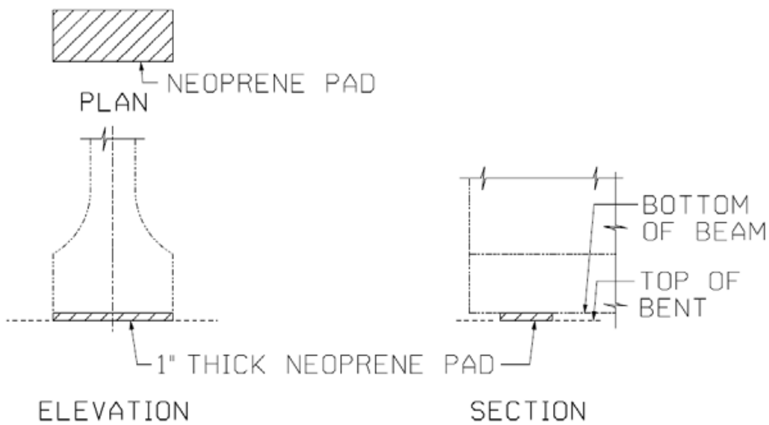


Figure 3.5 – Interior Girder Bearings

Bearings at interior girders (8)

- No resistance to uplift provided between beam and pier cap
  - Uplift capacity = 0 k
- No resistance to lateral loads provided between beam and pier cap
  - Lateral capacity = 0 k
- No resistance to longitudinal loads provided between beam and pier cap
  - Longitudinal capacity = 0 k

3.2 Resistance of Spans to Displacement

The spans on the I-10 Escambia Bay Bridge were restrained by the bearings described above. Uplift, lateral, and longitudinal loads were resisted by eight bolts, four at each end of the bridge. The extent to which the fasteners were simultaneously engaged is not known. The calculations that follow assume that the fasteners acted simultaneously for resisting uplift and lateral loads. However, this assumption was not used for longitudinal loads. This is because bearing plates at the expansion end had slots to permit thermal movement. Because slots were provided at the expansion end, simultaneous engagement of all 8 bolts was unlikely. Therefore, only the four bolts at the fixed end were assumed effective at resisting longitudinal loads. In addition to mechanical fasteners, friction and selfweight were considered.

Friction was assumed to contribute to the lateral and longitudinal capacity of the spans. The frictional resistance is a function of the normal force and coefficient of friction between contact surfaces. Because the normal force affects the frictional resistance, the lateral resistance of the span is dependant on the applied uplift load. The bearings used on the I-10 Escambia Bay Bridge had more than one interface where sliding could occur. Assuming failure of bolts in the fascia girder bearings, sliding between neoprene and concrete or neoprene and steel would occur. For interior girder bearings, sliding would occur between neoprene and concrete. Detailed information about the coefficient of friction between neoprene and steel or neoprene and concrete was not located. Available information indicated that the potential exists for the coefficient of friction to be significantly greater than unity. Because of the uncertainty involved, a coefficient of friction of 1.0 was assumed for both neoprene on concrete and neoprene on steel.

The self weight of the span provided the majority of the resistance to uplift and contributed to the lateral resistance of the span through friction when uplift loads were small. The self weight of the span was calculated to be 398 kips.

Binding or pinching of adjacent spans was not considered, although it is possible that this occurred.

3.2.1 Uplift Resistance of Spans

- Uplift resistance = selfweight + fasteners
- Uplift resistance = 398 k + 8 * 25.4 k = 601 k

3.2.2 Lateral Resistance of Spans

- Lateral resistance of spans with no simultaneous uplift loading:
    - Frictional resistance based on full dead load of the span
    - Lateral resistance = fastener capacity + frictional resistance
    - Lateral resistance = 8 * 15.9k + 1.0 * 398k = 525 k
- Lateral resistance of spans with simultaneous uplift loading less than the dead load:
    - Frictional resistance based on net vertical load of the span
    - Lateral resistance = fastener capacity + frictional resistance
    - Lateral resistance = 8 * 15.9 k + 1.0 * (535k – uplift)
- Lateral resistance of spans with simultaneous uplift loading greater than the dead load:
    - If the uplift load exceeds the dead load there is no frictional resistance
    - Lateral resistance = fastener capacity
    - Lateral resistance = 8 * 15.9 k = 127 k

Figure 3.6 shows the expected lateral resistance of the span as a function of the uplift load on the span. However, there is uncertainty in the coefficient of friction, so the slope of the line when the applied vertical load is less than the dead load could be modified accordingly.

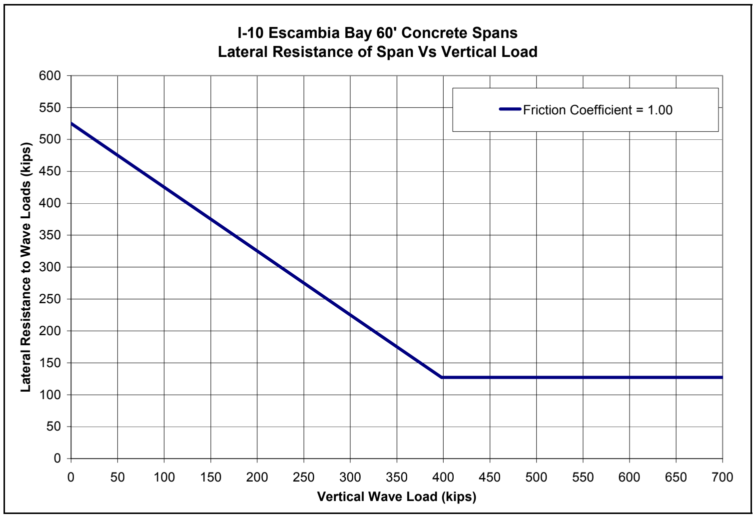


Figure 3.6 – Lateral Resistance of I-10 Escambia Bay Spans vs. Uplift Loads

3.2.3 Longitudinal Resistance of Spans

- Longitudinal resistance of spans with no simultaneous uplift loading:
  - Frictional resistance based on full dead load of span
  - Longitudinal resistance = fastener capacity + frictional resistance
  - Longitudinal resistance = 4 * 15.9k + 1.0 * 398 k = 462 k
- Longitudinal resistance of spans with simultaneous uplift loading less than the dead load:
  - Frictional resistance based on net vertical load of span
  - Longitudinal resistance = fastener capacity + frictional resistance
  - Longitudinal resistance = 4 * 15.9 k + 1.0 * (535k – uplift)
- Longitudinal resistance of spans with simultaneous uplift loading greater than the dead load:
  - If the uplift load exceeds the dead load there is no frictional resistance
  - Lateral resistance = fastener capacity
  - Lateral resistance = 4 * 15.9 k = 64 k

Similar to the prediction of lateral resistance, the prediction of longitudinal resistance contains uncertainty with respect to the coefficient of friction. The predicted longitudinal resistance of spans to displacement shown in Figure 3.7 could be modified accordingly.

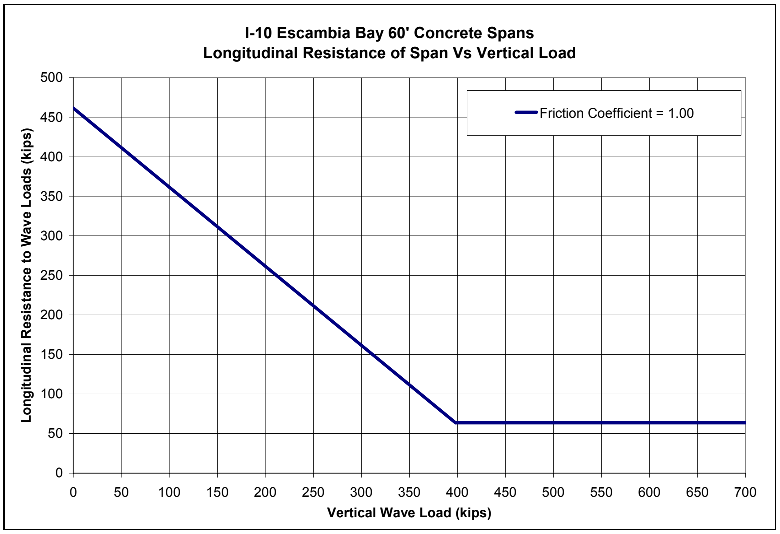


Figure 3.7 – Longitudinal Resistance of I-10 Escambia Bay Spans vs. Uplift Loads

3.3 Resistance of Girders

Negative bending of the girders was investigated to assess if the monolithic prestressed beam/deck segments of the I-10 Escambia Bay Bridge were susceptible to damage from uplift loads. The methods employed for calculating the cracking moment, ultimate moment, and corresponding forces were similar to those described for the I-10 Lake Ponchartrain Bridge.

The cracking moment and corresponding cracking force are:

- M_{cr} = 1,132 k-ft for a typical interior girder
- P_{cr} = 930 k per span (assuming a uniformly distributed load)

The ultimate moment, selfweight moment, applied ultimate moment, and applied ultimate force are:

- M_n = 1,166 k-ft for a typical interior girder
- M_{sw}= 406 k-ft for a typical interior girder
- M_a = 1,572 k-ft for a typical interior girder
- Note: Ma = 1,572 k-ft > M_{cr} = 1,132 k-ft
- P_a = 1292 k per span (assuming a uniformly distributed load)

3.4 Structural Resistance of Bents

The typical intermediate bent supporting 60' spans on the I-10 Escambia Bay Bridge consisted of three 36" diameter post-tensioned cylinder piles and a 3' deep reinforced concrete pier cap. The two exterior piles were battered at 1 inch per foot and the interior pile was plumb. The top 4' of the piles were filled with concrete and reinforced to facilitate continuity between the piles and the pier cap. The top of the pile was about 17' above the mud line for typical bents. The plastic moment capacity of each pile was determined to be about 880 k-ft using strain compatibility.

An investigation of the horizontal load required to cause plastic moments to form in the piles was conducted using COM624P. Investigations were conducted using a simplified soil profile considered to be generally representative of conditions at the site. The profile was developed using information from the foundation report for the new I-10 Escambia Bay Bridge. The foundation report contained information on over 100 borings taken along the new alignment as well as suggested soil parameters to be used in FLPIER. Typical borings away from the edge of the bay indicated one or more layers of clay or silt overlying various layers of sand. The assumed soil profile used in COM624P consisted of 40 feet of clay overlying sand. The clay had a soil modulus of 30 lb/in^3, unit weight of 100 lb/ft^3, cohesion of 110 lb/ft^2, and strain at 50 percent stress of 0.02. The sand had a soil modulus of 60 lb/in^3, unit weight of 120 lb/ft^3, and an angle of internal friction of 32°. Single piles were modeled in COM624P and the forces required to form plastic moments were multiplied by three to determine the bent capacity. The uncracked moment of inertia was used in COM624P calculations.

3.4.1 Lateral Resistance of Bents

The maximum lateral resistance of the bents was determined by applying a horizontal load to a fixed head pile until a plastic moment formed. The first plastic moment formed at the head of the pile. At this point the model was changed to a free head pile with an applied moment equal to the plastic moment. The new model was then loaded until a plastic moment formed below the mud line. This treatment of the bent neglects the possibility of plastic moments forming in the bent cap as well as the possibility that the pile to bent cap connection will fail. Both of these would reduce the capacity of the bent. However, the lateral capacity would still be greater than the longitudinal capacity discussed below. The possible increase in strength due to the 4’ plug at the top of the pile and the battering of exterior piles was also ignored.

- Lateral capacity
    - Mp forms at pile head at a horizontal load of 37 k
    - Mp forms 30’ below the mud line at a horizontal load of 55 k
    - Bent capacity = 3 * 55 k = 165 k per bent

As discussed in Section 2.4.1, because bents support two spans, the maximum load that may be applied to a span before bent failure occurs is a function of both the bent resistance and the loading applied to adjacent spans. Because of the uncertainty in application of wave loads, the possibility of simultaneous lateral loading of adjacent spans will be considered in addition to lateral loading of a single span.

- Lateral capacity
    - One span loaded = 330 k per span
    - Two spans loaded = 165 k per span

3.4.2 Longitudinal Resistance of Bents

The maximum longitudinal resistance of the bents was determined by applying a horizontal load to a free head pile until a plastic moment formed. The plastic moment formed below the mud line as would be expected for a cantilevered structure.

- Longitudinal capacity
  - Mp forms 17’ below the mud line at a horizontal load of 33 k
  - Bent capacity = 3 * 33 k = 99 k per bent

As discussed in section 2.4.2, the maximum longitudinal load that a span may be subjected to without causing bent failure is a function of both the bent capacity and the loading of adjacent spans. Similar to the I-10 Lake Ponchartrain Bridge, it will be assumed that bents restrain one half of the longitudinal load applied to a span when a single span is loaded. The I-10 Escambia Bay Bridge, however, is different than the Lake Ponchartrain Bridge in that plans indicate that bents are to support either two fixed bearings or two expansion bearings. When adjacent spans are simultaneously and equally longitudinally loaded, bents supporting fixed bearings on the I-10 Escambia Bay Bridge have the potential to be subjected to a load greater than that applied to one span. However, the capacity of the four bolts restraining each span longitudinally is small when compared to the longitudinal friction force that may be applied to each bent. For simplicity, the total load delivered to a fixed bearing bent when spans are simultaneously loaded in the longitudinally direction will be approximated as the load applied to each span.

- Longitudinal capacity
  - Two spans loaded = 99 k per span
  - One span loaded = 198 k per span

3.5 Summary of Force Estimates for I-10 Escambia Bay Bridge

The uplift resistance of the spans was:

601 k based on span dislodgment

930 k to 1,292 k based on negative bending of the beams

From the calculated loads it would be expected that the spans would dislodge before the girders were damaged in negative bending.

Lateral resistance of the bridge was:

127 k to 525 k based on dislodgment

165 k to 330 k per span based bent capacity

The wide range of resistance for span dislodgment under lateral loads is due to frictional resistance. The range of resistance based on bent capacity is due to uncertainty regarding the simultaneous loading of spans. The possibility for both types of failure will be possible depending on the magnitude of uplift loading.

Longitudinal resistance of the bridge was:

64 k to 462 k based on dislodgment

99 k to 198 k per span based on bent capacity

Similar to lateral loading, the wide range of resistance for span dislodgment is due to friction. The range of resistance based on bent capacity is due to uncertainty regarding the simultaneous loading of spans. The possibility for both types of failure will be possible depending on the magnitude of uplift loading.

Figure 3.8 shows the failure envelope for lateral and vertical loads that was developed using linear programming.

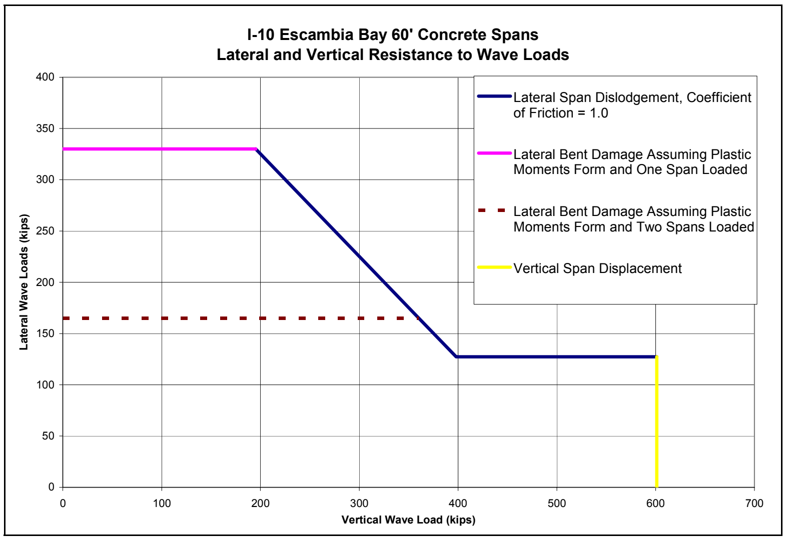


Figure 3.8 – Failure Envelope for Lateral and Vertical Load

Observations of damage reveal that both span dislodgment and bent failure occurred. As seen in Figure 3.9, many failed bents exhibited distress at the pile to bent cap connection in the area where the first plastic moments were expected to form. It is probable that this connection did not develop the full plastic capacity of the piles. If this is true, the horizontal lines in Figure 3.8 would be lower because the calculations, which did not account for this failure mode, overestimated the lateral capacity of the piers.

Figure 3.9 – Damage to a Pile to Bent Cap Connection on Escambia Bay Bridge
Source: FDOT Damage Report

Figure 3.10 shows the failure envelope for longitudinal and vertical loads that was developed using linear programming.

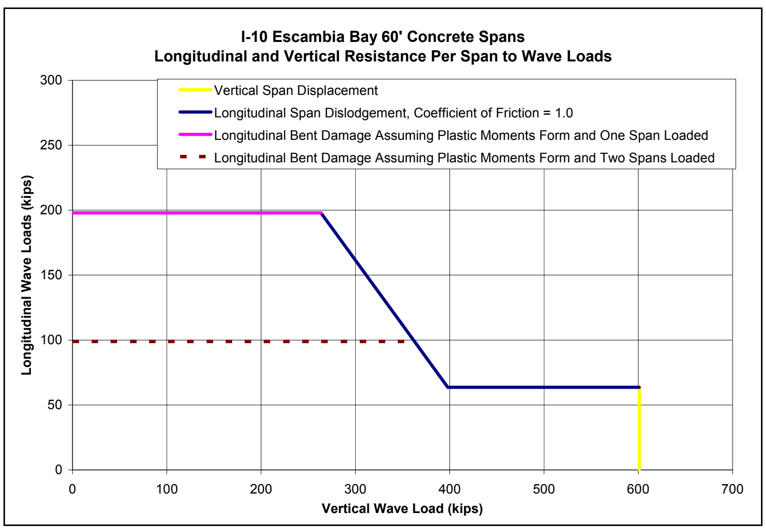


Figure 3.10 – Failure Envelope for Lateral and Vertical Load

Available photographs of damage, such as Figure 3.11 and Figure 3.12, show longitudinal damage to bents and longitudinal displacement of spans. However, it is possible that the observed longitudinal bent damage occurred after the spans were displaced. It is also possible that the fasteners were failed by uplift loading prior to longitudinal displacement of the spans.

Figure 3.11 –Bent Exhibiting Longitudinal Damage on Escambia Bay Bridge
Source: FDOT Damage Report


Figure 3.12 – Longitudinally Displaced Span on Escambia Bay Bridge
Source: FDOT Damage Report

The predicted failure modes are not inconsistent with the observed damage. It was predicted that uplift loads would cause span dislodgment before girders were damaged in negative bending. Calculations for lateral and longitudinal loads predict possible failure by both span dislodgment and bent failure. Observed damage generally consisted of displaced spans and damaged bents.

Similar to the I-10 Lake Ponchartrain Bridge it is believed that the resistance of the bridge to vertical loads was exceeded. This allows the vertical load actually reacted by the structure to be estimated as 601k. The non-occurrence of negative bending damage was a result of insufficient restraint of the superstructure and does exclude the possibility that the seastate could cause vertical loads in excess of the negative bending capacity of the beams.

Given the occurrence of vertical loads in excess of the resistance of the bridge spans, estimates of lateral or longitudinal loads actually reacted based on span displacement may not be made. If a span were lifted from the bents, i.e. uplift load breaks the fasteners and raises the span off of the bent cap, a relatively small horizontal force could result in the observed displacements.

The occurrence of bent damage does provide some indication of the horizontal forces actually reacted by the structure. However, the following points should be noted:

- Damage to bents may have occurred before or after spans dislodged. The sequence of events leading to bent damage is not known with certainty.
- Longitudinal damage to bents may be a result of spans which were displaced longitudinally causing the loss of bearing at one end. A span with one end resting on the bottom of the bay and the other end resting on a bent may exert significant longitudinal force on the bent. The dynamics involved during the movement of the span immediately after bearing is lost is also uncertain and may involve significant longitudinal forces.
- Failure of the pile to bent cap connections was observed in damage photographs. Failure of piles splices was reported in the FDOT damage report . These failures were not considered when estimating bent capacity. Attempts to quantify these failures is difficult due to the many unknowns.
- Missing bents and bents that are substantially underwater are difficult to evaluate and do not confirm or disprove predicted failure modes.
- Damage to bents due to debris and other causes unrelated to wave loads on the spans cannot be ruled out as a source of damage.

The fact that the pile to bent cap connections were damaged indicates that at some point the bents received significant levels of lateral load. Estimation of the lateral load required to cause the observed damage is difficult. Factors including the actual quantity, length, and placement of rebar, the strength and shrinkage of the cast in place concrete, and deterioration of the pile to pier cap connection prior to Hurricane Ivan are not easily determined. The lateral load required to cause the observed connection damage will, however, be (possibly significantly) less than the computed lateral resistance of the pier.

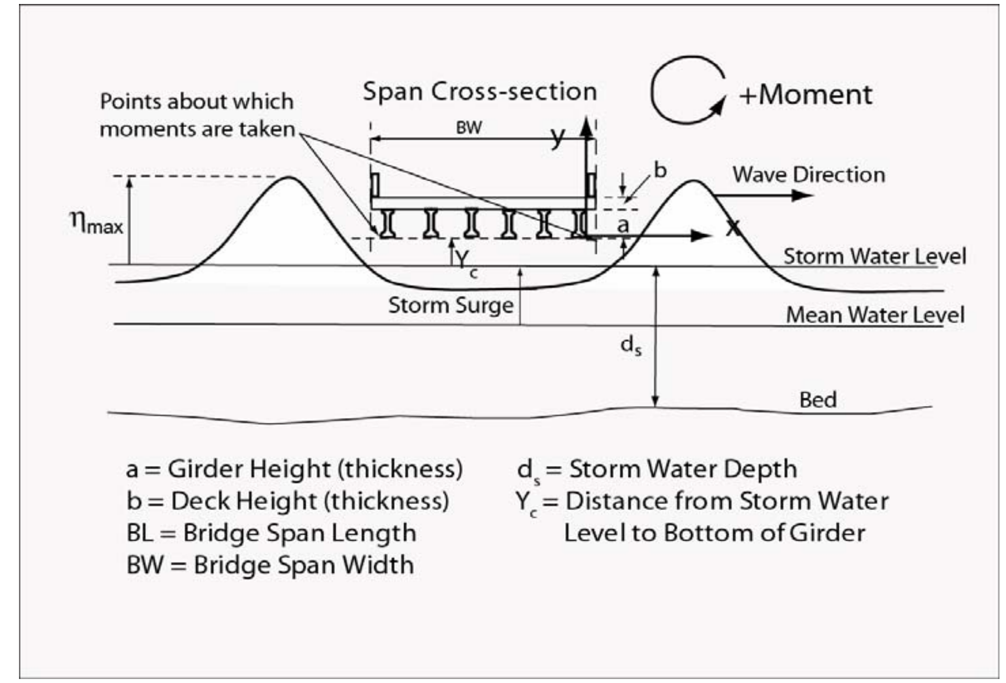
Attachment B

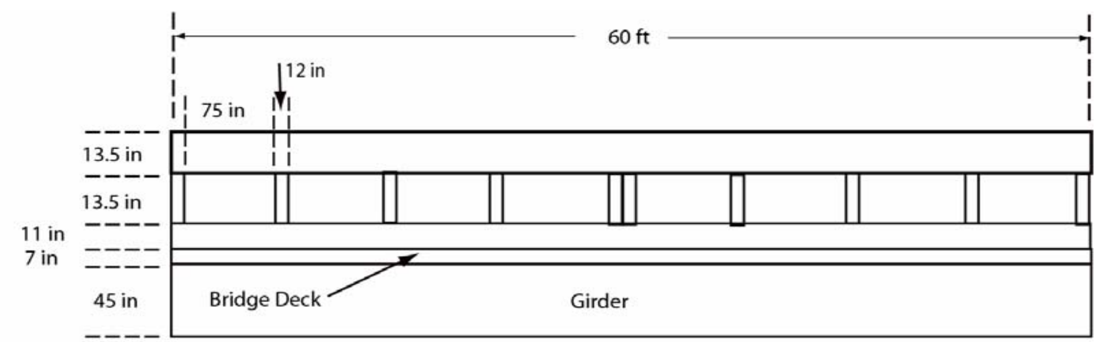
The Results of the Wave Force Calculations
for
One Span of Escambia Bay Bridge
Under Different Wave Conditions

Comment	Case #	d_s (ft)	Significant Wave Height (ft)	Maximum Wave Height (ft)	Wave Period (sec)	Wave Length (ft)	Wave Crest Height* (ft)	Y_c (ft)	Span Width (ft)	Span Length (ft)	b (ft)	a (ft)	Number of Girders
I10-Escambia Bay Span and Conditions	**123**	**37.1**	**4.7**	**8.2**	**4.1**	**92.3**	**4.95**	**1.6**	**35.3**	**60**	**0.58**	**3.75**	**6**
I10-Escambia Bay Span and Conditions	**131**	**39.5**	**4.7**	**8.2**	**4.1**	**92.7**	**4.94**	**-0.3**	**35.3**	**60**	**0.58**	**3.75**	**6**
I10 - Lake Pontchartrain	**W363**	**22.1**	**6.5**	**12.0**	**5.1**	**125.8**	**8.2**	**1.2**	**45.5**	**65**	**0.54**	**3.75**	**6**
I10 - Lake Pontchartrain	**W382**	**21.2**	**6.4**	**11.7**	**5.1**	**124.3**	**8.1**	**-3.8**	**45.5**	**65**	**0.54**	**3.75**	**6**

* Wave crest height above storm water level (h_{max})

I10-Escambia Bay Span Hypothetical Conditions	**1**	**40**	**4.4**	**8**	**3.1**	**58.7**	**5.27**	**3**	**35.3**	**60**	**0.58**	**3.8**	**6**
I10-Escambia Bay Span Hypothetical Conditions	**2**	**40**	**4.4**	**8**	**5**	**128.3**	**4.48**	**3**	**35.3**	**60**	**0.58**	**3.8**	**6**
I10-Escambia Bay Span Hypothetical Conditions	**3**	**40**	**4.4**	**8**	**8**	**254**	**4.55**	**3**	**35.3**	**60**	**0.58**	**3.8**	**6**
I10-Escambia Bay Span Hypothetical Conditions	**4**	**40**	**4.4**	**8**	**3.1**	**58.7**	**5.27**	**0**	**35.3**	**60**	**0.58**	**3.8**	**6**
I10-Escambia Bay Span Hypothetical Conditions	**5**	**40**	**4.4**	**8**	**5**	**128.3**	**4.48**	**0**	**35.3**	**60**	**0.58**	**3.8**	**6**
I10-Escambia Bay Span Hypothetical Conditions	**6**	**40**	**4.4**	**8**	**8**	**254**	**4.55**	**0**	**35.3**	**60**	**0.58**	**3.8**	**6**
I10-Escambia Bay Span Hypothetical Conditions	**7**	**40**	**4.4**	**8**	**3.1**	**58.7**	**5.27**	**-9**	**35.3**	**60**	**0.58**	**3.8**	**6**
I10-Escambia Bay Span Hypothetical Conditions	**8**	**40**	**4.4**	**8**	**5**	**128.3**	**4.48**	**-9**	**35.3**	**60**	**0.58**	**3.8**	**6**
I10-Escambia Bay Span Hypothetical Conditions	**9**	**40**	**4.4**	**8**	**8**	**254**	**4.55**	**-9**	**35.3**	**60**	**0.58**	**3.8**	**6**
I10-Escambia Bay Span Hypothetical Conditions	**10**	**25**	**4.4**	**8**	**3.1**	**58.2**	**5.38**	**3**	**35.3**	**60**	**0.58**	**3.8**	**6**
I10-Escambia Bay Span Hypothetical Conditions	**11**	**25**	**4.4**	**8**	**5**	**118.6**	**4.77**	**3**	**35.3**	**60**	**0.58**	**3.8**	**6**
I10-Escambia Bay Span Hypothetical Conditions	**12**	**25**	**4.4**	**8**	**8**	**217**	**5.16**	**3**	**35.3**	**60**	**0.58**	**3.8**	**6**
I10-Escambia Bay Span Hypothetical Conditions	**13**	**25**	**4.4**	**8**	**3.1**	**58.2**	**5.38**	**0**	**35.3**	**60**	**0.58**	**3.8**	**6**
I10-Escambia Bay Span Hypothetical Conditions	**14**	**25**	**4.4**	**8**	**5**	**118.6**	**4.77**	**0**	**35.3**	**60**	**0.58**	**3.8**	**6**
I10-Escambia Bay Span Hypothetical Conditions	**15**	**25**	**4.4**	**8**	**8**	**217**	**5.16**	**0**	**35.3**	**60**	**0.58**	**3.8**	**6**
I10-Escambia Bay Span Hypothetical Conditions	**16**	**25**	**4.4**	**8**	**3.1**	**58.2**	**5.38**	**-9**	**35.3**	**60**	**0.58**	**3.8**	**6**
I10-Escambia Bay Span Hypothetical Conditions	**17**	**25**	**4.4**	**8**	**5**	**118.6**	**4.77**	**-9**	**35.3**	**60**	**0.58**	**3.8**	**6**
I10-Escambia Bay Span Hypothetical Conditions	**18**	**25**	**4.4**	**8**	**8**	**217**	**5.16**	**-9**	**35.3**	**60**	**0.58**	**3.8**	**6**
I10-Escambia Bay Span Hypothetical Conditions	**19**	**40**	**6.7**	**12**	**4**	**95.1**	**7.66**	**3**	**35.3**	**60**	**0.58**	**3.8**	**6**
I10-Escambia Bay Span Hypothetical Conditions	**20**	**40**	**6.7**	**12**	**6**	**175.1**	**7.06**	**3**	**35.3**	**60**	**0.58**	**3.8**	**6**
I10-Escambia Bay Span Hypothetical Conditions	**21**	**40**	**6.7**	**12**	**8**	**259**	**7.25**	**3**	**35.3**	**60**	**0.58**	**3.8**	**6**
I10-Escambia Bay Span Hypothetical Conditions	**22**	**40**	**6.7**	**12**	**4**	**95.1**	**7.66**	**0**	**35.3**	**60**	**0.58**	**3.8**	**6**
I10-Escambia Bay Span Hypothetical Conditions	**23**	**40**	**6.7**	**12**	**6**	**175.1**	**7.06**	**0**	**35.3**	**60**	**0.58**	**3.8**	**6**
I10-Escambia Bay Span Hypothetical Conditions	**24**	**40**	**6.7**	**12**	**8**	**258.6**	**7.25**	**0**	**35.3**	**60**	**0.58**	**3.8**	**6**
I10-Escambia Bay Span Hypothetical Conditions	**25**	**40**	**6.7**	**12**	**4**	**95.1**	**7.66**	**-9**	**35.3**	**60**	**0.58**	**3.8**	**6**
I10-Escambia Bay Span Hypothetical Conditions	**26**	**40**	**6.7**	**12**	**6**	**175.1**	**7.06**	**-9**	**35.3**	**60**	**0.58**	**3.8**	**6**
I10-Escambia Bay Span Hypothetical Conditions	**27**	**40**	**6.7**	**12**	**8**	**258.6**	**7.25**	**-9**	**35.3**	**60**	**0.58**	**3.8**	**6**
I10-Escambia Bay Span Hypothetical Conditions	**28**	**25**	**5.6**	**10**	**4**	**89.1**	**6.37**	**3**	**35.3**	**60**	**0.58**	**3.8**	**6**
I10-Escambia Bay Span Hypothetical Conditions	**29**	**25**	**5.6**	**10**	**6**	**154.8**	**6.35**	**3**	**35.3**	**60**	**0.58**	**3.8**	**6**
I10-Escambia Bay Span Hypothetical Conditions	**30**	**25**	**5.6**	**10**	**8**	**221.2**	**6.77**	**3**	**35.3**	**60**	**0.58**	**3.8**	**6**
I10-Escambia Bay Span Hypothetical Conditions	**31**	**25**	**5.6**	**10**	**4**	**89.11**	**6.37**	**0**	**35.3**	**60**	**0.58**	**3.8**	**6**
I10-Escambia Bay Span Hypothetical Conditions	**32**	**25**	**5.6**	**10**	**6**	**154.8**	**6.35**	**0**	**35.3**	**60**	**0.58**	**3.8**	**6**
I10-Escambia Bay Span Hypothetical Conditions	**33**	**25**	**5.6**	**10**	**8**	**221.2**	**6.77**	**0**	**35.3**	**60**	**0.58**	**3.8**	**6**
I10-Escambia Bay Span Hypothetical Conditions	**34**	**25**	**5.6**	**10**	**4**	**89.1**	**6.37**	**-9**	**35.3**	**60**	**0.58**	**3.8**	**6**
I10-Escambia Bay Span Hypothetical Conditions	**35**	**25**	**5.6**	**10**	**6**	**154.8**	**6.35**	**-9**	**35.3**	**60**	**0.58**	**3.8**	**6**
I10-Escambia Bay Span Hypothetical Conditions	**36**	**25**	**5.6**	**10**	**8**	**221.2**	**6.77**	**-9**	**35.3**	**60**	**0.58**	**3.8**	**6**





I10 Escambia Bay Bridge Railing
(not to scale)

Wallingford Approach

Comment	Case #	d_s (ft)	Significant Wave Height (ft)	Maximum Wave Height (ft)	Wave Period (sec)	Wave Length (ft)	Wave Crest Height* (ft)	Y_c (ft)	Span Width (ft)	Span Length (ft)	b (ft)	a (ft)	Number of Girders	Vertical Force on Deck (kips)	Upper Limit (kips)	Lateral Force on Deck (kips)	Upper Limit (kips)	Vertical Force on Seaward Girder (kips)	Upper Limit (kips)	Vertical Force on Internal Girder (kips)	Upper Limit (kips)	Lateral Force on Seaward Girder (kips)	Upper Limit (kips)	Lateral Force on Internal Girder (kips)	Upper Limit (kips)	Vertical Impact Load on Deck (kips)	Lateral Impact Load on Deck (kips)	Vertical Impact Load on Seaward Girder (kips)	Lateral Impact Load on Seaward Girder (kips)	Vertical Impact Load on Internal Girder (kips)	Lateral Impact Load on Internal Girder (kips)	Lateral Deck Moment Arm	Lateral Girder Moment Arm	Total Quasi-Static Vertical Force (kips)	Total Quasi-Static Horizontal Force (kips)	Leading Edge Moment (ft-kips)	Trailing Edge Moment (ft-kips)	Total Impact Vertical Force (kips)	Total Impact Horizontal Force (kips)	Leading Edge Moment (ft-kips)	Trailing Edge Moment (ft-kips)
I10-Escambia Bay Span and Conditions	123	37.1	4.7	8.2	4.1	92.3	4.95	1.6	35.3	60	0.58	3.75	6	0	0	0	0	23.7	35.6	24.7	34.6	16.4	32.9	33.8	60.8	0	0	54.6	59.2	64.3	121.7	3.62	1.12	147	185	836	-1250	376	668	5883	-7375
I10-Escambia Bay Span and Conditions	131	39.5	4.7	8.2	4.1	92.7	4.94	-0.3	35.3	60	0.58	3.75	6	333.8	500.6	11.5	23	28.3	42.4	28.8	40.3	18.4	36.8	27.2	48.9	767.7	41.4	65	66.2	74.9	97.8	4.25	1.53	402	166	882	-1451	968	597	16039	-18086
I10 - Lake Pontchartrain	W363	22.1	6.5	12.0	5.1	125.8	8.2	1.2	45.5	65	0.54	3.75	6	769.9	1154.8	28.8	57.5	41.8	62.6	42.6	59.7	32.1	64.1	48.5	87.4	1770.8	103.5	96.1	115.4	110.8	174.8	4.81	1.65	839	303	2035	-3216	1994	1093	43228	-47480
I10 - Lake Pontchartrain	W363	22.1	6.5	12.0	5.1	125.8	8.2	4.2	45.5	65	0.54	3.75	6	283.1	424.7	9.4	18.9	33.6	50.4	35.2	49.3	31.8	63.6	72.9	131.2	651.2	33.9	77.2	114.5	91.6	262.5	3.83	1.32	424	406	2044	-3166	1029	1461	21396	-25435
I10 - Lake Pontchartrain	W382	21.2	6.4	11.7	5.1	124.3	8.1	-0.8	45.5	65	0.54	3.75	6	912.6	1368.9	28	56.1	45.4	68.2	45.8	64.1	29.5	59	37	66.5	2099	101	104.5	106.1	119	133.1	5.00	1.71	967	243	1907	-2920	2292	873	50319	-53965
I10 - Lake Pontchartrain	W382	21.2	6.4	11.7	5.1	124.3	8.1	-3.8	45.5	65	0.54	3.75	6	1091.5	1637.3	24.8	49.6	50.9	76.3	50.5	70.7	26.7	53.5	27	48.7	2510.5	89.2	117	96.3	131.4	97.3	5.09	1.76	1132	187	1780	-2601	2679	672	59462	-62418

* Wave crest height above storm water level (η_{max})

I10-Escambia Bay Span Hypothetical Conditions	1	40	4.4	8	3.1	58.7	5.27	3	35.3	60	0.58	3.75	6	0	0	0	0	19.6	29.4	20.7	29	12.5	25	32.6	58.8	0	0	45	45	53.9	117.5	3.29	0.76	123	176	660	-926	315	633	5064	-6022
I10-Escambia Bay Span Hypothetical Conditions	2	40	4.4	8	5	128.3	4.48	3	35.3	60	0.58	3.75	6	0	0	0	0	16.6	24.9	17.9	25.1	10.4	20.7	37.1	66.8	0	0	38.1	37.3	46.6	113.6	3.03	0.49	106	196	561	-754	271	605	4480	-5077
I10-Escambia Bay Span Hypothetical Conditions	3	40	4.4	8	8	254	4.55	3	35.3	60	0.58	3.75	6	0	0	0	0	16.9	25.3	18.2	25.5	10.6	21.1	36.6	65.9	0	0	38.8	38	47.4	131.8	3.05	0.52	108	194	570	-770	276	697	4501	-5221
I10-Escambia Bay Span Hypothetical Conditions	4	40	4.4	8	3.1	58.7	5.27	0	35.3	60	0.58	3.75	6	318.9	478.4	10.3	20.7	27.2	40.8	27.6	38.7	16.7	33.4	23.4	42.1	733.5	37.2	62.6	60.1	71.8	84.2	4.29	1.54	385	144	809	-1310	927	518	15430	-17234
I10-Escambia Bay Span Hypothetical Conditions	5	40	4.4	8	5	128.3	4.48	0	35.3	60	0.58	3.75	6	236.1	354.2	7.4	14.7	25.5	38.3	26.1	36.6	16.5	32.9	26	46.8	543	26.5	58.7	59.3	67.9	93.6	4.03	1.43	319	154	805	-1285	772	554	12745	-14470
I10-Escambia Bay Span Hypothetical Conditions	6	40	4.4	8	8	254	4.55	0	35.3	60	0.58	3.75	6	245.3	368	7.7	15.4	25.7	38.5	26.3	36.8	16.5	33	25.8	46.4	564.2	27.6	59.1	59.5	68.3	92.8	4.05	1.45	326	153	807	-1290	789	551	13038	-14776
I10-Escambia Bay Span Hypothetical Conditions	7	40	4.4	8	3.1	58.7	5.27	-9	35.3	60	0.58	3.75	6	685.8	1028.7	10.7	21.5	40.1	60.2	38.7	54.2	13	25.9	8.7	15.6	1577.4	38.7	92.3	46.7	100.7	31.3	5.05	1.80	706	67	667	-979	1682	242	29081	-30205
I10-Escambia Bay Span Hypothetical Conditions	8	40	4.4	8	5	128.3	4.48	-9	35.3	60	0.58	3.75	6	665.1	997.7	11.1	22.2	39.2	58.9	38	53.2	13.3	26.5	9.3	16.7	1529.8	39.9	90.2	47.7	98.8	33.4	5.04	1.80	687	71	677	-1004	1637	255	28274	-29448
I10-Escambia Bay Span Hypothetical Conditions	9	40	4.4	8	8	254	4.55	-9	35.3	60	0.58	3.75	6	667	1000.5	11.1	22.1	39.3	59	38.1	53.3	13.2	26.5	9.2	16.6	1534.1	39.8	90.4	47.6	99	33.2	5.04	1.80	689	70	677	-1001	1642	253	28349	-29519
I10-Escambia Bay Span Hypothetical Conditions	10	25	4.4	8	3.1	58.2	5.38	3	35.3	60	0.58	3.75	6	0	0	0	0	20	29.9	21.1	29.5	12.8	25.5	32.2	57.9	0	0	45.9	46	54.8	115.9	3.33	0.79	126	174	673	-949	320	626	5142	-6134
I10-Escambia Bay Span Hypothetical Conditions	11	25	4.4	8	5	118.6	4.77	3	35.3	60	0.58	3.75	6	0	0	0	0	17.8	26.7	19.1	26.7	11.2	22.4	35.2	63.3	0	0	40.9	40.3	49.5	126.6	3.12	0.59	113	187	600	-821	288	673	4686	-5480
I10-Escambia Bay Span Hypothetical Conditions	12	25	4.4	8	8	217	5.16	3	35.3	60	0.58	3.75	6	0	0	0	0	19.2	28.8	20.4	28.5	12.2	24.5	33.1	59.6	0	0	44.2	44	53	119.3	3.25	0.72	121	178	648	-903	309	641	4988	-5911
I10-Escambia Bay Span Hypothetical Conditions	13	25	4.4	8	3.1	58.2	5.38	0	35.3	60	0.58	3.75	6	328	492	10.7	21.3	27.4	41.1	27.8	38.9	16.7	33.4	23	41.4	754.4	38.4	63.1	60.1	72.3	82.9	4.33	1.55	392	142	808	-1310	944	513	15735	-17542
I10-Escambia Bay Span Hypothetical Conditions	14	25	4.4	8	5	118.6	4.77	0	35.3	60	0.58	3.75	6	271.2	406.8	8.6	17.2	26.2	39.2	26.7	37.4	16.6	33.2	25	45.1	623.7	31	60.2	59.8	69.4	90.2	4.12	1.48	346	150	809	-1299	837	542	13861	-15630
I10-Escambia Bay Span Hypothetical Conditions	15	25	4.4	8	8	217	5.16	0	35.3	60	0.58	3.75	6	309.4	464.1	10	20	27	40.5	27.4	38.4	16.7	33.4	23.7	42.7	711.6	35.9	62.1	60.1	72.3	85.5	4.25	1.53	377	145	810	-1309	914	524	15202	-17000
I10-Escambia Bay Span Hypothetical Conditions	16	25	4.4	8	3.1	58.2	5.38	-9	35.3	60	0.58	3.75	6	688.6	1032.9	10.7	21.4	40.2	60.4	38.8	54.4	12.9	25.8	8.6	15.5	1583.8	38.5	92.5	46.5	101	31	5.05	1.80	708	67	665	-974	1688	240	29192	-30308
I10-Escambia Bay Span Hypothetical Conditions	17	25	4.4	8	5	118.6	4.77	-9	35.3	60	0.58	3.75	6	672.8	1009.3	11	21.9	39.6	59.3	38.3	53.6	13.1	26.3	9	16.3	1547.6	39.5	91	47.3	99.5	32.6	5.05	1.80	694	69	674	-994	1654	250	28575	-29730
I10-Escambia Bay Span Hypothetical Conditions	18	25	4.4	8	8	217	5.16	-9	35.3	60	0.58	3.75	6	683	1024.5	10.8	21.6	40	60	38.6	54.1	13	26	8.8	15.8	1570.9	38.8	92	46.8	100.5	31.5	5.05	1.80	703	68	668	-982	1676	243	28977	-30105
I10-Escambia Bay Span Hypothetical Conditions	19	40	6.7	12	4	95.1	7.66	3	35.3	60	0.58	3.75	6	334.4	501.6	15.7	31.4	33.5	50.3	35	48.9	31.9	63.9	66.8	120.3	769.4	56.6	77.1	115	90.9	240.6	4.09	1.46	439	382	1427	-2627	1061	1375	16545	-20867
I10-Escambia Bay Span Hypothetical Conditions	20	40	6.7	12	6	175.1	7.06	3	35.3	60	0.58	3.75	6	209.7	314.6	9.3	18.6	31.8	47.6	33.4	46.7	31	62	71.8	129.3	482.4	33.4	73	111.5	86.7	258.6	3.89	1.34	343	399	1405	-2525	839	1438	12765	-16797
I10-Escambia Bay Span Hypothetical Conditions	21	40	6.7	12	8	259	7.25	3	35.3	60	0.58	3.75	6	259.8	389.6	11.8	23.6	32.3	48.5	33.9	47.4	31.4	62.8	70.3	126.6	597.5	42.6	74.4	113	88.1	253.2	3.95	1.39	381	395	1414	-2570	926	1422	14243	-18408
I10-Escambia Bay Span Hypothetical Conditions	22	40	6.7	12	4	95.1	7.66	0	35.3	60	0.58	3.75	6	600.6	900.9	27.3	54.6	40.7	61	41.4	57.9	30.7	61.4	44.5	80.1	1381.4	98.3	93.6	110.5	107.6	160.1	4.89	1.69	661	281	1386	-2509	1583	1009	25874	-29916
I10-Escambia Bay Span Hypothetical Conditions	23	40	6.7	12	6	175.1	7.06	0	35.3	60	0.58	3.75	6	562.3	843.5	27.2	54.4	39.4	59.1	40.3	56.4	31.2	62.5	48.1	86.5	1293.3	97.8	90.6	112.4	104.7	173	4.82	1.67	628	299	1397	-2565	1505	1075	24417	-28618
I10-Escambia Bay Span Hypothetical Conditions	24	40	6.7	12	8	258.6	7.25	0	35.3	60	0.58	3.75	6	574.9	862.3	27.3	54.6	39.8	59.7	40.6	56.9	31.1	62.1	46.9	84.4	1322.2	98.3	91.6	111.8	105.6	168.8	4.84	1.68	639	293	1393	-2548	1530	1054	24888	-29042
I10-Escambia Bay Span Hypothetical Conditions	25	40	6.7	12	4	95.1	7.66	-9	35.3	60	0.58	3.75	6	960.4	1440.5	18.9	37.9	55.1	82.6	53.9	75.5	23.4	46.8	19.1	34.4	2208.8	68.2	126.7	84.3	140.1	68.7	5.06	1.82	986	138	1174	-1798	2348	496	40260	-42506
I10-Escambia Bay Span Hypothetical Conditions	26	40	6.7	12	6	175.1	7.06	-9	35.3	60	0.58	3.75	6	942.6	1413.9	19.4	38.7	54.3	81.4	53.2	74.5	23.8	47.5	19.9	35.9	2168	69.7	124.9	85.6	138.4	71.7	5.06	1.82	969	143	1187	-1831	2310	514	39548	-41865
I10-Escambia Bay Span Hypothetical Conditions	27	40	6.7	12	8	258.6	7.25	-9	35.3	60	0.58	3.75	6	948.3	1422.4	19.2	38.4	54.5	81.8	53.5	74.8	23.7	47.3	19.6	35.4	2181.1	69.2	125.4	85.2	139	70.7	5.06	1.82	975	141	1183	-1820	2322	508	39780	-42074
I10-Escambia Bay Span Hypothetical Conditions	28	25	5.6	10	4	89.1	6.37	3	35.3	60	0.58	3.75	6	0	0	0	0	26.5	39.7	27.8	38.9	21.7	43.3	50.5	90.9	0	0	60.9	78	72.3	181.8	3.66	1.12	166	274	1067	-1683	422	987	6336	-8553
I10-Escambia Bay Span Hypothetical Conditions	29	25	5.6	10	6	154.8	6.35	3	35.3	60	0.58	3.75	6	0	0	0	0	26.4	39.6	27.8	38.9	21.6	43.2	50.6	91	0	0	60.7	77.8	72.2	182.1	3.65	1.12	165	275	1064	-1678	422	988	6329	-8536
I10-Escambia Bay Span Hypothetical Conditions	30	25	5.6	10	8	221.2	6.77	3	35.3	60	0.58	3.75	6	0	0	0	0	27.7	41.5	28.9	40.5	22.8	45.5	48.8	87.9	0	0	63.6	82	75.1	175.7	3.79	1.26	172	267	1110	-1781	439	961	6532	-8946
I10-Escambia Bay Span Hypothetical Conditions	31	25	5.6	10	4	89.11	6.37	0	35.3	60	0.58	3.75	6	459.4	689.1	19.2	38.5	33.9	50.9	34.5	48.4	24	48	34.9	62.9	1056.6	69.2	78	86.4	89.8	125.7	4.66	1.63	523	218	1110	-1936	1254	784	20622	-23598
I10-Escambia Bay Span Hypothetical Conditions	32	25	5.6	10	6	154.8	6.35	0	35.3	60	0.58	3.75	6	458	687	19.2	38.3	33.9	50.8	34.5	48.3	24	48	35	63	1053.4	69	77.9	86.5	89.7	126.1	4.65	1.63	522	218	1111	-1938	1252	786	20571	-23550
I10-Escambia Bay Span Hypothetical Conditions	33	25	5.6	10	8	221.2	6.77	0	35.3	60	0.58	3.75	6	486.1	729.1	20.2	40.4	34.7	52.1	35.3	49.4	23.8	47.6	33.1	59.5	1117.9	72.8	79.9	85.6	91.7	119.1	4.77	1.65	546	210	1100	-1918	1308	754	21582	-24528
I10-Escambia Bay Span Hypothetical Conditions	34	25	5.6	10	4	89.1	6.37	-9	35.3	60	0.58	3.75	6	826.1	1239.1	15	30	47.8	71.8	46.6	65.2	18.3	36.6	13.9	25	1890	54	110	65.9	121.1	50	5.06	1.81	850	103	927	-1396	2017	370	34701	-36391
I10-Escambia Bay Span Hypothetical Conditions	35	25	5.6	10	6	154.8	6.35	-9	35.3	60	0.58	3.75	6	825.5	1238.3	15	30	47.8	71.7	46.6	65.2	18.3	36.6	13.9	25	1898.7	54	110	65.9	121.1	50	5.06	1.81	849	103	927	-1396	2023	370	34806	-36497
I10-Escambia Bay Span Hypothetical Conditions	36	25	5.6	10	8	221.2	6.77	-9	35.3	60	0.58	3.75	6	837.1	1255.6	14.8	29.6	48.3	72.5	47	65.8	18.1	36.2	13.4	24.2	1925.3	53.2	111.1	65.2	122.2	48.5	5.06	1.81	860	100	920	-1378	2048	361	35264	-36918

I10-Escambia Bay Span Hypothetical Conditions	101	40	4.4	8	3.1	58.7	5.27	3	35.3	60	0.58	3.75	6	0	0	0	0	19.6	29.4	20.7	29	12.5	25	32.6	58.8	0	0	45	45	53.9	117.5	3.29	0.76	123	176	660	-926	315	633	5064	-6022
I10-Escambia Bay Span Hypothetical Conditions	102	40	4.4	8	3.1	58.7	5.27	2	35.3	60	0.58	3.75	6	0	0	0	0	22.6	33.9	23.5	32.9	14.7	29.4	29.3	52.7	0	0	51.9	52.8	61	105.3	3.62	1.09	140	161	755	-1106	357	579	5659	-6922
I10-Escambia Bay Span Hypothetical Conditions	103	40	4.4	8	3.1	58.7	5.27	1	35.3	60	0.58	3.75	6	204.4	306.6	6.3	12.5	25.1	37.6	25.7	36	16.3	32.6	26.7	48	470.2	22.5	57.6	58.7	66.8	96	3.96	1.39	294	156	801	-1268	715	561	11769	-13447
I10-Escambia Bay Span Hypothetical Conditions	104	40	4.4	8	3.1	58.7	5.27	0	35.3	60	0.58	3.75	6	318.9	478.4	10.3	20.7	27.2	40.8	27.6	38.7	16.7	33.4	23.4	42.1	733.5	37.2	62.6	60.1	71.8	84.2	4.29	1.54	385	144	809	-1310	927	518	15430	-17234
I10-Escambia Bay Span Hypothetical Conditions	105	40	4.4	8	3.1	58.7	5.27	-1	35.3	60	0.58	3.75	6	390.5	585.7	13	25.9	29.1	43.7	29.3	41	16.5	33	20.3	36.6	898.1	46.7	67	59.5	76.2	73.2	4.62	1.63	444	131	797	-1301	1066	472	17887	-19702
I10-Escambia Bay Span Hypothetical Conditions	106	40	4.4	8	3.1	58.7	5.27	-2	35.3	60	0.58	3.75	6	445.8	668.7	14.2	28.3	30.8	46.3	30.8	43.1	16.1	32.2	17.8	32	1025.4	51	70.9	58	80.1	64	4.85	1.68	492	119	777	-1267	1177	429	19871	-21633
I10-Escambia Bay Span Hypothetical Conditions	107	40	4.4	8	3.1	58.7	5.27	-3	35.3	60	0.58	3.75	6	492.1	738.1	14	27.9	32.4	48.6	32.2	45.1	15.6	31.3	15.7	28.8	1131.8	50.3	75.6	56	83.7	56.4	4.93	1.71	532	108	757	-1217	1273	388	21617	-23269
I10-Escambia Bay Span Hypothetical Conditions	108	40	4.4	8	3.1	58.7	5.27	-4	35.3	60	0.58	3.75	6	532.4	798.6	13.4	26.9	33.9	50.9	33.5	46.8	15.1	30.3	14	25.1	1224.5	48.3	78	54.5	87	50.2	4.97	1.74	568	99	746	-1175	1356	354	23130	-24672
I10-Escambia Bay Span Hypothetical Conditions	109	40	4.4	8	3.1	58.7	5.27	-5	35.3	60	0.58	3.75	6	568.4	852.6	12.8	25.6	35.3	52.9	34.6	48.5	14.6	29.3	12.5	22.5	1307.4	16.2	81.1	52.7	90.1	45.1	5.00	1.76	600	90	729	-1128	1432	294	24665	-25804
I10-Escambia Bay Span Hypothetical Conditions	110	40	4.4	8	3.1	58.7	5.27	-6	35.3	60	0.58	3.75	6	601.2	901.8	12.2	24.5	36.6	54.9	35.8	50.1	14.2	28.4	11.3	20.4	1382.8	44	84.2	51.1	93	40.7	5.02	1.77	630	83	714	-1087	1501	299	25788	-27132
I10-Escambia Bay Span Hypothetical Conditions	111	40	4.4	8	3.1	58.7	5.27	-7	35.3	60	0.58	3.75	6	631.4	947.1	11.7	23.4	37.8	56.7	36.8	51.5	13.8	27.5	10.3	18.5	1452.2	42.1	87	49.5	95.7	37.1	5.03	1.78	657	77	697	-1048	1565	277	26958	-28220
I10-Escambia Bay Span Hypothetical Conditions	112	40	4.4	8	3.1	58.7	5.27	-8	35.3	60	0.58	3.75	6	659.5	989.2	11.2	22.4	39	58.5	37.8	52.9	13.3	26.7	9.4	17	1516.8	40.3	89.7	48	98.3	34	5.04	1.79	682	72	681	-1011	1626	258	28056	-29244
I10-Escambia Bay Span Hypothetical Conditions	113	40	4.4	8	3.1	58.7	5.27	-9	35.3	60	0.58	3.75	6	685.8	1028.7	10.7	21.5	40.1	60.2	38.7	54.2	13	25.9	8.7	15.6	1577.4	38.7	92.3	46.7	100.7	31.3	5.05	1.80	706	67	667	-979	1682	242	29081	-30205

Comment	Case #	d_s (ft)	Significant Wave Height (ft)	Maximum Wave Height (ft)	Wave Period (sec)	Wave Length (ft)	Wave Crest Height* (ft)	Y_c (ft)	Span Width (ft)	Span Length (ft)	b (ft)	a (ft)	Number of Girders
I10-Escambia Bay Span and Conditions	123	37.1	4.7	8.2	4.1	92.3	4.95	1.6	35.3	60	0.58	3.75	6
I10-Escambia Bay Span and Conditions	131	39.5	4.7	8.2	4.1	92.7	4.94	-0.3	35.3	60	0.58	3.75	6
I10 - Lake Pontchartrain	W363	22.1	6.5	12.0	5.1	125.8	8.2	1.2	45.5	65	0.54	3.75	6
I10 - Lake Pontchartrain	W363	22.1	6.5	12.0	5.1	125.8	8.2	4.2	45.5	65	0.54	3.75	6
I10 - Lake Pontchartrain	W382	21.2	6.4	11.7	5.1	124.3	8.1	-0.8	45.5	65	0.54	3.75	6
I10 - Lake Pontchartrain	W382	21.2	6.4	11.7	5.1	124.3	8.1	-3.8	45.5	65	0.54	3.75	6
I10-Escambia Bay Span Hypothetical Conditions	1	40	4.4	8	3.1	58.7	5.27	3	35.3	60	0.58	3.75	6
I10-Escambia Bay Span Hypothetical Conditions	2	40	4.4	8	5	128.3	4.48	3	35.3	60	0.58	3.75	6
I10-Escambia Bay Span Hypothetical Conditions	3	40	4.4	8	8	254	4.55	3	35.3	60	0.58	3.75	6
I10-Escambia Bay Span Hypothetical Conditions	4	40	4.4	8	3.1	58.7	5.27	0	35.3	60	0.58	3.75	6
I10-Escambia Bay Span Hypothetical Conditions	5	40	4.4	8	5	128.3	4.48	0	35.3	60	0.58	3.75	6
I10-Escambia Bay Span Hypothetical Conditions	6	40	4.4	8	8	254	4.55	0	35.3	60	0.58	3.75	6
I10-Escambia Bay Span Hypothetical Conditions	7	40	4.4	8	3.1	58.7	5.27	-9	35.3	60	0.58	3.75	6
I10-Escambia Bay Span Hypothetical Conditions	8	40	4.4	8	5	128.3	4.48	-9	35.3	60	0.58	3.75	6
I10-Escambia Bay Span Hypothetical Conditions	9	40	4.4	8	8	254	4.55	-9	35.3	60	0.58	3.75	6
I10-Escambia Bay Span Hypothetical Conditions	10	25	4.4	8	3.1	58.2	5.38	3	35.3	60	0.58	3.75	6
I10-Escambia Bay Span Hypothetical Conditions	11	25	4.4	8	5	118.6	4.77	3	35.3	60	0.58	3.75	6
I10-Escambia Bay Span Hypothetical Conditions	12	25	4.4	8	8	217	5.16	3	35.3	60	0.58	3.75	6
I10-Escambia Bay Span Hypothetical Conditions	13	25	4.4	8	3.1	58.2	5.38	0	35.3	60	0.58	3.75	6
I10-Escambia Bay Span Hypothetical Conditions	14	25	4.4	8	5	118.6	4.77	0	35.3	60	0.58	3.75	6
I10-Escambia Bay Span Hypothetical Conditions	15	25	4.4	8	8	217	5.16	0	35.3	60	0.58	3.75	6
I10-Escambia Bay Span Hypothetical Conditions	16	25	4.4	8	3.1	58.2	5.38	-9	35.3	60	0.58	3.75	6
I10-Escambia Bay Span Hypothetical Conditions	17	25	4.4	8	5	118.6	4.77	-9	35.3	60	0.58	3.75	6
I10-Escambia Bay Span Hypothetical Conditions	18	25	4.4	8	8	217	5.16	-9	35.3	60	0.58	3.75	6
I10-Escambia Bay Span Hypothetical Conditions	19	40	6.7	12	4	95.1	7.66	3	35.3	60	0.58	3.75	6
I10-Escambia Bay Span Hypothetical Conditions	20	40	6.7	12	6	175.1	7.06	3	35.3	60	0.58	3.75	6
I10-Escambia Bay Span Hypothetical Conditions	21	40	6.7	12	8	259	7.25	3	35.3	60	0.58	3.75	6
I10-Escambia Bay Span Hypothetical Conditions	22	40	6.7	12	4	95.1	7.66	0	35.3	60	0.58	3.75	6
I10-Escambia Bay Span Hypothetical Conditions	23	40	6.7	12	6	175.1	7.06	0	35.3	60	0.58	3.75	6
I10-Escambia Bay Span Hypothetical Conditions	24	40	6.7	12	8	258.6	7.25	0	35.3	60	0.58	3.75	6
I10-Escambia Bay Span Hypothetical Conditions	25	40	6.7	12	4	95.1	7.66	-9	35.3	60	0.58	3.75	6
I10-Escambia Bay Span Hypothetical Conditions	26	40	6.7	12	6	175.1	7.06	-9	35.3	60	0.58	3.75	6
I10-Escambia Bay Span Hypothetical Conditions	27	40	6.7	12	8	258.6	7.25	-9	35.3	60	0.58	3.75	6
I10-Escambia Bay Span Hypothetical Conditions	28	25	5.6	10	4	89.1	6.37	3	35.3	60	0.58	3.75	6
I10-Escambia Bay Span Hypothetical Conditions	29	25	5.6	10	6	154.8	6.35	3	35.3	60	0.58	3.75	6
I10-Escambia Bay Span Hypothetical Conditions	30	25	5.6	10	8	221.2	6.77	3	35.3	60	0.58	3.75	6
I10-Escambia Bay Span Hypothetical Conditions	31	25	5.6	10	4	89.11	6.37	0	35.3	60	0.58	3.75	6
I10-Escambia Bay Span Hypothetical Conditions	32	25	5.6	10	6	154.8	6.35	0	35.3	60	0.58	3.75	6
I10-Escambia Bay Span Hypothetical Conditions	33	25	5.6	10	8	221.2	6.77	0	35.3	60	0.58	3.75	6
I10-Escambia Bay Span Hypothetical Conditions	34	25	5.6	10	4	89.1	6.37	-9	35.3	60	0.58	3.75	6
I10-Escambia Bay Span Hypothetical Conditions	35	25	5.6	10	6	154.8	6.35	-9	35.3	60	0.58	3.75	6
I10-Escambia Bay Span Hypothetical Conditions	36	25	5.6	10	8	221.2	6.77	-9	35.3	60	0.58	3.75	6
I10-Escambia Bay Span Hypothetical Conditions	101	40	4.4	8	3.1	58.7	5.27	3	35.3	60	0.58	3.75	6
I10-Escambia Bay Span Hypothetical Conditions	102	40	4.4	8	3.1	58.7	5.27	2	35.3	60	0.58	3.75	6
I10-Escambia Bay Span Hypothetical Conditions	103	40	4.4	8	3.1	58.7	5.27	1	35.3	60	0.58	3.75	6
I10-Escambia Bay Span Hypothetical Conditions	104	40	4.4	8	3.1	58.7	5.27	0	35.3	60	0.58	3.75	6
I10-Escambia Bay Span Hypothetical Conditions	105	40	4.4	8	3.1	58.7	5.27	-1	35.3	60	0.58	3.75	6
I10-Escambia Bay Span Hypothetical Conditions	106	40	4.4	8	3.1	58.7	5.27	-2	35.3	60	0.58	3.75	6
I10-Escambia Bay Span Hypothetical Conditions	107	40	4.4	8	3.1	58.7	5.27	-3	35.3	60	0.58	3.75	6
I10-Escambia Bay Span Hypothetical Conditions	108	40	4.4	8	3.1	58.7	5.27	-4	35.3	60	0.58	3.75	6
I10-Escambia Bay Span Hypothetical Conditions	109	40	4.4	8	3.1	58.7	5.27	-5	35.3	60	0.58	3.75	6
I10-Escambia Bay Span Hypothetical Conditions	110	40	4.4	8	3.1	58.7	5.27	-6	35.3	60	0.58	3.75	6
I10-Escambia Bay Span Hypothetical Conditions	111	40	4.4	8	3.1	58.7	5.27	-7	35.3	60	0.58	3.75	6
I10-Escambia Bay Span Hypothetical Conditions	112	40	4.4	8	3.1	58.7	5.27	-8	35.3	60	0.58	3.75	6
I10-Escambia Bay Span Hypothetical Conditions	113	40	4.4	8	3.1	58.7	5.27	-9	35.3	60	0.58	3.75	6

* Wave crest height above storm water level (η_{max})

New Wallingford Approach

Case #	Vertical Force on Deck - External (kips)	Vertical Force on Deck - Internal (kips)	Lateral Force on Deck - External (kips)	Vertical Force on Seaward Girder (kips)	Vertical Force on Internal Girder (kips)	Lateral Force on Seaward Girder (kips)	Lateral Force on Internal Girder (kips)	Vertical Impact Load on Deck - External (kips)	Vertical Impact Load on Deck - Internal (kips)	Lateral Impact Load on Deck (kips)	Vertical Impact Load on Seaward Girder (kips)	Lateral Impact Load on Seaward Girder (kips)	Vertical Impact Load on Internal Girder (kips)	Lateral Impact Load on Internal Girder (kips)	Lateral Deck Moment Arm	Lateral Girder Moment Arm
123	0	0	0	18.5	19.2	32.5	48.4	0	0	0	42	80	50	162	3.62	1.12
131	369	502.8	12.8	20	21.2	39.8	55.8	819	1151	31	46	97	55	187	4.25	1.11
W363	923.8	1182.2	47	40	44.1	81.8	94	2051	2707	115	91	200	114	315	4.66	1.05
W363	661.6	920	3	32.6	34.9	65.4	86.3	1469	2107	7	74	160	91	289	3.83	1.21
W382	1091.9	1346.3	55	44.9	50.2	92.8	98.4	2424	3083	135	102	227	130	329	4.56	1.02
W382	1361.1	1615.5	67.9	52.5	59.6	109.6	106.3	3022	3699	166	120	269	154	356	4.52	1.00
1	0	0	0	16.2	16.6	19.1	30	0	0	0	37	47	43	100	3.29	0.76
2	0	0	0	15.5	15.8	11.9	19.3	0	0	0	35	29	41	65	3.03	0.49
3	0	0	0	15.5	15.9	12.5	20.2	0	0	0	35	31	41	68	3.05	0.52
4	344.6	469.8	11.8	18.7	19.8	37.7	52.9	765	1076	29	43	92	51	177	4.29	1.13
5	326.1	451.3	5.2	18	18.9	36.2	52.2	724	1034	13	41	89	49	175	4.03	1.18
6	327.7	453	5.7	18.1	19	36.3	52.2	727	1037	14	41	89	49	175	4.05	1.18
7	555.2	680.5	32.9	26.4	29.2	54.9	61	1233	1558	81	60	134	76	204	4.49	1.00
8	536.8	662	31.8	25.7	28.4	53.4	60.6	1192	1516	78	59	131	74	202	4.49	1.00
9	538.4	663.6	31.9	25.7	28.5	53.5	60.3	1195	1520	78	59	131	74	202	4.49	1.00
10	0	0	0	17.5	18.2	21.8	32.3	0	0	0	40	54	47	108	3.33	0.79
11	0	0	0	16.6	17.2	15.4	23.6	0	0	0	38	38	45	79	3.12	0.59
12	0	0	0	17.2	17.9	19.4	29.1	0	0	0	39	48	46	98	3.25	0.72
13	369.3	494.6	13.5	21.5	23.3	44.1	55.9	820	1133	33	49	108	60	187	4.33	1.12
14	346.5	471.7	7.8	20.7	22.2	42.2	55	769	1080	19	47	103	58	184	4.12	1.16
15	361.1	486.3	11.4	21.2	22.9	43.4	55.6	802	1114	28	48	106	59	186	4.25	1.14
16	706.4	831.7	41.4	33.8	38.4	71.6	68.8	1568	1905	101	77	175	99	231	4.49	1.00
17	683.6	808.8	40.1	33	37.4	6937	68	1518	1852	98	75	171	97	228	4.49	1.00
18	698.2	823.4	40.9	33.5	38	70.9	68.5	1550	1886	100	76	174	99	230	4.49	1.00
19	502.9	693.6	10.1	27.7	29.1	55.6	79.7	1116	1588	25	63	136	75	267	4.09	1.17
20	481.5	672.3	2.9	26.9	28.2	53.8	78.9	1069	1539	7	61	132	73	264	3.89	1.23
21	488.3	679	5.1	27.2	28.5	54.4	79.1	1084	1555	13	62	133	74	265	3.95	1.21
22	609.9	800.6	36.8	31.6	33.9	64.3	83.8	1354	1833	90	72	158	88	281	4.57	1.05
23	588.5	779.2	35.6	30.8	33	62.6	83	1306	1784	87	70	153	85	278	4.60	1.07
24	595.2	786	36	31	33.3	63.1	83.2	1321	1800	88	71	155	86	279	4.59	1.06
25	930.7	1121.4	54.9	43.2	48.3	90.5	96.1	2066	2568	134	99	222	125	322	4.48	0.99
26	909.3	1100	53.7	42.5	47.4	88.8	95.3	2019	2519	131	97	217	123	319	4.48	0.99
27	916.1	1106.8	54.1	42.7	47.7	89.3	95.6	2034	2535	132	97	219	123	320	4.48	0.99
28	0	0	0	23.9	25.3	42.8	59.8	0	0	0	55	105	66	200	3.66	1.12
29	0	0	0	23.9	25.3	42.5	59.4	0	0	0	54	104	65	199	3.65	1.12
30	0	0	0	24.6	26.2	49.4	67.6	0	0	0	56	121	68	227	3.79	1.26
31	517.2	676.7	30.5	29.1	31.7	59.9	72.9	1148	1550	75	66	147	82	244	4.66	1.09
32	516.3	675.7	30.2	29.1	31.7	59.8	72.9	1146	1547	74	66	147	82	244	4.65	1.09
33	536.3	695.7	32.3	29.8	32.6	61.5	73.7	1191	1593	79	68	151	84	247	4.63	1.07
34	946.3	1105.7	55.4	44.7	51	94.9	89.4	2101	2532	136	102	233	132	300	4.48	1.00
35	945.3	1104.1	55.3	44.7	51	94.8	89.4	2099	2530	136	102	231	132	299	4.48	1.00
36	965.3	1124.8	56.4	45.4	51.9	96.5	90.2	2143	2576	138	104	236	134	302	4.48	0.99
101	0	0	0	18.8	25.3	19.1	30	0	0	0	43	47	66	100	3.29	0.76
102	0	0	0	20	26.5	29.1	44	0	0	0	46	71	69	147	3.62	1.09
103	321.1	446.4	3.5	21.3	27.8	35.8	52	713	1022	9	48	88	72	174	3.96	1.20
104	344.6	469.8	11.8	22.5	29	37.7	52.9	765	1076	29	51	92	75	177	4.29	1.13
105	368	493.2	21	23.7	30.2	39.6	53.8	817	1129	51	54	97	78	180	4.62	1.09
106	391.4	516.6	23.7	24.9	31.4	41.5	54.7	869	1183	58	57	102	81	183	4.59	1.06
107	414.8	540	25	26.1	32.6	43.4	55.6	921	1237	61	60	106	84	186	4.55	1.04
108	438.2	563.4	26.3	27.3	33.8	45.3	56.5	973	1290	64	62	111	88	189	4.53	1.03
109	461.6	586.9	27.6	28.5	35	47.2	57.4	1025	1344	68	65	116	91	192	4.51	1.02
110	485	610.3	28.9	29.8	36.2	49.1	58.3	1077	1398	71	68	120	94	195	4.50	1.01
111	508.4	633.7	30.2	31	37.5	51	59.2	1129	1451	74	71	125	97	198	4.50	1.01
112	531.8	657.1	31.6	32.2	38.7	53	60.1	1181	1505	77	76	130	100	201	4.49	1.00
113	555.2	680.5	32.9	33.4	39.9	54.9	61	1233	1558	81	76	134	103	204	4.49	1.00

Case #	Total Quasi-Static Vertical Force (kips)	Total Quasi-Static Horizontal Force (kips)	Leading Edge Moment (ft-kips)	Trailing Edge Moment (ft-kips)	Total Impact Vertical Force (kips)	Total Impact Horizontal Force (kips)	Leading Edge Moment (ft-kips)	Trailing Edge Moment (ft-kips)
123	115	275	1711	-2325	292	890	4152	-6141
131	472	332	7913	-8731	1113	1063	18346	-20904
W363	1157	599	25532	-27131	2715	1890	59357	-64164
W363	905	500	19976	-21205	2128	1612	46428	-50375
W382	1317	640	29121	-30817	3091	2007	67793	-72848
W382	1576	709	34912	-36803	3696	2215	81301	-86883
1	99	169	1620	-1876	252	547	4027	-4856
2	95	108	1612	-1719	240	354	4055	-4405
3	95	114	1616	-1733	240	371	4038	-4422
4	441	314	7384	-8167	1039	1006	17082	-19537
5	423	302	7087	-7832	998	977	16396	-18781
6	425	303	7118	-7864	1000	978	16433	-18816
7	641	393	10789	-11804	1513	1235	25145	-28178
8	623	388	10489	-11489	1473	1219	24464	-27451
9	625	387	10518	-11516	1476	1219	24513	-27500
10	109	183	1767	-2058	275	594	4376	-5318
11	103	133	1730	-1887	263	433	4380	-4891
12	107	165	1762	-1999	269	538	4354	-5128
13	479	337	8012	-8856	1129	1076	18586	-21214
14	456	325	7645	-8445	1081	1042	17780	-20310
15	471	333	7880	-8706	1110	1064	18269	-20859
16	798	457	13472	-14673	1884	1431	31418	-34979
17	777	7317	6228	-21157	1835	1409	30592	-34096
18	790	454	13335	-14528	1870	1424	31179	-34722
19	651	464	10898	-12041	1531	1496	25170	-28810
20	631	451	10555	-11680	1486	1459	24372	-27996
21	637	455	10668	-11793	1503	1471	24674	-28294
22	752	520	12582	-13937	1774	1653	29209	-33326
23	732	513	12233	-13580	1723	1630	28326	-32420
24	739	515	12345	-13693	1740	1638	28623	-32723
25	1057	626	17814	-19438	2492	1966	41507	-46339
26	1037	619	17473	-19076	2446	1943	40732	-45502
27	1043	621	17582	-19192	2457	1951	40915	-45707
28	150	342	2267	-3035	385	1105	5545	-8027
29	150	340	2272	-3030	379	1099	5452	-7907
30	156	387	2255	-3229	396	1256	5401	-8558
31	654	455	10916	-12121	1543	1442	25366	-29031
32	653	455	10905	-12107	1541	1441	25335	-28990
33	672	462	11229	-12452	1585	1465	26078	-29786
34	1061	597	17912	-19487	2505	1869	41822	-46490
35	1060	597	17893	-19468	2504	1862	41805	-46459
36	1079	604	18227	-19820	2548	1884	42549	-47254
101	145	169	2433	-2689	373	547	6160	-6988
102	153	249	2416	-2959	391	806	6013	-7770
103	468	299	7872	-8612	1112	967	18404	-20782
104	491	314	8261	-9045	1167	1006	19338	-21793
105	514	330	8631	-9497	1221	1048	20205	-22846
106	538	339	9032	-9918	1277	1075	21152	-23845
107	561	346	9437	-10335	1332	1097	22112	-24830
108	584	354	9839	-10754	1390	1120	23123	-25881
109	608	362	10243	-11175	1445	1144	24068	-26881
110	631	370	10646	-11597	1501	1166	25017	-27877
111	655	377	11055	-12027	1555	1189	25951	-28864
112	678	385	11455	-12448	1612	1212	26931	-29899
113	701	393	11856	-12870	1664	1235	27806	-30839

Douglass Approach

Comment	Case #	d_s (ft)	Significant Wave Height (ft)	Maximum Wave Height (ft)	Wave Period (sec)	Wave Length (ft)	Wave Crest Height* (ft)	Y_c (ft)	Span Width (ft)	Span Length (ft)	b (ft)	a (ft)	Number of Girders	Vertical Force on Deck (kips)	Upper Limit (kips)	Lateral Force on Deck (kips)	Upper Limit (kips)	Vertical Force on Girder (kips)	Upper Limit (kips)	Lateral Force on All 6 Girders (kips)	Upper Limit (kips)	Vertical Impact Load on Deck (kips)	Lateral Impact Load on Deck (kips)	Vertical Impact Load on Girder (kips)	Lateral Impact Load on All 6 Girders (kips)	Lateral Deck Moment Arm	Lateral Girder Moment Arm	Total Quasi-Static Vertical Force (kips)	Total Quasi-Static Horizontal Force (kips)	Leading Edge Moment (ft-kips)	Trailing Edge Moment (ft-kips)	Total Impact Vertical Force (kips)	Total Impact Horizontal Force (kips)	Leading Edge Moment (ft-kips)	Trailing Edge Moment (ft-kips)
I10-Escambia Bay Span and Conditions	123	37.1	4.7	8.2	4.1	92.3	4.95	1.6	35.3	60	0.58	3.75	6	0	0	0	0	23.5	47.1	64.6	129.3	0	0	94.2	193.9	3.62	1.12	141	65	2413	-2557	565	194	9745	-10178
I10-Escambia Bay Span and Conditions	131	39.5	4.7	8.2	4.1	92.7	4.94	-0.3	35.3	60	0.58	3.75	6	202	404	4.3	8.5	36.8	73.6	145.4	290.7	807.9	29.8	147.3	436.1	4.25	1.53	360	150	6103	-6583	1440	466	24588	-26173
I10 - Lake Pontchartrain	W363	22.1	6.5	12.0	5.1	125.8	8.2	1.2	45.5	65	0.54	3.75	6	615.2	1230.3	21.7	43.4	53.3	106.6	239.9	479.7	2460.6	151.7	213.2	719.6	4.81	1.65	787	262	17395	-18393	3146	871	69658	-73485
I10 - Lake Pontchartrain	W363	22.1	6.5	12.0	5.1	125.8	8.2	4.2	45.5	65	0.54	3.75	6	47.3	94.6	0.1	0.3	30.5	60.9	99.5	198.9	189.3	0.9	121.8	298.4	3.83	1.32	219	100	4848	-5112	874	299	19494	-20292
I10 - Lake Pontchartrain	W382	21.2	6.4	11.7	5.1	124.3	8.1	-0.8	45.5	65	0.54	3.75	6	974.8	1949.6	44.4	88.8	67.8	135.5	328.8	657.5	3899.2	310.8	271	986.3	5.00	1.71	1146	373	25296	-26863	4584	1297	101052	-107531
I10 - Lake Pontchartrain	W382	21.2	6.4	11.7	5.1	124.3	8.1	-3.8	45.5	65	0.54	3.75	6	1542.6	3085.3	80.3	160.6	90.6	181.2	469.2	938.3	6170.5	562	362.4	1407.5	5.09	1.76	1714	550	37759	-40225	6856	1970	150638	-161303
I10-Escambia Bay Span Hypothetical Conditions	1	40	4.4	8	3.1	58.7	5.27	3	35.3	60	0.58	3.75	6	0	0	0	0	16	31.9	29.7	59.4	0	0	63.8	89	3.29	0.76	96	30	1670	-1714	383	89	6679	-6814
I10-Escambia Bay Span Hypothetical Conditions	2	40	4.4	8	5	128.3	4.48	3	35.3	60	0.58	3.75	6	0	0	0	0	10.4	20.8	12.6	25.2	0	0	41.4	37.9	3.03	0.49	62	13	1094	-1106	248	38	4359	-4397
I10-Escambia Bay Span Hypothetical Conditions	3	40	4.4	8	8	254	4.55	3	35.3	60	0.58	3.75	6	0	0	0	0	10.9	21.8	13.8	27.7	0	0	43.6	41.5	3.05	0.52	65	14	1146	-1160	262	42	4589	-4632
I10-Escambia Bay Span Hypothetical Conditions	4	40	4.4	8	3.1	58.7	5.27	0	35.3	60	0.58	3.75	6	199.3	398.5	4.1	8.3	37	74.1	147.5	295.1	797	29	148.1	442.6	4.29	1.54	359	152	6086	-6576	1437	472	24526	-26141
I10-Escambia Bay Span Hypothetical Conditions	5	40	4.4	8	5	128.3	4.48	0	35.3	60	0.58	3.75	6	92.2	184.4	0.9	1.8	31.5	63	112.9	225.9	368.7	6.2	125.9	338.8	4.03	1.43	252	114	4284	-4615	1009	345	17277	-18299
I10-Escambia Bay Span Hypothetical Conditions	6	40	4.4	8	8	254	4.55	0	35.3	60	0.58	3.75	6	101.7	203.3	1	2.2	32	63.9	116	232	406.7	7.6	127.9	348	4.05	1.45	262	117	4446	-4790	1047	356	17927	-18995
I10-Escambia Bay Span Hypothetical Conditions	7	40	4.4	8	3.1	58.7	5.27	-9	35.3	60	0.58	3.75	6	1419.2	2838.5	92.3	184.6	100.3	200.6	541.5	1083	5676.9	646.2	401.1	1624.5	5.05	1.80	1579	634	26386	-29271	6315	2271	105113	-117498
I10-Escambia Bay Span Hypothetical Conditions	8	40	4.4	8	5	128.3	4.48	-9	35.3	60	0.58	3.75	6	1312.1	2624.3	84.3	168.7	94.7	189.5	506.9	1013.9	5248.6	590.4	378.9	1520.8	5.04	1.80	1472	591	24601	-27272	5887	2111	98051	-109470
I10-Escambia Bay Span Hypothetical Conditions	9	40	4.4	8	8	254	4.55	-9	35.3	60	0.58	3.75	6	1321.6	2643.3	85.1	170.1	95.2	190.4	510	1020	5286.5	595.4	380.9	1530	5.04	1.80	1481	595	24759	-27451	5925	2125	98679	-110184
I10-Escambia Bay Span Hypothetical Conditions	10	25	4.4	8	3.1	58.2	5.38	3	35.3	60	0.58	3.75	6	0	0	0	0	16.7	33.4	32.6	65.3	0	0	66.9	97.9	3.33	0.79	100	33	1740	-1792	401	98	6997	-7152
I10-Escambia Bay Span Hypothetical Conditions	11	25	4.4	8	5	118.6	4.77	3	35.3	60	0.58	3.75	6	0	0	0	0	12.4	24.9	1830	36.1	0	0	49.8	54.1	3.12	0.59	74	1830	232	-2391	299	54	5234	-5298
I10-Escambia Bay Span Hypothetical Conditions	12	25	4.4	8	8	217	5.16	3	35.3	60	0.58	3.75	6	0	0	0	0	15.2	30.4	26.9	53.7	0	0	60.7	80.6	3.25	0.72	91	27	1588	-1627	364	81	6361	-6477
I10-Escambia Bay Span Hypothetical Conditions	13	25	4.4	8	3.1	58.2	5.38	0	35.3	60	0.58	3.75	6	214.2	428.3	4.8	9.6	37.8	75.6	152.3	304.7	856.7	33.6	151.2	457	4.33	1.55	374	157	6339	-6854	1497	491	25530	-27241
I10-Escambia Bay Span Hypothetical Conditions	14	25	4.4	8	5	118.6	4.77	0	35.3	60	0.58	3.75	6	131.5	263	1.8	3.6	33.5	67	125.6	251.3	525.9	12.6	134.1	376.9	4.12	1.48	292	127	4945	-5332	1167	390	19953	-21173
I10-Escambia Bay Span Hypothetical Conditions	15	25	4.4	8	8	217	5.16	0	35.3	60	0.58	3.75	6	184.4	368.7	3.6	7.1	36.3	72.5	142.7	285.4	737.4	24.9	145	428.1	4.25	1.53	345	146	5843	-6310	1378	453	23521	-25043
I10-Escambia Bay Span Hypothetical Conditions	16	25	4.4	8	3.1	58.2	5.38	-9	35.3	60	0.58	3.75	6	1434.1	2868.3	93.4	186.8	101.1	202.1	546.3	1092.6	5736.6	653.9	404.2	1639	5.05	1.80	1594	640	26637	-29551	6375	2293	106098	-118617
I10-Escambia Bay Span Hypothetical Conditions	17	25	4.4	8	5	118.6	4.77	-9	35.3	60	0.58	3.75	6	1351.5	2702.9	87.3	174.5	96.8	193.5	519.6	1039.2	5405.8	610.9	387.1	1558.9	5.05	1.80	1511	607	25262	-28012	6045	2170	100648	-112422
I10-Escambia Bay Span Hypothetical Conditions	18	25	4.4	8	8	217	5.16	-9	35.3	60	0.58	3.75	6	1404.3	2808.6	91.2	182.4	99.5	199	536.7	1073.4	5617.3	638.4	398	1610.1	5.05	1.80	1564	628	26136	-28991	6256	2249	104130	-116379
I10-Escambia Bay Span Hypothetical Conditions	19	40	6.7	12	4	95.1	7.66	3	35.3	60	0.58	3.75	6	116.6	233.1	1.4	2.8	32.7	65.5	120.8	241.6	466.3	9.9	131	362.5	4.09	1.46	276	122	4690	-5056	1107	372	18941	-20083
I10-Escambia Bay Span Hypothetical Conditions	20	40	6.7	12	6	175.1	7.06	3	35.3	60	0.58	3.75	6	35.2	70.5	0.1	0.3	28.5	57.1	94.6	189.1	141	0.9	114.1	283.7	3.89	1.34	195	95	3314	-3568	782	285	13393	-14162
I10-Escambia Bay Span Hypothetical Conditions	21	40	6.7	12	8	259	7.25	3	35.3	60	0.58	3.75	6	61	122	0.4	0.8	29.9	59.7	102.9	205.7	244	2.7	119.5	308.6	3.95	1.39	221	103	3758	-4047	885	311	15159	-16037
I10-Escambia Bay Span Hypothetical Conditions	22	40	6.7	12	4	95.1	7.66	0	35.3	60	0.58	3.75	6	523.2	1046.5	25.7	51.4	53.8	107.7	252.2	504.3	2092.9	179.8	215.3	756.5	4.89	1.69	683	278	11486	-12590	2733	936	46007	-50323
I10-Escambia Bay Span Hypothetical Conditions	23	40	6.7	12	6	175.1	7.06	0	35.3	60	0.58	3.75	6	441.9	883.8	19.6	39.3	49.6	99.2	225.9	451.8	1767.6	137.4	198.4	677.7	4.82	1.67	602	246	10137	-11079	2407	815	40639	-44222
I10-Escambia Bay Span Hypothetical Conditions	24	40	6.7	12	8	258.6	7.25	0	35.3	60	0.58	3.75	6	467.7	935.3	21.5	43.1	50.9	101.9	234.2	468.4	1870.6	150.8	203.8	702.6	4.84	1.68	627	256	10562	-11555	2511	853	42344	-46159
I10-Escambia Bay Span Hypothetical Conditions	25	40	6.7	12	4	95.1	7.66	-9	35.3	60	0.58	3.75	6	1743.2	3486.4	116.4	232.8	117.1	234.1	646.1	1292.3	6972.8	814.8	468.3	1938.4	5.06	1.82	1903	763	31773	-35301	7611	2753	126488	-141787
I10-Escambia Bay Span Hypothetical Conditions	26	40	6.7	12	6	175.1	7.06	-9	35.3	60	0.58	3.75	6	1661.9	3323.7	110.4	220.7	112.8	225.7	619.9	1239.7	6647.5	772.5	451.4	1859.6	5.06	1.82	1821	730	30412	-33779	7285	2632	121119	-135687
I10-Escambia Bay Span Hypothetical Conditions	27	40	6.7	12	8	258.6	7.25	-9	35.3	60	0.58	3.75	6	1687.6	3375.2	112.3	224.5	114.2	228.4	628.2	1256.4	6750.5	785.9	456.8	1884.6	5.06	1.82	1847	741	30847	-34265	7389	2671	122824	-137624
I10-Escambia Bay Span Hypothetical Conditions	28	25	5.6	10	4	89.1	6.37	3	35.3	60	0.58	3.75	6	0	0	0	0	23.7	47.4	65.4	130.8	0	0	94.7	196.2	3.66	1.12	142	65	2433	-2580	568	196	9794	-10235
I10-Escambia Bay Span Hypothetical Conditions	29	25	5.6	10	6	154.8	6.35	3	35.3	60	0.58	3.75	6	0	0	0	0	23.5	47.1	64.6	129.3	0	0	94.2	193.9	3.65	1.12	141	65	2413	-2557	565	194	9745	-10178
I10-Escambia Bay Span Hypothetical Conditions	30	25	5.6	10	8	221.2	6.77	3	35.3	60	0.58	3.75	6	0	0	0	0	26.5	53	81.9	163.7	0	0	106	245.6	3.79	1.26	159	82	2699	-2905	636	246	10901	-11518
I10-Escambia Bay Span Hypothetical Conditions	31	25	5.6	10	4	89.11	6.37	0	35.3	60	0.58	3.75	6	348.4	696.7	12.7	25.4	44.8	89.5	195.7	391.4	1393.5	88.8	179.1	587	4.66	1.63	509	208	8587	-9344	2034	676	34479	-37221
I10-Escambia Bay Span Hypothetical Conditions	32	25	5.6	10	6	154.8	6.35	0	35.3	60	0.58	3.75	6	345.7	691.3	12.5	25	44.6	89.2	194.8	38.6	1382.6	87.4	178.5	584.4	4.65	1.63	506	207	8536	-9287	2023	672	34295	-37013
I10-Escambia Bay Span Hypothetical Conditions	33	25	5.6	10	8	221.2	6.77	0	35.3	60	0.58	3.75	6	402.6	805.2	16.7	33.4	47.6	95.1	213.2	426.4	1610.4	117	190.3	639.6	4.77	1.65	563	230	9487	-10351	2251	757	38052	-41281
I10-Escambia Bay Span Hypothetical Conditions	34	25	5.6	10	4	89.1	6.37	-9	35.3	60	0.58	3.75	6	1568.3	3136.7	104	206.8	108	216	589.7	1179.3	6273.3	723.8	432	1769	5.06	1.81	1728	694	28858	-32046	6911	2493	114947	-128674
I10-Escambia Bay Span Hypothetical Conditions	35	25	5.6	10	6	154.8	6.35	-9	35.3	60	0.58	3.75	6	1565.6	3131.3	103.2	206.4	107.9	215.7	588.8	1177.6	6262.5	722.4	431.5	1766.4	5.06	1.81	1725	692	28821	-31997	6901	2489	114775	-128478
I10-Escambia Bay Span Hypothetical Conditions	36	25	5.6	10	8	221.2	6.77	-9	35.3	60	0.58	3.75	6	1622.6	3245.1	107.4	214.8	110.8	221.6	607.2	1214.3	6490.2	752	443.3	1821.5	5.06	1.81	1782	715	29763	-33052	7128	2574	118531	-132744
I10-Escambia Bay Span Hypothetical Conditions	101	40	4.4	8	3.1	58.7	5.27	3	35.3	60	0.58	3.75	6	0	0	0	0	16	31.9	29.7	59.4	0	0	63.8	89	3.29	0.76	96	30	1670	-1714	383	89	6679	-6814
I10-Escambia Bay Span Hypothetical Conditions	102	40	4.4	8	3.1	58.7	5.27	2	35.3	60	0.58	3.75	6	0	0	0	0	23	46	61.6	123.2	0	0	91.9	184.8	3.62	1.09	138	62	2365	-2499	551	185	9517	-9920
I10-Escambia Bay Span Hypothetical Conditions	103	40	4.4	8	3.1	58.7	5.27	1	35.3	60	0.58	3.75	6	63.7	127.4	0.4	0.8	30	60	103.7	207.5	254.8	3	120	311.2	3.96	1.39	224	104	3800	-4091	895	314	15337	-16227
I10-Escambia Bay Span Hypothetical Conditions	104	40	4.4	8	3.1	58.7	5.27	0	35.3	60	0.58	3.75	6	199.3	398.5	4.1	8.3	37	74.1	147.5	295.1	797	29	148.1	442.6	4.29	1.54	359	152	6086	-6576	1437	472	24526	-26141
I10-Escambia Bay Span Hypothetical Conditions	105	40	4.4	8	3.1	58.7	5.27	-1	35.3	60	0.58	3.75	6	334.8	669.6	11.7	23.4	44.1	88.1	191.3	382.6	1339.3	82	176.2	573.9	4.62	1.63	495	203	8361	-9091	1979	656	33574	-36197
I10-Escambia Bay Span Hypothetical Conditions	106	40	4.4	8	3.1	58.7	5.27	-2	35.3	60	0.58	3.75	6	470.4	940.7	21.7	43.5	51.1	102.2	235.1	470.2	1881.5	152.2	204.4	705.2	4.85	1.68	630	257	10613	-11611	2522	857	42528	-46367
I10-Escambia Bay Span Hypothetical Conditions	107	40	4.4	8	3.1	58.7	5.27	-3	35.3	60	0.58	3.75	6	605.9	1211.8	31.8	63.7	58.1	116.2	278.9	557.7	2423.7	222.8	232.5	836.6	4.93	1.71	766	311	12863	-14131	3064	1059	51468	-56529
I10-Escambia Bay Span Hypothetical Conditions	108	40	4.4	8	3.1	58.7	5.27	-4	35.3	60	0.58	3.75	6	741.5	1482.9	41.9	83.8	65.1	130.3	322.6	645.3	2965.9	293.4	260.6	967.9	4.97	1.74	901	365	15114	-16651	3606	1261	60409	-66690
I10-Escambia Bay Span Hypothetical Conditions	109	40	4.4	8	3.1	58.7	5.27	-5	35.3	60	0.58	3.75	6	877	1754	52	104	72.2	144.3	366.4	732.8	3508.1	363.9	288.7	1099.2	5.00	1.76	1037	418	17374	-19181	4148	1463	69351	-76851
I10-Escambia Bay Span Hypothetical Conditions	110	40	4.4	8	3.1	58.7	5.27	-6	35.3	60	0.58	3.75	6	1012.6	2025.1	62.1	124.1	79.2	159.4	410.2	820.4	4050.3	434.5	316.8	1230.5	5.02	1.77	1172	472	19625	-21702	4689	1665	78291	-87013
I10-Escambia Bay Span Hypothetical Conditions	111	40	4.4	8	3.1	58.7	5.27	-7	35.3	60	0.58	3.75	6	1148.1	2296.3	72.1	144.3	86.2	172.4	454	907.9	4592.5	505	344.9	1361.9	5.03	1.78	1308	526	21875	-24221	5231	1867	87232	-97174
I10-Escambia Bay Span Hypothetical Conditions	112	40	4.4	8	3.1	58.7	5.27	-8	35.3	60	0.58	3.75	6	1283.7	2567.4	82.2	164.4	93.3	186.5	497.7	995.5	5134.7	575.6	373	1493.2	5.04	1.79	1444	580	24137	-26752	5773	2069	96173	-107335
I10-Escambia Bay Span Hypothetical Conditions	113	40	4.4	8	3.1	58.7	5.27	-9	35.3	60	0.58	3.75	6	1419.2	2838.5	92.3	184.6	100.3	200.6	541.5	1083	5676.9	646.2	401.1	1624.5	5.05	1.80	1579	634	26386	-29271	6315	2271	105113	-117498

New Wallingford Approach With 50% Air

Comment	Case #	d_s (ft)	Significant Wave Height (ft)	Maximum Wave Height (ft)	Wave Period (sec)	Wave Length (ft)	Wave Crest Height* (ft)	Y_c (ft)	Span Width (ft)	Span Length (ft)	b (ft)	a (ft)	Number of Girders
I10-Escambia Bay Span and Conditions	123	37.1	4.7	8.2	4.1	92.3	4.95	1.6	35.3	60	0.58	3.75	6
I10-Escambia Bay Span and Conditions	131	39.5	4.7	8.2	4.1	92.7	4.94	-0.3	35.3	60	0.58	3.75	6
I10 - Lake Pontchartrain	W363	22.1	6.5	12.0	5.1	125.8	8.2	1.2	45.5	65	0.54	3.75	6
I10 - Lake Pontchartrain	W363	22.1	6.5	12.0	5.1	125.8	8.2	4.2	45.5	65	0.54	3.75	6
I10 - Lake Pontchartrain	W382	21.2	6.4	11.7	5.1	124.3	8.1	-0.8	45.5	65	0.54	3.75	6
I10 - Lake Pontchartrain	W382	21.2	6.4	11.7	5.1	124.3	8.1	-3.8	45.5	65	0.54	3.75	6
I10-Escambia Bay Span Hypothetical Conditions	1	40	4.4	8	3.1	58.7	5.27	3	35.3	60	0.58	3.75	6
I10-Escambia Bay Span Hypothetical Conditions	2	40	4.4	8	5	128.3	4.48	3	35.3	60	0.58	3.75	6
I10-Escambia Bay Span Hypothetical Conditions	3	40	4.4	8	8	254	4.55	3	35.3	60	0.58	3.75	6
I10-Escambia Bay Span Hypothetical Conditions	4	40	4.4	8	3.1	58.7	5.27	0	35.3	60	0.58	3.75	6
I10-Escambia Bay Span Hypothetical Conditions	5	40	4.4	8	5	128.3	4.48	0	35.3	60	0.58	3.75	6
I10-Escambia Bay Span Hypothetical Conditions	6	40	4.4	8	8	254	4.55	0	35.3	60	0.58	3.75	6
I10-Escambia Bay Span Hypothetical Conditions	7	40	4.4	8	3.1	58.7	5.27	-9	35.3	60	0.58	3.75	6
I10-Escambia Bay Span Hypothetical Conditions	8	40	4.4	8	5	128.3	4.48	-9	35.3	60	0.58	3.75	6
I10-Escambia Bay Span Hypothetical Conditions	9	40	4.4	8	8	254	4.55	-9	35.3	60	0.58	3.75	6
I10-Escambia Bay Span Hypothetical Conditions	10	25	4.4	8	3.1	58.2	5.38	3	35.3	60	0.58	3.75	6
I10-Escambia Bay Span Hypothetical Conditions	11	25	4.4	8	5	118.6	4.77	3	35.3	60	0.58	3.75	6
I10-Escambia Bay Span Hypothetical Conditions	12	25	4.4	8	8	217	5.16	3	35.3	60	0.58	3.75	6
I10-Escambia Bay Span Hypothetical Conditions	13	25	4.4	8	3.1	58.2	5.38	0	35.3	60	0.58	3.75	6
I10-Escambia Bay Span Hypothetical Conditions	14	25	4.4	8	5	118.6	4.77	0	35.3	60	0.58	3.75	6
I10-Escambia Bay Span Hypothetical Conditions	15	25	4.4	8	8	217	5.16	0	35.3	60	0.58	3.75	6
I10-Escambia Bay Span Hypothetical Conditions	16	25	4.4	8	3.1	58.2	5.38	-9	35.3	60	0.58	3.75	6
I10-Escambia Bay Span Hypothetical Conditions	17	25	4.4	8	5	118.6	4.77	-9	35.3	60	0.58	3.75	6
I10-Escambia Bay Span Hypothetical Conditions	18	25	4.4	8	8	217	5.16	-9	35.3	60	0.58	3.75	6
I10-Escambia Bay Span Hypothetical Conditions	19	40	6.7	12	4	95.1	7.66	3	35.3	60	0.58	3.75	6
I10-Escambia Bay Span Hypothetical Conditions	20	40	6.7	12	6	175.1	7.06	3	35.3	60	0.58	3.75	6
I10-Escambia Bay Span Hypothetical Conditions	21	40	6.7	12	8	259	7.25	3	35.3	60	0.58	3.75	6
I10-Escambia Bay Span Hypothetical Conditions	22	40	6.7	12	4	95.1	7.66	0	35.3	60	0.58	3.75	6
I10-Escambia Bay Span Hypothetical Conditions	23	40	6.7	12	6	175.1	7.06	0	35.3	60	0.58	3.75	6
I10-Escambia Bay Span Hypothetical Conditions	24	40	6.7	12	8	258.6	7.25	0	35.3	60	0.58	3.75	6
I10-Escambia Bay Span Hypothetical Conditions	25	40	6.7	12	4	95.1	7.66	-9	35.3	60	0.58	3.75	6
I10-Escambia Bay Span Hypothetical Conditions	26	40	6.7	12	6	175.1	7.06	-9	35.3	60	0.58	3.75	6
I10-Escambia Bay Span Hypothetical Conditions	27	40	6.7	12	8	258.6	7.25	-9	35.3	60	0.58	3.75	6
I10-Escambia Bay Span Hypothetical Conditions	28	25	5.6	10	4	89.1	6.37	3	35.3	60	0.58	3.75	6
I10-Escambia Bay Span Hypothetical Conditions	29	25	5.6	10	6	154.8	6.35	3	35.3	60	0.58	3.75	6
I10-Escambia Bay Span Hypothetical Conditions	30	25	5.6	10	8	221.2	6.77	3	35.3	60	0.58	3.75	6
I10-Escambia Bay Span Hypothetical Conditions	31	25	5.6	10	4	89.11	6.37	0	35.3	60	0.58	3.75	6
I10-Escambia Bay Span Hypothetical Conditions	32	25	5.6	10	6	154.8	6.35	0	35.3	60	0.58	3.75	6
I10-Escambia Bay Span Hypothetical Conditions	33	25	5.6	10	8	221.2	6.77	0	35.3	60	0.58	3.75	6
I10-Escambia Bay Span Hypothetical Conditions	34	25	5.6	10	4	89.1	6.37	-9	35.3	60	0.58	3.75	6
I10-Escambia Bay Span Hypothetical Conditions	35	25	5.6	10	6	154.8	6.35	-9	35.3	60	0.58	3.75	6
I10-Escambia Bay Span Hypothetical Conditions	36	25	5.6	10	8	221.2	6.77	-9	35.3	60	0.58	3.75	6
I10-Escambia Bay Span Hypothetical Conditions	101	40	4.4	8	3.1	58.7	5.27	3	35.3	60	0.58	3.75	6
I10-Escambia Bay Span Hypothetical Conditions	102	40	4.4	8	3.1	58.7	5.27	2	35.3	60	0.58	3.75	6
I10-Escambia Bay Span Hypothetical Conditions	103	40	4.4	8	3.1	58.7	5.27	1	35.3	60	0.58	3.75	6
I10-Escambia Bay Span Hypothetical Conditions	104	40	4.4	8	3.1	58.7	5.27	0	35.3	60	0.58	3.75	6
I10-Escambia Bay Span Hypothetical Conditions	105	40	4.4	8	3.1	58.7	5.27	-1	35.3	60	0.58	3.75	6
I10-Escambia Bay Span Hypothetical Conditions	106	40	4.4	8	3.1	58.7	5.27	-2	35.3	60	0.58	3.75	6
I10-Escambia Bay Span Hypothetical Conditions	107	40	4.4	8	3.1	58.7	5.27	-3	35.3	60	0.58	3.75	6
I10-Escambia Bay Span Hypothetical Conditions	108	40	4.4	8	3.1	58.7	5.27	-4	35.3	60	0.58	3.75	6
I10-Escambia Bay Span Hypothetical Conditions	109	40	4.4	8	3.1	58.7	5.27	-5	35.3	60	0.58	3.75	6
I10-Escambia Bay Span Hypothetical Conditions	110	40	4.4	8	3.1	58.7	5.27	-6	35.3	60	0.58	3.75	6
I10-Escambia Bay Span Hypothetical Conditions	111	40	4.4	8	3.1	58.7	5.27	-7	35.3	60	0.58	3.75	6
I10-Escambia Bay Span Hypothetical Conditions	112	40	4.4	8	3.1	58.7	5.27	-8	35.3	60	0.58	3.75	6
I10-Escambia Bay Span Hypothetical Conditions	113	40	4.4	8	3.1	58.7	5.27	-9	35.3	60	0.58	3.75	6

* Wave crest height above storm water level (h_{max})

Case #	Vertical Force on Deck - External (kips)	Vertical Force on Deck - Internal (kips)	Lateral Force on Deck - External (kips)	Vertical Force on Seaward Girder (kips)	Vertical Force on Internal Girder (kips)	Lateral Force on Seaward Girder (kips)	Lateral Force on Internal Girder (kips)	Vertical Impact Load on Deck - External (kips)	Vertical Impact Load on Deck - Internal (kips)	Lateral Impact Load on Deck (kips)	Vertical Impact Load on Seaward Girder (kips)	Lateral Impact Load on Seaward Girder (kips)	Vertical Impact Load on Internal Girder (kips)	Lateral Impact Load on Internal Girder (kips)	Lateral Deck Moment Arm	Lateral Girder Moment Arm
123	371.1	504.8	0	18.5	19.2	32.5	48.4	824	1156	0	42	80	50	162	3.62	1.12
131	416.5	550.3	12.8	20	21.2	39.8	55.8	925	1260	31	46	97	55	187	4.25	1.11
W363	1087.7	1346.1	47	40	44.1	81.8	94	2415	3083	115	91	200	114	315	4.66	1.05
W363	825.5	1083.9	3	32.6	34.9	65.4	86.3	1833	2482	7	74	160	91	289	3.83	1.21
W382	1260.1	1514.5	55	44.9	50.2	92.8	98.4	2798	3468	135	102	227	130	329	4.56	1.02
W382	1529.3	1783.7	67.9	52.5	59.6	109.6	106.3	3395	4085	166	120	269	154	356	4.52	1.00
1	318.8	444.1	0	16.2	16.6	19.1	30	708	1017	0	37	47	43	100	3.29	0.76
2	0	0	0	15.5	15.8	11.9	19.3	0	0	0	35	29	41	65	3.03	0.49
3	0	0	0	15.5	15.9	12.5	20.2	0	0	0	35	31	41	68	3.05	0.52
4	389	514.3	11.8	18.7	19.8	37.7	52.9	864	1178	29	43	92	51	177	4.29	1.13
5	370.5	495.8	5.2	18	18.9	36.2	52.2	823	1135	13	41	89	49	175	4.03	1.18
6	372.2	497.4	5.7	18.1	19	36.3	52.2	826	1139	14	41	89	49	175	4.05	1.18
7	599.7	725	32.9	26.4	29.2	54.9	61	1331	1660	81	60	134	76	204	4.49	1.00
8	581.2	706.5	31.8	25.7	28.4	53.4	60.6	1290	1618	78	59	131	74	202	4.49	1.00
9	582.9	708	31.9	25.7	28.5	53.5	60.3	1294	1622	78	59	131	74	202	4.49	1.00
10	328.1	453.4	0	17.5	18.2	21.8	32.3	728	1038	0	40	54	47	108	3.33	0.79
11	0	0	0	16.6	17.2	15.4	23.6	0	0	0	38	38	45	79	3.12	0.59
12	319.9	445.1	0	17.2	17.9	19.4	29.1	710	1019	0	39	48	46	98	3.25	0.72
13	440.5	565.7	13.5	21.5	23.3	44.1	55.9	978	1296	33	49	108	60	187	4.33	1.12
14	417.6	542.9	7.8	20.7	22.2	42.2	55	927	1243	19	47	103	58	184	4.12	1.16
15	432.2	557.5	11.4	21.2	22.9	43.4	55.6	960	1277	28	48	106	59	186	4.25	1.14
16	777.6	902.8	41.4	33.8	38.4	71.6	68.8	1726	2068	101	77	175	99	231	4.49	1.00
17	754.7	880	40.1	33	37.4	6937	68	1676	2015	98	75	171	97	228	4.49	1.00
18	769.4	894.6	40.9	33.5	38	70.9	68.5	1708	2049	100	76	174	99	230	4.49	1.00
19	570.6	761.4	10.1	27.7	29.1	55.6	79.7	1267	1744	25	63	136	75	267	4.09	1.17
20	549.3	740	2.9	26.9	28.2	53.8	78.9	1219	1695	7	61	132	73	264	3.89	1.23
21	556	746.8	5.1	27.2	28.5	54.4	79.1	1234	1710	13	62	133	74	265	3.95	1.21
22	677.6	868.3	36.8	31.6	33.9	64.3	83.8	1504	1988	90	72	158	88	281	4.57	1.05
23	656.2	846.9	35.6	30.8	33	62.6	83	1457	1939	87	70	153	85	278	4.60	1.07
24	663	853.7	36	31	33.3	63.1	83.2	1472	1955	88	71	155	86	279	4.59	1.06
25	998.4	1189.1	54.9	43.2	48.3	90.5	96.1	2216	2723	134	99	222	125	322	4.48	0.99
26	977	1167.7	53.7	42.5	47.4	88.8	95.3	2169	2674	131	97	217	123	319	4.48	0.99
27	983.8	1174.5	54.1	42.7	47.7	89.3	95.6	2184	2690	132	97	219	123	320	4.48	0.99
28	464.8	624.2	0	23.9	25.3	42.8	59.8	1032	1429	0	55	105	66	200	3.66	1.12
29	463.9	623.3	0	23.9	25.3	42.5	59.4	1030	1427	0	54	104	65	199	3.65	1.12
30	483.9	643.3	0	24.6	26.2	49.4	67.6	1074	1473	0	56	121	68	227	3.79	1.26
31	607.8	767.2	30.5	29.1	31.7	59.9	72.9	1349	1757	75	66	147	82	244	4.66	1.09
32	606.9	766.3	30.2	29.1	31.7	59.8	72.9	1347	1755	74	66	147	82	244	4.65	1.09
33	623.9	786.3	32.3	29.8	32.6	61.5	73.7	1392	1801	79	68	151	84	247	4.63	1.07
34	1036.9	1196.3	55.4	44.7	51	94.9	89.4	2302	2739	136	102	233	132	300	4.48	1.00
35	1035.9	1195.3	55.3	44.7	51	94.8	89.4	2300	2737	136	102	231	132	299	4.48	1.00
36	1055.9	1215.3	56.4	45.4	51.9	96.5	90.2	2344	2783	138	104	236	134	302	4.48	0.99
101	318.8	444.1	0	18.8	25.3	19.1	30	708	1017	0	43	47	66	100	3.29	0.76
102	342.2	467.5	0	20	26.5	29.1	44	760	1070	0	46	71	69	147	3.62	1.09
103	365.6	490.9	3.5	21.3	27.8	35.8	52	812	1124	9	48	88	72	174	3.96	1.20
104	389	514.3	11.8	22.5	29	37.7	52.9	864	1178	29	51	92	75	177	4.29	1.13
105	415.4	537.7	21	23.7	30.2	39.6	53.8	916	1231	51	54	97	78	180	4.62	1.09
106	435.9	561.1	23.7	24.9	31.4	41.5	54.7	968	1285	58	57	102	81	183	4.59	1.06
107	459.3	584.5	25	26.1	32.6	43.4	55.6	1020	1339	61	60	106	84	186	4.55	1.04
108	482.7	607.9	26.3	27.3	33.8	45.3	56.5	1072	1392	64	62	111	88	189	4.53	1.03
109	506.1	631.3	27.6	28.5	35	47.2	57.4	1124	1446	68	65	116	91	192	4.51	1.02
110	529.5	654.7	28.9	29.8	36.2	49.1	58.3	1175	1499	71	68	120	94	195	4.50	1.01
111	552.9	678.2	30.2	31	37.5	51	59.2	1227	1553	74	71	125	97	198	4.50	1.01
112	576.3	701.6	31.6	32.2	38.7	53	60.1	1279	1607	77	76	130	100	201	4.49	1.00
113	599.7	725	32.9	33.4	39.9	54.9	61	1331	1660	81	76	134	103	204	4.49	1.00

Case #	Total Quasi-Static Vertical Force (kips)	Total Quasi-Static Horizontal Force (kips)	Leading Edge Moment (ft kips)	Trailing Edge Moment (ft kips)	Total Impact Vertical Force (kips)	Total Impact Horizontal Force (kips)	Leading Edge Moment (ft kips)	Trailing Edge Moment (ft kips)
123	462	275	7837	-8450	1088	890	18180	-20169
131	505	332	8490	-9308	1189	1063	19668	-22227
W363	1282	599	28361	-29960	3000	1890	65847	-70653
W363	1029	500	22805	-24034	2412	1612	52900	-56848
W382	1445	640	32024	-33720	3383	2007	74438	-79493
W382	1704	709	37815	-39706	3989	2215	87963	-93546
1	405	169	7010	-7266	952	547	16369	-17197
2	95	108	1612	-1719	240	354	4055	-4405
3	95	114	1616	-1733	240	371	4038	-4422
4	472	314	7924	-8707	1109	1006	18320	-20775
5	454	302	7627	-8372	1067	977	17621	-20007
6	456	303	7656	-8402	1070	978	17671	-20054
7	672	393	11329	-12344	1583	1235	26383	-29415
8	654	388	11029	-12029	1543	1219	25702	-28689
9	656	387	11057	-12055	1546	1219	25751	-28738
10	421	183	7269	-7560	990	594	16972	-17914
11	103	133	1730	-1887	263	433	4380	-4891
12	413	165	7163	-7401	971	538	16719	-17494
13	527	337	8875	-9719	1241	1076	20564	-23192
14	505	325	8509	-9309	1193	1042	19758	-22288
15	520	333	8744	-9570	1222	1064	20247	-22837
16	847	457	14335	-15536	1996	1431	33396	-36957
17	826	7317	7092	-22021	1947	1409	32570	-36074
18	839	454	14199	-15392	1982	1424	33157	-36700
19	697	464	11721	-12864	1639	1496	27063	-30703
20	677	451	11377	-12501	1593	1459	26265	-29889
21	684	455	11491	-12616	1609	1471	26555	-30175
22	799	520	13404	-14759	1881	1653	31090	-35207
23	779	513	13055	-14402	1830	1630	30207	-34301
24	785	515	13167	-14515	1847	1638	30504	-34604
25	1103	626	18636	-20259	2599	1966	43388	-48220
26	1083	619	18295	-19898	2553	1943	42613	-47383
27	1090	621	18403	-20014	2564	1951	42796	-47588
28	580	342	9842	-10609	1369	1105	22886	-25367
29	580	340	9835	-10594	1362	1099	22769	-25224
30	599	387	10062	-11036	1410	1256	23276	-26433
31	716	455	12014	-13219	1686	1442	27878	-31543
32	715	455	12005	-13206	1684	1441	27859	-31514
33	734	462	12329	-13551	1728	1465	28602	-32310
34	1123	597	19012	-20587	2648	1869	44334	-49002
35	1123	597	19000	-20574	2646	1862	44317	-48971
36	1142	604	19325	-20918	2690	1884	45061	-49766
101	451	169	7822	-8078	1073	547	18501	-19329
102	474	249	8089	-8632	1128	806	18997	-20754
103	498	299	8412	-9152	1182	967	19642	-22020
104	522	314	8801	-9585	1237	1006	20576	-23031
105	545	330	9171	-10037	1292	1048	21443	-24084
106	568	339	9572	-10458	1347	1075	22390	-25082
107	592	346	9977	-10875	1402	1097	23350	-26068
108	615	354	10379	-11294	1460	1120	24360	-27119
109	638	362	10781	-11714	1516	1144	25306	-28119
110	662	370	11185	-12136	1570	1166	26243	-29103
111	685	377	11595	-12567	1625	1189	27189	-30102
112	709	385	11995	-12988	1682	1212	28169	-31137
113	732	393	12396	-13410	1734	1235	29044	-32077

New Wallingford Approach With 100% Air

Comment	Case #	d_s (ft)	Significant Wave Height (ft)	Maximum Wave Height (ft)	Wave Period (sec)	Wave Length (ft)	Wave Crest Height* (ft)	Y_c (ft)	Span Width (ft)	Span Length (ft)	b (ft)	a (ft)	Number of Girders
I10-Escambia Bay Span and Conditions	123	37.1	4.7	8.2	4.1	92.3	4.95	1.6	35.3	60	0.58	3.75	6
I10-Escambia Bay Span and Conditions	131	39.5	4.7	8.2	4.1	92.7	4.94	-0.3	35.3	60	0.58	3.75	6
I10 - Lake Pontchartrain	W363	22.1	6.5	12.0	5.1	125.8	8.2	1.2	45.5	65	0.54	3.75	6
I10 - Lake Pontchartrain	W363	22.1	6.5	12.0	5.1	125.8	8.2	4.2	45.5	65	0.54	3.75	6
I10 - Lake Pontchartrain	W382	21.2	6.4	11.7	5.1	124.3	8.1	-0.8	45.5	65	0.54	3.75	6
I10 - Lake Pontchartrain	W382	21.2	6.4	11.7	5.1	124.3	8.1	-3.8	45.5	65	0.54	3.75	6
I10-Escambia Bay Span Hypothetical Conditions	1	40	4.4	8	3.1	58.7	5.27	3	35.3	60	0.58	3.75	6
I10-Escambia Bay Span Hypothetical Conditions	2	40	4.4	8	5	128.3	4.48	3	35.3	60	0.58	3.75	6
I10-Escambia Bay Span Hypothetical Conditions	3	40	4.4	8	8	254	4.55	3	35.3	60	0.58	3.75	6
I10-Escambia Bay Span Hypothetical Conditions	4	40	4.4	8	3.1	58.7	5.27	0	35.3	60	0.58	3.75	6
I10-Escambia Bay Span Hypothetical Conditions	5	40	4.4	8	5	128.3	4.48	0	35.3	60	0.58	3.75	6
I10-Escambia Bay Span Hypothetical Conditions	6	40	4.4	8	8	254	4.55	0	35.3	60	0.58	3.75	6
I10-Escambia Bay Span Hypothetical Conditions	7	40	4.4	8	3.1	58.7	5.27	-9	35.3	60	0.58	3.75	6
I10-Escambia Bay Span Hypothetical Conditions	8	40	4.4	8	5	128.3	4.48	-9	35.3	60	0.58	3.75	6
I10-Escambia Bay Span Hypothetical Conditions	9	40	4.4	8	8	254	4.55	-9	35.3	60	0.58	3.75	6
I10-Escambia Bay Span Hypothetical Conditions	10	25	4.4	8	3.1	58.2	5.38	3	35.3	60	0.58	3.75	6
I10-Escambia Bay Span Hypothetical Conditions	11	25	4.4	8	5	118.6	4.77	3	35.3	60	0.58	3.75	6
I10-Escambia Bay Span Hypothetical Conditions	12	25	4.4	8	8	217	5.16	3	35.3	60	0.58	3.75	6
I10-Escambia Bay Span Hypothetical Conditions	13	25	4.4	8	3.1	58.2	5.38	0	35.3	60	0.58	3.75	6
I10-Escambia Bay Span Hypothetical Conditions	14	25	4.4	8	5	118.6	4.77	0	35.3	60	0.58	3.75	6
I10-Escambia Bay Span Hypothetical Conditions	15	25	4.4	8	8	217	5.16	0	35.3	60	0.58	3.75	6
I10-Escambia Bay Span Hypothetical Conditions	16	25	4.4	8	3.1	58.2	5.38	-9	35.3	60	0.58	3.75	6
I10-Escambia Bay Span Hypothetical Conditions	17	25	4.4	8	5	118.6	4.77	-9	35.3	60	0.58	3.75	6
I10-Escambia Bay Span Hypothetical Conditions	18	25	4.4	8	8	217	5.16	-9	35.3	60	0.58	3.75	6
I10-Escambia Bay Span Hypothetical Conditions	19	40	6.7	12	4	95.1	7.66	3	35.3	60	0.58	3.75	6
I10-Escambia Bay Span Hypothetical Conditions	20	40	6.7	12	6	175.1	7.06	3	35.3	60	0.58	3.75	6
I10-Escambia Bay Span Hypothetical Conditions	21	40	6.7	12	8	259	7.25	3	35.3	60	0.58	3.75	6
I10-Escambia Bay Span Hypothetical Conditions	22	40	6.7	12	4	95.1	7.66	0	35.3	60	0.58	3.75	6
I10-Escambia Bay Span Hypothetical Conditions	23	40	6.7	12	6	175.1	7.06	0	35.3	60	0.58	3.75	6
I10-Escambia Bay Span Hypothetical Conditions	24	40	6.7	12	8	258.6	7.25	0	35.3	60	0.58	3.75	6
I10-Escambia Bay Span Hypothetical Conditions	25	40	6.7	12	4	95.1	7.66	-9	35.3	60	0.58	3.75	6
I10-Escambia Bay Span Hypothetical Conditions	26	40	6.7	12	6	175.1	7.06	-9	35.3	60	0.58	3.75	6
I10-Escambia Bay Span Hypothetical Conditions	27	40	6.7	12	8	258.6	7.25	-9	35.3	60	0.58	3.75	6
I10-Escambia Bay Span Hypothetical Conditions	28	25	5.6	10	4	89.1	6.37	3	35.3	60	0.58	3.75	6
I10-Escambia Bay Span Hypothetical Conditions	29	25	5.6	10	6	154.8	6.35	3	35.3	60	0.58	3.75	6
I10-Escambia Bay Span Hypothetical Conditions	30	25	5.6	10	8	221.2	6.77	3	35.3	60	0.58	3.75	6
I10-Escambia Bay Span Hypothetical Conditions	31	25	5.6	10	4	89.11	6.37	0	35.3	60	0.58	3.75	6
I10-Escambia Bay Span Hypothetical Conditions	32	25	5.6	10	6	154.8	6.35	0	35.3	60	0.58	3.75	6
I10-Escambia Bay Span Hypothetical Conditions	33	25	5.6	10	8	221.2	6.77	0	35.3	60	0.58	3.75	6
I10-Escambia Bay Span Hypothetical Conditions	34	25	5.6	10	4	89.1	6.37	-9	35.3	60	0.58	3.75	6
I10-Escambia Bay Span Hypothetical Conditions	35	25	5.6	10	6	154.8	6.35	-9	35.3	60	0.58	3.75	6
I10-Escambia Bay Span Hypothetical Conditions	36	25	5.6	10	8	221.2	6.77	-9	35.3	60	0.58	3.75	6
I10-Escambia Bay Span Hypothetical Conditions	101	40	4.4	8	3.1	58.7	5.27	3	35.3	60	0.58	3.75	6
I10-Escambia Bay Span Hypothetical Conditions	102	40	4.4	8	3.1	58.7	5.27	2	35.3	60	0.58	3.75	6
I10-Escambia Bay Span Hypothetical Conditions	103	40	4.4	8	3.1	58.7	5.27	1	35.3	60	0.58	3.75	6
I10-Escambia Bay Span Hypothetical Conditions	104	40	4.4	8	3.1	58.7	5.27	0	35.3	60	0.58	3.75	6
I10-Escambia Bay Span Hypothetical Conditions	105	40	4.4	8	3.1	58.7	5.27	-1	35.3	60	0.58	3.75	6
I10-Escambia Bay Span Hypothetical Conditions	106	40	4.4	8	3.1	58.7	5.27	-2	35.3	60	0.58	3.75	6
I10-Escambia Bay Span Hypothetical Conditions	107	40	4.4	8	3.1	58.7	5.27	-3	35.3	60	0.58	3.75	6
I10-Escambia Bay Span Hypothetical Conditions	108	40	4.4	8	3.1	58.7	5.27	-4	35.3	60	0.58	3.75	6
I10-Escambia Bay Span Hypothetical Conditions	109	40	4.4	8	3.1	58.7	5.27	-5	35.3	60	0.58	3.75	6
I10-Escambia Bay Span Hypothetical Conditions	110	40	4.4	8	3.1	58.7	5.27	-6	35.3	60	0.58	3.75	6
I10-Escambia Bay Span Hypothetical Conditions	111	40	4.4	8	3.1	58.7	5.27	-7	35.3	60	0.58	3.75	6
I10-Escambia Bay Span Hypothetical Conditions	112	40	4.4	8	3.1	58.7	5.27	-8	35.3	60	0.58	3.75	6
I10-Escambia Bay Span Hypothetical Conditions	113	40	4.4	8	3.1	58.7	5.27	-9	35.3	60	0.58	3.75	6

Case #	Vertical Force on Deck - External (kips)	Vertical Force on Deck - Internal (kips)	Lateral Force on Deck - External (kips)	Vertical Force on Seaward Girder (kips)	Vertical Force on Internal Girder (kips)	Lateral Force on Seaward Girder (kips)	Lateral Force on Internal Girder (kips)	Vertical Impact Load on Deck - External (kips)	Vertical Impact Load on Deck - Internal (kips)	Lateral Impact Load on Deck (kips)	Vertical Impact Load on Seaward Girder (kips)	Lateral Impact Load on Seaward Girder (kips)	Vertical Impact Load on Internal Girder (kips)	Lateral Impact Load on Internal Girder (kips)	Lateral Deck Moment Arm	Lateral Girder Moment Arm
123	421.6	555.4	0	18.5	19.2	32.5	48.4	936	1272	0	42	80	50	162	3.62	1.12
131	464	597.8	12.8	20	21.2	39.8	55.8	1030	1369	31	46	97	55	187	4.25	1.11
W363	1251.6	1510	47	40	44.1	81.8	94	2779	3458	115	91	200	114	315	4.66	1.05
W363	989.4	1247.7	3	32.6	34.9	65.4	86.3	2196	2857	7	74	160	91	289	3.83	1.21
W382	1428.4	1682.7	55	44.9	50.2	92.8	98.4	3171	3853	135	102	227	130	329	4.56	1.02
W382	1697.5	1951.9	67.9	52.5	59.6	109.6	106.3	3768	4470	166	120	269	154	356	4.52	1.00
1	363.3	488.5	0	16.2	16.6	19.1	30	806	1119	0	37	47	43	100	3.29	0.76
2	344.8	470	0	15.5	15.8	11.9	19.3	765	1076	0	35	29	41	65	3.03	0.49
3	346.4	471.7	0	15.5	15.9	12.5	20.2	769	1080	0	35	31	41	68	3.05	0.52
4	433.5	558.8	11.8	18.7	19.8	37.7	52.9	962	1280	29	43	92	51	177	4.29	1.13
5	415	540.3	5.2	18	18.9	36.2	52.2	921	1237	13	41	89	49	175	4.03	1.18
6	416.7	541.9	5.7	18.1	19	36.3	52.2	925	1241	14	41	89	49	175	4.05	1.18
7	644.2	769.5	32.9	26.4	29.2	54.9	61	1430	1762	81	60	134	76	204	4.49	1.00
8	625.7	751	31.8	25.7	28.4	53.4	60.6	1389	1720	78	59	131	74	202	4.49	1.00
9	627.3	752.6	31.9	25.7	28.5	53.5	60.3	1393	1723	78	59	131	74	202	4.49	1.00
10	399.3	524.5	0	17.5	18.2	21.8	32.3	886	1201	0	40	54	47	108	3.33	0.79
11	376.4	501.7	0	16.6	17.2	15.4	23.6	836	1149	0	38	38	45	79	3.12	0.59
12	391	516.3	0	17.2	17.9	19.4	29.1	868	1182	0	39	48	46	98	3.25	0.72
13	511.7	636.9	13.5	21.5	23.3	44.1	55.9	1136	1459	33	49	108	60	187	4.33	1.12
14	488.8	614.1	7.8	20.7	22.2	42.2	55	1085	1406	19	47	103	58	184	4.12	1.16
15	503.1	628.7	11.4	21.2	22.9	43.4	55.6	1118	1440	28	48	106	59	186	4.25	1.14
16	848.8	974	41.4	33.8	38.4	71.6	68.8	1884	2230	101	77	175	99	231	4.49	1.00
17	825.9	951.2	40.1	33	37.4	6937	68	1834	2178	98	75	171	97	228	4.49	1.00
18	840.5	965.8	40.9	33.5	38	70.9	68.5	1866	2212	100	76	174	99	230	4.49	1.00
19	638.4	829.1	10.1	27.7	29.1	55.6	79.7	1417	1899	25	63	136	75	267	4.09	1.17
20	617	807.7	2.9	26.9	28.2	53.8	78.9	1370	1850	7	61	132	73	264	3.89	1.23
21	623.8	814.5	5.1	27.2	28.5	54.4	79.1	1385	1865	13	62	133	74	265	3.95	1.21
22	745.3	936	36.8	31.6	33.9	64.3	83.8	1655	2144	90	72	158	88	281	4.57	1.05
23	723.9	914.7	35.6	30.8	33	62.6	83	1607	2095	87	70	153	85	278	4.60	1.07
24	730.7	921.4	36	31	33.3	63.1	83.2	1622	2110	88	71	155	86	279	4.59	1.06
25	1066.1	1256.9	54.9	43.2	48.3	90.5	96.1	2367	2878	134	99	222	125	322	4.48	0.99
26	1044.8	1235.5	53.7	42.5	47.4	88.8	95.3	2319	2829	131	97	217	123	319	4.48	0.99
27	1051.5	1242.2	54.1	42.7	47.7	89.3	95.6	2334	2845	132	97	219	123	320	4.48	0.99
28	555.4	714.8	0	23.9	25.3	42.8	59.8	1233	1637	0	55	105	66	200	3.66	1.12
29	554.4	713.8	0	23.9	25.3	42.5	59.4	1231	1635	0	54	104	65	199	3.65	1.12
30	574.4	733.9	0	24.6	26.2	49.4	67.6	1275	1681	0	56	121	68	227	3.79	1.26
31	698.4	857.8	30.5	29.1	31.7	59.9	72.9	1550	1964	75	66	147	82	244	4.66	1.09
32	697.4	856.8	30.2	29.1	31.7	59.8	72.9	1548	1962	74	66	147	82	244	4.65	1.09
33	717.5	876.9	32.3	29.8	32.6	61.5	73.7	1593	2008	79	68	151	84	247	4.63	1.07
34	1127.4	1286.8	55.4	44.7	51	94.9	89.4	2503	2947	136	102	233	132	300	4.48	1.00
35	1126.5	1285.9	55.3	44.7	51	94.8	89.4	2501	2945	136	102	231	132	299	4.48	1.00
36	1146.5	1305.9	56.4	45.4	51.9	96.5	90.2	2545	2991	138	104	236	134	302	4.48	0.99
101	363.3	488.5	0	18.8	25.3	19.1	30	806	1119	0	43	47	66	100	3.29	0.76
102	386.7	511.9	0	20	26.5	29.1	44	858	1172	0	46	71	69	147	3.62	1.09
103	410.1	535.4	3.5	21.3	27.8	35.8	52	910	1226	9	48	88	72	174	3.96	1.20
104	433.5	558.8	11.8	22.5	29	37.7	52.9	962	1280	29	51	92	75	177	4.29	1.13
105	456.9	582.2	21	23.7	30.2	39.6	53.8	1014	1333	51	54	97	78	180	4.62	1.09
106	480.3	605.6	23.7	24.9	31.4	41.5	54.7	1066	1387	58	57	102	81	183	4.59	1.06
107	503.7	629	25	26.1	32.6	43.4	55.6	1118	1440	61	60	106	84	186	4.55	1.04
108	527.2	652.4	26.3	27.3	33.8	45.3	56.5	1170	1494	64	62	111	88	189	4.53	1.03
109	550.6	675.8	27.6	28.5	35	47.2	57.4	1222	1548	68	65	116	91	192	4.51	1.02
110	574	699.2	28.9	29.8	36.2	49.1	58.3	1274	1601	71	68	120	94	195	4.50	1.01
111	597.4	722.6	30.2	31	37.5	51	59.2	1326	1655	74	71	125	97	198	4.50	1.01
112	620.8	746	31.6	32.2	38.7	53	60.1	1378	1708	77	76	130	100	201	4.49	1.00
113	644.2	769.5	32.9	33.4	39.9	54.9	61	1430	1762	81	76	134	103	204	4.49	1.00

Case #	Total Quasi-Static Vertical Force (kips)	Total Quasi-Static Horizontal Force (kips)	Leading Edge Moment (ft-kips)	Trailing Edge Moment (ft-kips)	Total Impact Vertical Force (kips)	Total Impact Horizontal Force (kips)	Leading Edge Moment (ft-kips)	Trailing Edge Moment (ft-kips)
123	497	275	8451	-9064	1168	890	19588	-21576
131	538	332	9066	-9884	1264	1063	20991	-23550
W363	1406	599	31190	-32788	3285	1890	72320	-77126
W363	1154	500	25633	-26861	2697	1612	59373	-63320
W382	1573	640	34927	-36623	3675	2007	81084	-86138
W382	1831	709	40718	-42609	4281	2215	94609	-100191
1	436	169	7548	-7804	1022	547	17606	-18435
2	418	108	7316	-7422	981	354	17113	-17462
3	420	114	7340	-7457	984	371	17144	-17528
4	502	314	8464	-9247	1179	1006	19558	-22012
5	485	302	8167	-8912	1138	977	18859	-21245
6	486	303	8196	-8942	1140	978	18909	-21292
7	702	393	11869	-12884	1653	1235	27620	-30653
8	685	388	11569	-12569	1613	1219	26940	-29927
9	686	387	11598	-12596	1615	1219	26976	-29963
10	470	183	8132	-8422	1102	594	18950	-19892
11	448	133	7818	-7975	1054	433	18323	-18834
12	462	165	8027	-8265	1083	538	18697	-19472
13	577	337	9739	-10583	1354	1076	22542	-25170
14	555	325	9373	-10173	1305	1042	21736	-24266
15	569	333	9608	-10434	1334	1064	22225	-24815
16	896	457	15199	-16400	2107	1431	35362	-38923
17	875	7317	7956	-22885	2060	1409	34548	-38052
18	888	454	15063	-16256	2094	1424	35135	-38678
19	744	464	12542	-13685	1745	1496	28944	-32584
20	724	451	12198	-13323	1700	1459	28146	-31770
21	730	455	12312	-13438	1716	1471	28436	-32055
22	846	520	14225	-15580	1988	1653	32983	-37100
23	826	513	13877	-15224	1937	1630	32100	-36194
24	832	515	13988	-15336	1954	1638	32385	-36485
25	1150	626	19458	-21082	2706	1966	45269	-50101
26	1130	619	19118	-20720	2660	1943	44494	-49264
27	1136	621	19225	-20835	2671	1951	44677	-49469
28	643	342	10941	-11709	1512	1105	25410	-27892
29	642	340	10934	-11692	1505	1099	25293	-27748
30	661	387	11161	-12135	1553	1256	25800	-28957
31	778	455	13113	-14318	1828	1442	30390	-34055
32	778	455	13103	-14305	1827	1441	30371	-34026
33	797	462	13428	-14651	1871	1465	31114	-34822
34	1186	597	20110	-21685	2791	1869	46858	-51526
35	1185	597	20100	-21674	2790	1862	46841	-51495
36	1204	604	20424	-22017	2833	1884	47585	-52290
101	482	169	8361	-8617	1143	547	19739	-20567
102	505	249	8628	-9171	1198	806	20235	-21992
103	529	299	8952	-9692	1252	967	20879	-23258
104	552	314	9341	-10125	1307	1006	21814	-24268
105	576	330	9711	-10577	1362	1048	22681	-25322
106	599	339	10112	-10998	1417	1075	23628	-26320
107	622	346	10517	-11415	1471	1097	24575	-27293
108	645	354	10919	-11834	1531	1120	25598	-28357
109	669	362	11321	-12254	1586	1144	26543	-29356
110	692	370	11725	-12676	1640	1166	27480	-30340
111	716	377	12134	-13105	1695	1189	28426	-31340
112	739	385	12534	-13527	1752	1212	29394	-32363
113	763	393	12936	-13950	1804	1235	30282	-33315

OEA No Air

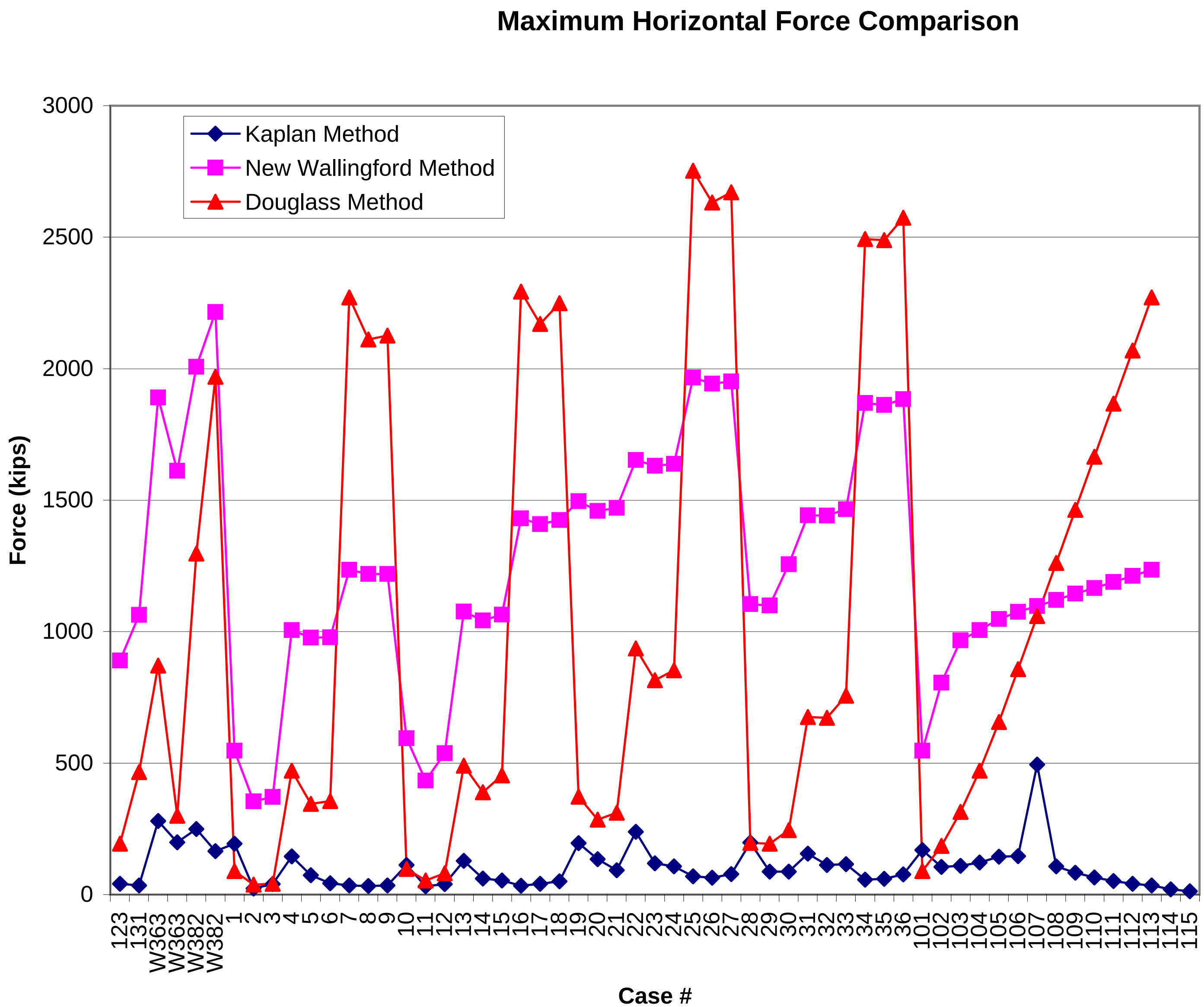
														Maximum Positive Horizontal Force								Maximum Positive Vertical Force								Largest Positive Moment about the Leading Edge								Largest Negative Moment about the Trailing Edge							
Bridge #	Case#	d_s (ft)	Significant Wave Height (ft)	Maximum Wave Height (ft)	Wave Period (sec)	Wave Length (ft)	Wave Crest Height* (ft)	Y_c (ft)	Span Width (ft)	Span Length (ft)	b (ft)	a (ft)	Number of Girders	Maximum Positive Horizontal Force	Associate Vertical Force	Associate Moment about the Leading Edge	Associate Moment about the Trailing Edge	Associate Resultant Force	Associate Vertical Force without Slamming Force	Associate Moment about the Leading Edge without Slamming Force	Associate Moment about the Trailing Edge without Slamming Force	Maximum Positive Vertical Force	Associate Horizontal Force	Associate Moment about the Leading Edge	Associate Moment about the Trailing Edge	Associate Resultant Force	Associate Vertical Force without Slamming Force	Associate Moment about the Leading Edge without Slamming Force	Associate Moment about the Trailing Edge without Slamming Force	Largest Moment about the Leading Edge	Associate Horizontal Force	Associate Vertical Force	Associate Moment about the Trailing Edge	Associate Resultant Force	Associate Vertical Force without Slamming Force	Associate Moment about the Leading Edge without Slamming Force	Associate Moment about the Trailing Edge without Slamming Force	Lowest Moment about the Trailing Edge	Associate Horizontal Force	Associate Vertical Force	Associate Moment about the Leading Edge	Associate Resultant Force	Associate Vertical Force without Slamming Force	Associate Moment about the Leading Edge without Slamming Force	Associate Moment about the Trailing Edge without Slamming Force
I10-Escambia Bay Span and Conditions	123	37.1	4.7	8.2	4.1	92.3	4.95	1.6	35.3	60	0.5833	3.75	6	28	-133	-2774	1958	136	-133	-2774	1958	20	7	141	-564	21	20	141	-564	180	14	14	-324	20	14	180	-324	-564	7	20	141	21	20	141	-564
I10-Escambia Bay Span and Conditions	131	39.5	4.7	8.2	4.1	92.7	4.94	-0.3	35.3	60	0.5833	3.75	6	33	-51	-248	1570	61	-51	-248	1570	111	5	797	-1381	111	40	413	-997	797	5	111	-1381	111	40	413	-997	-1381	5	111	797	111	40	413	-997
I10 - Lake Pontchartrain	W363	22.1	6.5	12.0	5.1	125.8	8.2	1.2	45.5	65	0.54	3.75	6	271	-29	-665	509	272	-29	-665	509	84	83	384	-2190	118	55	211	-2017	384	83	84	-2190	118	55	211	-2017	-2201	128	71	78	147	52	-13	-2110
I10 - Lake Pontchartrain	W363	22.1	6.5	12.0	5.1	125.8	8.2	4.2	45.5	65	0.54	3.75	6	192	-138	-5110	424	237	-138	-5110	424	27	29	100	-994	40	27	100	-994	100	29	27	-994	40	27	100	-994	-994	29	27	100	40	27	100	-994
I10 - Lake Pontchartrain	W382	21.2	6.4	11.7	5.1	124.3	8.1	-0.8	45.5	65	0.54	3.75	6	240	-81	-474	2760	253	-81	-474	2760	212	88	1036	-3853	229	83	250	-3067	1036	88	212	-3853	229	83	250	-3067	-3853	88	212	1036	229	83	250	-3067
I10 - Lake Pontchartrain	W382	21.2	6.4	11.7	5.1	124.3	8.1	-3.8	45.5	65	0.54	3.75	6	159	34	2740	1359	162	34	2740	1359	631	12	4667	-8945	631	212	2117	-6395	4667	12	631	-8945	631	212	2117	-6395	-8945	12	631	4667	631	212	2117	-6395
I10-Escambia Bay Span Hypothetical Conditions	1	40	4.4	8	3.1	58.7	5.27	3	35.3	60	0.58	3.75	6	101	-51	-309	1481	113	-51	-309	1481	9	24	27	-296	26	9	27	-296	27	16	8	-272	18	8	27	-272	-296	24	9	27	26	9	27	-296
I10-Escambia Bay Span Hypothetical Conditions	2	40	4.4	8	5	128.3	4.48	3	35.3	60	0.58	3.75	6	23	-164	-2861	2966	165	-164	-2861	2966	7	4	26	-224	8	7	26	-224	26	4	7	-224	8	7	26	-224	-224	4	7	26	8	7	26	-224
I10-Escambia Bay Span Hypothetical Conditions	3	40	4.4	8	8	254	4.55	3	35.3	60	0.58	3.75	6	20	-86	-1849	1226	89	-86	-1849	1226	7	3	29	-205	7	7	29	-205	54	6	6	-153	8	6	54	-153	-205	3	7	29	7	7	29	-205
I10-Escambia Bay Span Hypothetical Conditions	4	40	4.4	8	3.1	58.7	5.27	0	35.3	60	0.58	3.75	6	141	-24	-383	475	144	-24	-383	475	21	6	78	-659	22	21	78	-659	78	5	21	-657	21	21	78	-657	-659	6	21	78	22	21	78	-659
I10-Escambia Bay Span Hypothetical Conditions	5	40	4.4	8	5	128.3	4.48	0	35.3	60	0.58	3.75	6	42	-145	-3380	1796	151	-145	-3380	1796	34	14	381	-834	37	34	381	-834	389	28	12	-48	30	12	389	-48	-940	6	33	232	33	33	232	-940
I10-Escambia Bay Span Hypothetical Conditions	6	40	4.4	8	8	254	4.55	0	35.3	60	0.58	3.75	6	27	-1	15	66	27	-1	15	66	39	5	509	-860	39	39	509	-860	674	9	37	-635	38	37	674	-635	-860	5	39	509	39	39	509	-860
I10-Escambia Bay Span Hypothetical Conditions	7	40	4.4	8	3.1	58.7	5.27	-9	35.3	60	0.58	3.75	6	33	219	4313	-3438	222	219	4313	-3438	369	5	6175	-6370	369	348	6064	-6258	6572	18	320	-4741	320	320	6572	-4741	-6731	-3	325	4775	325	325	4775	-6731
I10-Escambia Bay Span Hypothetical Conditions	8	40	4.4	8	5	128.3	4.48	-9	35.3	60	0.58	3.75	6	31	215	4744	-2926	218	215	4744	-2926	378	-7	6656	-6803	378	378	6656	-6803	6681	-3	376	-6705	376	376	6681	-6705	-7035	-2	365	5949	365	365	5949	-7035
I10-Escambia Bay Span Hypothetical Conditions	9	40	4.4	8	8	254	4.55	-9	35.3	60	0.58	3.75	6	34	155	2818	-2700	159	155	2818	-2700	295	2	5193	-5286	295	295	5193	-5286	5256	7	292	-5116	292	292	5256	-5116	-5298	1	294	5141	294	294	5141	-5298
I10-Escambia Bay Span Hypothetical Conditions	10	25	4.4	8	3.1	58.2	5.38	3	35.3	60	0.58	3.75	6	107	-94	-1036	2271	142	-94	-1036	2271	8	25	21	-256	26	8	21	-256	26	16	8	-244	18	8	26	-244	-259	36	8	9	37	8	9	-259
I10-Escambia Bay Span Hypothetical Conditions	11	25	4.4	8	5	118.6	4.77	3	35.3	60	0.58	3.75	6	29	-132	-1620	3069	135	-132	-1620	3069	10	6	40	-309	12	10	40	-309	40	6	10	-309	12	10	40	-309	-309	6	10	40	12	10	40	-309
I10-Escambia Bay Span Hypothetical Conditions	12	25	4.4	8	8	217	5.16	3	35.3	60	0.58	3.75	6	33	-135	-2808	2038	139	-135	-2808	2038	8	13	61	-219	15	8	61	-219	61	13	8	-219	15	8	61	-219	-236	4	7	32	9	7	32	-236
I10-Escambia Bay Span Hypothetical Conditions	13	25	4.4	8	3.1	58.2	5.38	0	35.3	60	0.58	3.75	6	122	-125	-756	3671	175	-125	-756	3671	22	5	83	-679	22	22	83	-679	83	5	22	-679	22	22	83	-679	-679	5	22	83	22	22	83	-679
I10-Escambia Bay Span Hypothetical Conditions	14	25	4.4	8	5	118.6	4.77	0	35.3	60	0.58	3.75	6	51	-56	-43	1950	76	-56	-43	1950	46	9	292	-994	47	32	216	-917	331	23	29	-701	37	29	331	-701	-994	9	46	292	47	32	216	-917
I10-Escambia Bay Span Hypothetical Conditions	15	25	4.4	8	8	217	5.16	0	35.3	60	0.58	3.75	6	52	-32	-80	1076	61	-32	-80	1076	131	13	1017	-1326	132	37	512	-821	1017	13	131	-1326	132	37	512	-821	-1326	13	131	1017	132	37	512	-821
I10-Escambia Bay Span Hypothetical Conditions	16	25	4.4	8	3.1	58.2	5.38	-9	35.3	60	0.58	3.75	6	32	214	4099	-3483	217	214	4099	-3483	360	4	5994	-6213	360	339	5882	-6101	6339	18	310	-4626	311	310	6339	-4626	-6543	-3	318	4695	318	318	4695	-6543
I10-Escambia Bay Span Hypothetical Conditions	17	25	4.4	8	5	118.6	4.77	-9	35.3	60	0.58	3.75	6	38	214	4702	-2902	217	214	4702	-2902	339	-6	5923	-6131	339	339	5923	-6131	5949	5	328	-5726	328	328	5949	-5726	-6371	-2	333	5490	333	333	5490	-6371
I10-Escambia Bay Span Hypothetical Conditions	18	25	4.4	8	8	217	5.16	-9	35.3	60	0.58	3.75	6	50	134	2622	-2169	143	134	2622	-2169	276	8	4895	-4987	276	276	4895	-4987	4945	15	272	-4783	272	272	4945	-4783	-5024	4	274	4798	274	274	4798	-5024
I10-Escambia Bay Span Hypothetical Conditions	19	40	6.7	12	4	95.1	7.66	3	35.3	60	0.58	3.75	6	189	-61	-697	1477	199	-61	-697	1477	29	20	111	-927	36	29	111	-927	111	20	29	-927	36	29	111	-927	-927	20	29	111	36	29	111	-927
I10-Escambia Bay Span Hypothetical Conditions	20	40	6.7	12	6	175.1	7.06	3	35.3	60	0.58	3.75	6	86	-234	-4968	3345	250	-234	-4968	3345	30	22	223	-835	37	30	223	-835	226	45	16	-326	47	16	226	-326	-835	22	30	223	37	30	223	-835
I10-Escambia Bay Span Hypothetical Conditions	21	40	6.7	12	8	259	7.25	3	35.3	60	0.58	3.75	6	68	-97	-1464	1968	118	-97	-1464	1968	28	12	206	-791	30	28	206	-791	314	41	14	-181	43	14	314	-181	-791	12	28	206	30	28	206	-791
I10-Escambia Bay Span Hypothetical Conditions	22	40	6.7	12	4	95.1	7.66	0	35.3	60	0.58	3.75	6	232	-14	-616	-127	232	-14	-616	-127	60	69	197	-1935	91	60	197	-1935	302	48	58	-1767	76	58	302	-1767	-2022	89	60	98	108	60	98	-2022
I10-Escambia Bay Span Hypothetical Conditions	23	40	6.7	12	6	175.1	7.06	0	35.3	60	0.58	3.75	6	114	-36	212	1475	119	-36	212	1475	252	48	1835	-2712	257	74	879	-1755	1835	48	252	-2712	257	74	879	-1755	-2712	48	252	1835	257	74	879	-1755
I10-Escambia Bay Span Hypothetical Conditions	24	40	6.7	12	8	258.6	7.25	0	35.3	60	0.58	3.75	6	107	257	2630	-2155	278	82	1688	-1213	407	106	3474	-3044	421	86	1747	-1318	3474	106	407	-3044	421	86	1747	-1318	-3044	106	407	3474	421	86	1747	-1318
I10-Escambia Bay Span Hypothetical Conditions	25	40	6.7	12	4	95.1	7.66	-9	35.3	60	0.58	3.75	6	66	235	6087	-2254	245	235	6087	-2254	463	0	7289	-8450	463	436	7142	-8303	7454	32	395	-6526	396	395	7454	-6526	-8450	0	463	7289	463	436	7142	-8303
I10-Escambia Bay Span Hypothetical Conditions	26	40	6.7	12	6	175.1	7.06	-9	35.3	60	0.58	3.75	6	63	64	1605	-680	90	64	1605	-680	377	12	6623	-6742	377	377	6623	-6742	6687	13	377	-6669	377	377	6687	-6669	-7079	0	356	5534	356	356	5534	-7079
I10-Escambia Bay Span Hypothetical Conditions	27	40	6.7	12	8	258.6	7.25	-9	35.3	60	0.58	3.75	6	77	113	2023	-1986	137	113	2023	-1986	358	7	6223	-6500	358	358	6223	-6500	6337	17	354	-6252	354	354	6337	-6252	-6520	5	356	6124	356	356	6124	-6520
I10-Escambia Bay Span Hypothetical Conditions	28	25	5.6	10	4	89.1	6.37	3	35.3	60	0.58	3.75	6	113	-178	-1854	4437	211	-178	-1854	4437	20	19	72	-625	28	20	72	-625	72	19	20	-625	28	20	72	-625	-625	19	20	72	28	20	72	-625
I10-Escambia Bay Span Hypothetical Conditions	29	25	5.6	10	6	154.8	6.35	3	35.3	60	0.58	3.75	6	70	-247	-4377	4402	256	-247	-4377	4402	16	7	66	-500	17	16	66	-500	108	33	15	-428	37	15	108	-428	-500	7	16	66	17	16	66	-500
I10-Escambia Bay Span Hypothetical Conditions	30	25	5.6	10	8	221.2	6.77	3	35.3	60	0.58	3.75	6	75	-189	-4139	2542	203	-189	-4139	2542	21	21	142	-593	30	21	142	-593	142	21	21	-593	30	21	142	-593	-593	21	21	142	30	21	142	-593
I10-Escambia Bay Span Hypothetical Conditions	31	25	5.6	10	4	89.11	6.37	0	35.3	60	0.58	3.75	6	149	-40	-369	1043	154	-40	-369	1043	40	31	254	-1149	50	40	254	-1149	254	31	40	-1149	50	40	254	-1149	-1149	31	40	254	50	40	254	-1149
I10-Escambia Bay Span Hypothetical Conditions	32	25	5.6	10	6	154.8	6.35	0	35.3	60	0.58	3.75	6	103	-78	-104	2675	130	-78	-104	2675	184	24	1202	-1885	185	45	455	-1138	1202	24	184	-1885	185	45	455	-1138	-1885	24	184	1202	185	45	455	-1138
I10-Escambia Bay Span Hypothetical Conditions	33	25	5.6	10	8	221.2	6.77	0	35.3	60	0.58	3.75	6	112	-16	253	820	113	-16	253	820	275	24	1858	-2435	276	54	671	-1248	1858	24	275	-2435	276	54	671	-1248	-2435	24	275	1858	276	54	671	-1248
I10-Escambia Bay Span Hypothetical Conditions	34	25	5.6	10	4	89.1	6.37	-9	35.3	60	0.58	3.75	6	53	247	5757	-2987	252	247	5757	-2987	397	-5	6847	-7204	397	396	6844	-7201	6942	3	389	-6824	389	389	6942	-6824	-7279	-8	392	6620	392	392	6620	-7279
I10-Escambia Bay Span Hypothetical Conditions	35	25	5.6	10	6	154.8	6.35	-9	35.3	60	0.58	3.75	6	57	165	3856	-2025	175	165	3856	-2025	333	5	5742	-6114	333	333	5742	-6114	5860	24	319	-5506	320	319	5860	-5506	-6154	-1	331	5640	331	331	5640	-6154
I10-Escambia Bay Span Hypothetical Conditions	36	25	5.6	10	8	221.2	6.77	-9	35.3	60	0.58	3.75	6	75	113	2270	-1714	135	113	2270	-1714	302	13	5201	-5503	303	302	5201	-5503	5313	29	294	-5105	296	294	5313	-5105	-5536	8	300	5081	300	300	5081	-5536
I10-Escambia Bay Span Hypothetical Conditions	101	40	4.4	8	3.1	58.7	5.27	3	35.3	60	0.58	3.75	6	94	-42	-260	1213	103	-42	-260	1213	9	36	15	-306	37	9	15	-306	30	16	9	-280	19	9	30	-280	-306	36	9	15	37	9	15	-306
I10-Escambia Bay Span Hypothetical Conditions	102	40	4.4	8	3.1	58.7	5.27	2	35.3	60	0.58	3.75	6	105	-63	-2112	120	123	-63	-2112	120	17	20	61	-539	26	17	61	-539	61	20	17	-539	26	17	61	-539	-539	20	17	61	26	17	61	-539
I10-Escambia Bay Span Hypothetical Conditions	103	40	4.4	8	3.1	58.7	5.27	1	35.3	60	0.58	3.75	6	104	-89	-2967	169	137	-89	-2967	169	21	12	80	-664	24	21	80	-664	80	12	21	-664	24	21	80	-664	-664	12	21	80	24	21	80	-664
I10-Escambia Bay Span Hypothetical Conditions	104	40	4.4	8	3.1	58.7	5.27	0	35.3	60	0.58	3.75	6	118	-124	-4193	181	171	-124	-4193	181	23	6	89	-718	24	23	89	-718	89	6	23	-718	24	23	89	-718	-718	6	23	89	24	23	89	-718
I10-Escambia Bay Span Hypothetical Conditions	105	40	4.4	8	3.1	58.7	5.27	-1	35.3	60	0.58	3.75	6	140	-34	-484	716	144	-34	-484	716	32	28	205	-940	43	32	205	-940	205	28	32	-940	43	32	205	-940	-972	36	32	163	49	32	163	-972
I10-Escambia Bay Span Hypothetical Conditions	106	40	4.4	8	3.1	58.7	5.27	-2	35.3	60	0.58	3.75	6	142	-32	-402	732	145	-32	-402	732	40	8	258	-1166	41	40	258	-1166	259	10	40	-1160	41	40	259	-1160	-1166	8	40	258	41	40	258	-1166
I10-Escambia Bay Span Hypothetical Conditions	107	40	4.4	8	3.1	58.7	5.27	-3	35.3	60	0.58	3.75	6	138	-58	-174	1887	150	-58	-174	1887	99	0	1490	-2011	99	99	1490	-2011	1496	3	91	-1723	91	91	1496	-1723	-2015	0	99	1484	99	99	1484	-2015
I10-Escambia Bay Span Hypothetical Conditions	108	40	4.4	8	3.1	58.7	5.27	-4	35.3	60	0.58	3.75	6	103	-36	487	1763	110	-36	487	1763	172	-2	2794	-3296	172	172	2794	-3296	2850	-2	171	-3212	171	171	2850	-3212	-3299	-2	172	2769	172	172	2769	-3299
I10-Escambia Bay Span Hypothetical Conditions	109	40	4.4	8	3.1	58.7	5.27	-5	35.3	60	0.58	3.75	6	80	55	1418	-518	97	55	1418	-518	199	2	2963	-4074	199	199	2963	-4074	3000	3	199	-4025	199	199	3000	-4025	-4095	2	199	2926	199	199	2926	-4095
I10-Escambia Bay Span Hypothetical Conditions	110	40	4.4	8	3.1	58.7	5.27	-6	35.3	60	0.58	3.75	6	64	115	2271	-1800	131	115	2271	-1800	289	11	3809	-5229	290	241	3548	-4968	3809	11	289	-5229	290	241	3548	-4968	-5229	11	289	3809	290	241	3548	-4968
I10-Escambia Bay Span Hypothetical Conditions	111	40	4.4	8	3.1	58.7	5.27	-7	35.3	60	0.58	3.75	6	50	181	3450	-2955	188	181	3450	-2955	341	4	4857	-6255	341	302	4647	-6045	5051	27	287	-5111	289	287	5051	-5111	-6255	4	341	4857	341	302	4647	-6045
I10-Escambia Bay Span Hypothetical Conditions	112	40	4.4	8	3.1	58.7	5.27	-8	35.3	60	0.58	3.75	6	39	229	4524	-3578	232	229	4524	-3578	385	7	6414	-6711	385	365	6306	-6603	6817	28	331	-4903	333	331	6817	-4903	-7100	-3	340	4934	340	340	4934	-7100
I10-Escambia Bay Span Hypothetical Conditions	113	40	4.4	8	3.1	58.7	5.27	-9	35.3	60	0.58	3.75	6	33	219	4313	-3438	222	219	4313	-3438	369	5	6175	-6370	369	348	6064	-6258	6572	18	320	-4741	320	320	6572	-4741	-6731	-3	325	4775	325	325	4775	-6731
I10-Escambia Bay Span Hypothetical Conditions	114	40	4.4	8	3.1	58.7	5.27	-12	35.3	60	0.58	3.75	6	19	204	3775	-3425	205	204	3775	-3425	335	2	5578	-5647	335	310	5440	-5509	5849	9	288	-4342	288	288	5849	-4342	-5877	-3	292	4452	292	292	4452	-5877
I10-Escambia Bay Span Hypothetical Conditions	115	40	4.4	8	3.1	58.7	5.27	-15	35.3	60	0.58	3.75	6	12	194	3428	-3446	195	194	3428	-3446	314	1	5163	-5181	314	283	4998	-5016	5301	6	269	-4197	269	269	5301	-4197	-5295	-2	270	4260	270	270	4260	-5295

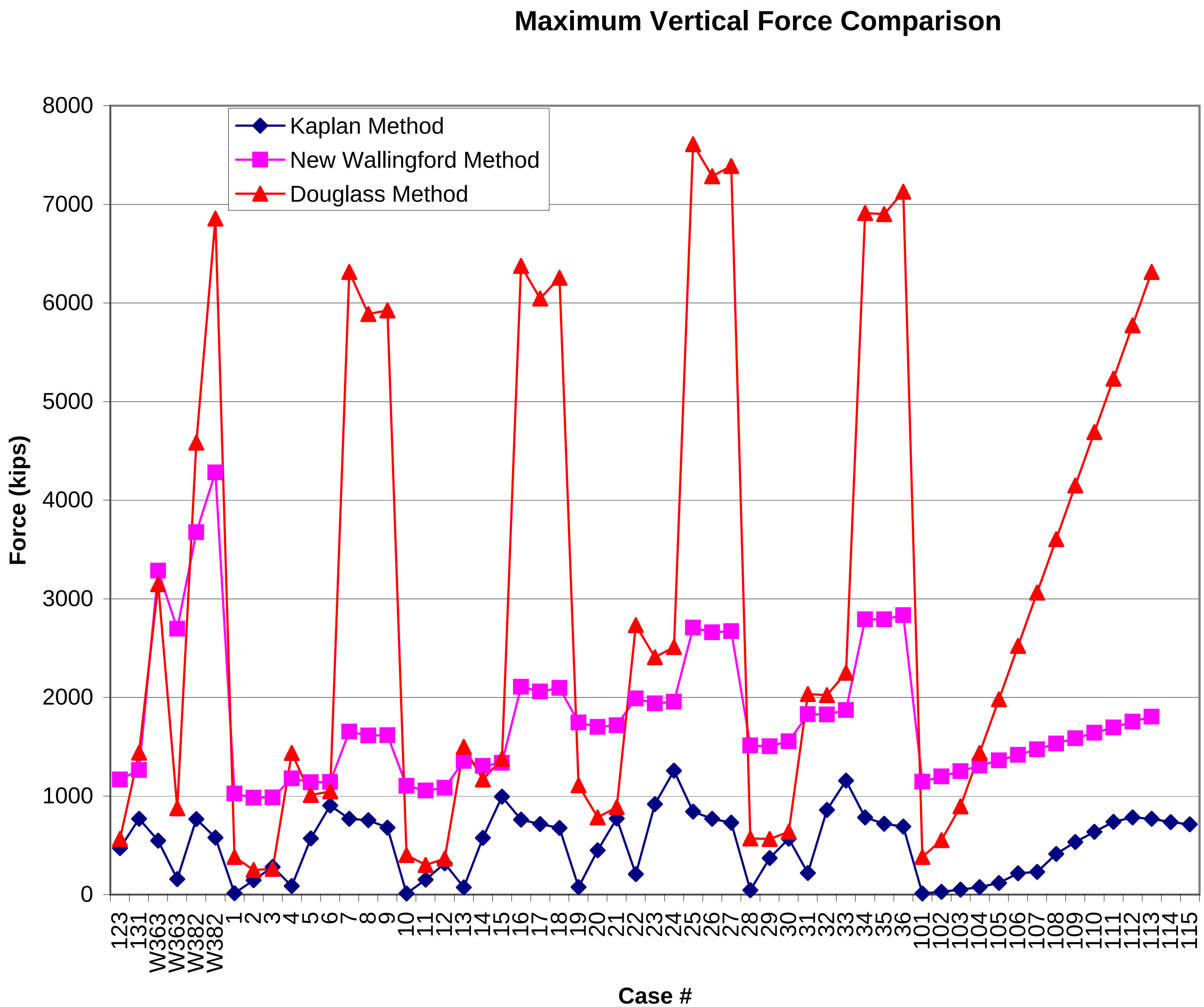
OEA 50% Air

														Maximum Positive Horizontal Force								Maximum Positive Vertical Force								Largest Positive Moment about the Leading Edge								Largest Negative Moment about the Trailing Edge							
Bridge #	Case#	d_s (ft)	Significant Wave Height (ft)	Maximum Wave Height (ft)	Wave Period (sec)	Wave Length (ft)	Wave Crest Height* (ft)	Y_c (ft)	Span Width (ft)	Span Length (ft)	b (ft)	a (ft)	Number of Girders	Maximum Positive Horizontal Force	Associate Vertical Force	Associate Moment about the Leading Edge	Associate Moment about the Trailing Edge	Associate Resultant Force	Associate Vertical Force without Slamming Force	Associate Moment about the Leading Edge without Slamming Force	Associate Moment about the Trailing Edge without Slamming Force	Maximum Positive Vertical Force	Associate Horizontal Force	Associate Moment about the Leading Edge	Associate Moment about the Trailing Edge	Associate Resultant Force	Associate Vertical Force without Slamming Force	Associate Moment about the Leading Edge without Slamming Force	Associate Moment about the Trailing Edge without Slamming Force	Largest Moment about the Leading Edge	Associate Horizontal Force	Associate Vertical Force	Associate Moment about the Trailing Edge	Associate Resultant Force	Associate Vertical Force without Slamming Force	Associate Moment about the Leading Edge without Slamming Force	Associate Moment about the Trailing Edge without Slamming Force	Lowest Moment about the Trailing Edge	Associate Horizontal Force	Associate Vertical Force	Associate Moment about the Leading Edge	Associate Resultant Force	Associate Vertical Force without Slamming Force	Associate Moment about the Leading Edge without Slamming Force	Associate Moment about the Trailing Edge without Slamming Force
I10-Escambia Bay Span and Conditions	123	37.1	4.7	8.2	4.1	92.3	4.95	1.6	35.3	60	0.5833	3.75	6	31	-117	-2626	1532	121	-117	-2626	1532	128	7	724	-1147	128	20	141	-564	724	7	128	-1147	128	20	141	-564	-1147	7	128	724	128	20	141	-564
I10-Escambia Bay Span and Conditions	131	39.5	4.7	8.2	4.1	92.7	4.94	-0.3	35.3	60	0.5833	3.75	6	34	132	2498	-2202	136	132	2498	-2202	467	32	4465	-4264	468	149	2753	-2552	4465	32	467	-4264	468	149	2753	-2552	-4264	32	467	4465	468	149	2753	-2552
I10 - Lake Pontchartrain	W363	22.1	6.5	12.0	5.1	125.8	8.2	1.2	45.5	65	0.54	3.75	6	276	64	-58	-2640	284	64	-58	-2640	240	172	939	-4904	295	104	114	-4080	939	172	240	-4904	295	104	114	-4080	-4904	172	240	939	295	104	114	-4080
I10 - Lake Pontchartrain	W363	22.1	6.5	12.0	5.1	125.8	8.2	4.2	45.5	65	0.54	3.75	6	192	-124	-4589	389	229	-124	-4589	389	41	29	182	-1076	50	27	100	-994	182	29	41	-1076	50	27	100	-994	-1076	29	41	182	50	27	100	-994
I10 - Lake Pontchartrain	W382	21.2	6.4	11.7	5.1	124.3	8.1	-0.8	45.5	65	0.54	3.75	6	246	74	964	-2015	257	74	964	-2015	504	130	2864	-7854	521	164	790	-5780	2864	130	504	-7854	521	164	790	-5780	-7854	130	504	2864	521	164	790	-5780
I10 - Lake Pontchartrain	W382	21.2	6.4	11.7	5.1	124.3	8.1	-3.8	45.5	65	0.54	3.75	6	164	266	6357	-4286	312	266	6357	-4286	732	34	5819	-12664	733	343	3444	-10289	7373	134	241	-2279	276	241	7373	-2279	-12664	34	732	5819	733	343	3444	-10289
I10-Escambia Bay Span Hypothetical Conditions	1	40	4.4	8	3.1	58.7	5.27	3	35.3	60	0.58	3.75	6	110	-58	-383	1650	124	-58	-383	1650	9	24	27	-296	26	9	27	-296	27	16	8	-272	18	8	27	-272	-296	24	9	27	26	9	27	-296
I10-Escambia Bay Span Hypothetical Conditions	2	40	4.4	8	5	128.3	4.48	3	35.3	60	0.58	3.75	6	23	-164	-2861	2966	165	-164	-2861	2966	7	4	26	-224	8	7	26	-224	26	4	7	-224	8	7	26	-224	-224	4	7	26	8	7	26	-224
I10-Escambia Bay Span Hypothetical Conditions	3	40	4.4	8	8	254	4.55	3	35.3	60	0.58	3.75	6	20	-86	-1849	1226	89	-86	-1849	1226	7	3	29	-205	7	7	29	-205	54	6	6	-153	8	6	54	-153	-205	3	7	29	7	7	29	-205
I10-Escambia Bay Span Hypothetical Conditions	4	40	4.4	8	3.1	58.7	5.27	0	35.3	60	0.58	3.75	6	144	16	-230	-788	145	16	-230	-788	21	6	78	-659	22	21	78	-659	78	5	21	-657	21	21	78	-657	-851	142	16	-274	143	16	-274	-851
I10-Escambia Bay Span Hypothetical Conditions	5	40	4.4	8	5	128.3	4.48	0	35.3	60	0.58	3.75	6	56	-84	-1816	1173	101	-84	-1816	1173	233	28	1677	-2039	235	49	685	-1047	1677	28	233	-2039	235	49	685	-1047	-2039	28	233	1677	235	49	685	-1047
I10-Escambia Bay Span Hypothetical Conditions	6	40	4.4	8	8	254	4.55	0	35.3	60	0.58	3.75	6	27	233	3471	-3517	234	181	3192	-3238	529	27	5242	-5196	529	192	3428	-3382	5242	27	529	-5196	529	192	3428	-3382	-5196	27	529	5242	529	192	3428	-3382
I10-Escambia Bay Span Hypothetical Conditions	7	40	4.4	8	3.1	58.7	5.27	-9	35.3	60	0.58	3.75	6	34	412	7455	-7122	414	412	7455	-7122	584	3	9786	-10273	584	560	9654	-10142	10067	14	541	-9054	541	541	10067	-9054	-10483	-4	543	8727	543	543	8727	-10483
I10-Escambia Bay Span Hypothetical Conditions	8	40	4.4	8	5	128.3	4.48	-9	35.3	60	0.58	3.75	6	32	402	7968	-6351	403	402	7968	-6351	580	-8	10248	-10410	580	580	10248	-10410	10255	-5	579	-10375	579	579	10255	-10375	-10552	-1	572	9818	572	572	9818	-10552
I10-Escambia Bay Span Hypothetical Conditions	9	40	4.4	8	8	254	4.55	-9	35.3	60	0.58	3.75	6	35	356	6378	-6271	357	356	6378	-6271	496	2	8740	-8892	496	496	8740	-8892	8775	5	495	-8819	495	495	8775	-8819	-8899	0	495	8691	495	495	8691	-8899
I10-Escambia Bay Span Hypothetical Conditions	10	25	4.4	8	3.1	58.2	5.38	3	35.3	60	0.58	3.75	6	107	-95	-1048	2311	143	-95	-1048	2311	8	25	21	-256	26	8	21	-256	26	16	8	-244	18	8	26	-244	-259	36	8	9	37	8	9	-259
I10-Escambia Bay Span Hypothetical Conditions	11	25	4.4	8	5	118.6	4.77	3	35.3	60	0.58	3.75	6	29	-132	-1620	3069	135	-132	-1620	3069	10	6	40	-309	12	10	40	-309	40	6	10	-309	12	10	40	-309	-309	6	10	40	12	10	40	-309
I10-Escambia Bay Span Hypothetical Conditions	12	25	4.4	8	8	217	5.16	3	35.3	60	0.58	3.75	6	33	-135	-2808	2038	139	-135	-2808	2038	8	13	61	-219	15	8	61	-219	61	13	8	-219	15	8	61	-219	-236	4	7	32	9	7	32	-236
I10-Escambia Bay Span Hypothetical Conditions	13	25	4.4	8	3.1	58.2	5.38	0	35.3	60	0.58	3.75	6	125	-71	-534	1984	143	-71	-534	1984	22	5	83	-679	22	22	83	-679	83	5	22	-679	22	22	83	-679	-679	5	22	83	22	22	83	-679
I10-Escambia Bay Span Hypothetical Conditions	14	25	4.4	8	5	118.6	4.77	0	35.3	60	0.58	3.75	6	55	-93	-1992	1328	108	-93	-1992	1328	249	28	1518	-2373	250	49	444	-1300	1518	28	249	-2373	250	49	444	-1300	-2373	28	249	1518	250	49	444	-1300
I10-Escambia Bay Span Hypothetical Conditions	15	25	4.4	8	8	217	5.16	0	35.3	60	0.58	3.75	6	53	165	2985	-2922	173	165	2985	-2922	574	52	5421	-5258	577	180	3297	-3134	5421	52	574	-5258	577	180	3297	-3134	-5258	52	574	5421	577	180	3297	-3134
I10-Escambia Bay Span Hypothetical Conditions	16	25	4.4	8	3.1	58.2	5.38	-9	35.3	60	0.58	3.75	6	33	405	7202	-7109	406	405	7202	-7109	575	3	9630	-10098	575	551	9497	-9966	9883	15	528	-8787	528	528	9883	-8787	-10278	-3	535	8636	535	535	8636	-10278
I10-Escambia Bay Span Hypothetical Conditions	17	25	4.4	8	5	118.6	4.77	-9	35.3	60	0.58	3.75	6	40	404	7958	-6402	406	404	7958	-6402	542	-7	9549	-9751	542	542	9549	-9751	9556	-5	542	-9737	542	542	9556	-9737	-9924	-1	535	9121	535	535	9121	-9924
I10-Escambia Bay Span Hypothetical Conditions	18	25	4.4	8	8	217	5.16	-9	35.3	60	0.58	3.75	6	50	344	6372	-5934	347	344	6372	-5934	488	9	8712	-8748	488	488	8712	-8748	8714	17	482	-8538	482	482	8714	-8538	-8790	4	487	8634	487	487	8634	-8790
I10-Escambia Bay Span Hypothetical Conditions	19	40	6.7	12	4	95.1	7.66	3	35.3	60	0.58	3.75	6	191	-23	-494	324	193	-23	-494	324	29	20	111	-927	36	29	111	-927	111	20	29	-927	36	29	111	-927	-927	20	29	111	36	29	111	-927
I10-Escambia Bay Span Hypothetical Conditions	20	40	6.7	12	6	175.1	7.06	3	35.3	60	0.58	3.75	6	106	-172	-3678	2434	202	-172	-3678	2434	158	45	1025	-1435	164	31	343	-753	1025	45	158	-1435	164	31	343	-753	-1435	45	158	1025	164	31	343	-753
I10-Escambia Bay Span Hypothetical Conditions	21	40	6.7	12	8	259	7.25	3	35.3	60	0.58	3.75	6	75	-91	-2098	1124	118	-91	-2098	1124	288	67	2399	-2030	296	54	1140	-770	2399	67	288	-2030	296	54	1140	-770	-2030	67	288	2399	296	54	1140	-770
I10-Escambia Bay Span Hypothetical Conditions	22	40	6.7	12	4	95.1	7.66	0	35.3	60	0.58	3.75	6	237	76	37	-2638	248	76	37	-2638	111	123	246	-3702	166	111	246	-3702	552	140	-16	1133	141	-16	552	1133	-3706	120	111	241	164	111	241	-3706
I10-Escambia Bay Span Hypothetical Conditions	23	40	6.7	12	6	175.1	7.06	0	35.3	60	0.58	3.75	6	117	468	5248	-4361	482	185	3725	-2838	581	109	5899	-4928	592	185	3767	-2796	5899	109	581	-4928	592	185	3767	-2796	-4928	109	581	5899	592	185	3767	-2796
I10-Escambia Bay Span Hypothetical Conditions	24	40	6.7	12	8	258.6	7.25	0	35.3	60	0.58	3.75	6	107	270	4893	-4722	291	270	4893	-4722	945	100	8581	-8774	951	290	5054	-5247	8581	100	945	-8774	951	290	5054	-5247	-8774	100	945	8581	951	290	5054	-5247
I10-Escambia Bay Span Hypothetical Conditions	25	40	6.7	12	4	95.1	7.66	-9	35.3	60	0.58	3.75	6	69	409	8886	-5596	415	409	8886	-5596	660	-4	10832	-12109	660	642	10733	-12009	10891	9	630	-11433	630	630	10891	-11433	-12109	-4	660	10832	660	642	10733	-12009
I10-Escambia Bay Span Hypothetical Conditions	26	40	6.7	12	6	175.1	7.06	-9	35.3	60	0.58	3.75	6	64	284	5499	-4583	291	284	5499	-4583	589	9	10156	-10716	589	589	10156	-10716	10343	15	587	-10458	587	587	10343	-10458	-10894	1	573	9412	573	573	9412	-10894
I10-Escambia Bay Span Hypothetical Conditions	27	40	6.7	12	8	258.6	7.25	-9	35.3	60	0.58	3.75	6	77	309	5494	-5476	318	309	5494	-5476	552	9	9659	-9985	553	552	9659	-9985	9710	13	551	-9886	551	551	9710	-9886	-10018	5	551	9564	551	551	9564	-10018
I10-Escambia Bay Span Hypothetical Conditions	28	25	5.6	10	4	89.1	6.37	3	35.3	60	0.58	3.75	6	136	-188	-2026	4623	232	-188	-2026	4623	20	19	72	-625	28	20	72	-625	72	19	20	-625	28	20	72	-625	-625	19	20	72	28	20	72	-625
I10-Escambia Bay Span Hypothetical Conditions	29	25	5.6	10	6	154.8	6.35	3	35.3	60	0.58	3.75	6	74	-236	-4153	4248	247	-236	-4153	4248	105	7	545	-978	105	16	66	-500	545	7	105	-978	105	16	66	-500	-978	7	105	545	105	16	66	-500
I10-Escambia Bay Span Hypothetical Conditions	30	25	5.6	10	8	221.2	6.77	3	35.3	60	0.58	3.75	6	79	-143	-3189	1887	164	-143	-3189	1887	164	21	913	-1365	166	21	142	-593	913	21	164	-1365	166	21	142	-593	-1365	21	164	913	166	21	142	-593
I10-Escambia Bay Span Hypothetical Conditions	31	25	5.6	10	4	89.11	6.37	0	35.3	60	0.58	3.75	6	153	38	118	-1239	157	38	118	-1239	70	105	128	-2344	126	70	128	-2344	348	103	-16	927	104	-16	348	927	-2356	122	69	77	140	69	77	-2356
I10-Escambia Bay Span Hypothetical Conditions	32	25	5.6	10	6	154.8	6.35	0	35.3	60	0.58	3.75	6	107	-99	-2436	1076	145	-99	-2436	1076	501	71	3149	-4984	506	111	1051	-2887	3149	71	501	-4984	506	111	1051	-2887	-4984	71	501	3149	506	111	1051	-2887
I10-Escambia Bay Span Hypothetical Conditions	33	25	5.6	10	8	221.2	6.77	0	35.3	60	0.58	3.75	6	115	505	5204	-4909	518	190	3508	-3213	782	106	6911	-6342	789	196	3762	-3193	6911	106	782	-6342	789	196	3762	-3193	-6342	106	782	6911	789	196	3762	-3193
I10-Escambia Bay Span Hypothetical Conditions	34	25	5.6	10	4	89.1	6.37	-9	35.3	60	0.58	3.75	6	56	432	8838	-6476	436	432	8838	-6476	602	-5	10433	-10890	602	602	10433	-10890	10493	0	596	-10634	596	596	10493	-10634	-10955	-9	599	10259	599	599	10259	-10955
I10-Escambia Bay Span Hypothetical Conditions	35	25	5.6	10	6	154.8	6.35	-9	35.3	60	0.58	3.75	6	59	356	7142	-5551	361	356	7142	-5551	530	3	9244	-9628	530	530	9244	-9628	9320	12	527	-9455	527	527	9320	-9455	-9648	0	529	9194	529	529	9194	-9648
I10-Escambia Bay Span Hypothetical Conditions	36	25	5.6	10	8	221.2	6.77	-9	35.3	60	0.58	3.75	6	76	315	5837	-5316	324	315	5837	-5316	507	12	8850	-9094	507	507	8850	-9094	8912	20	505	-8981	506	505	8912	-8981	-9115	9	505	8774	505	505	8774	-9115
I10-Escambia Bay Span Hypothetical Conditions	101	40	4.4	8	3.1	58.7	5.27	3	35.3	60	0.58	3.75	6	94	-42	-260	1213	103	-42	-260	1213	9	36	15	-306	37	9	15	-306	30	16	9	-280	19	9	30	-280	-306	36	9	15	37	9	15	-306
I10-Escambia Bay Span Hypothetical Conditions	102	40	4.4	8	3.1	58.7	5.27	2	35.3	60	0.58	3.75	6	105	-67	-2239	144	125	-67	-2239	144	17	20	61	-539	26	17	61	-539	61	20	17	-539	26	17	61	-539	-539	20	17	61	26	17	61	-539
I10-Escambia Bay Span Hypothetical Conditions	103	40	4.4	8	3.1	58.7	5.27	1	35.3	60	0.58	3.75	6	104	-89	-2930	221	137	-89	-2930	221	21	12	80	-664	24	21	80	-664	80	12	21	-664	24	21	80	-664	-664	12	21	80	24	21	80	-664
I10-Escambia Bay Span Hypothetical Conditions	104	40	4.4	8	3.1	58.7	5.27	0	35.3	60	0.58	3.75	6	120	-35	-369	866	125	-35	-369	866	23	6	89	-718	24	23	89	-718	89	6	23	-718	24	23	89	-718	-795	94	21	-46	97	21	-46	-795
I10-Escambia Bay Span Hypothetical Conditions	105	40	4.4	8	3.1	58.7	5.27	-1	35.3	60	0.58	3.75	6	143	21	-229	-968	144	21	-229	-968	42	80	39	-1445	90	42	39	-1445	206	29	36	-1054	46	36	206	-1054	-1488	106	40	-80	113	40	-80	-1488
I10-Escambia Bay Span Hypothetical Conditions	106	40	4.4	8	3.1	58.7	5.27	-2	35.3	60	0.58	3.75	6	146	39	-80	-1467	151	39	-80	-1467	82	73	337	-2548	110	82	337	-2548	522	69	-7	753	69	-7	522	753	-2548	73	82	337	110	82	337	-2548
I10-Escambia Bay Span Hypothetical Conditions	107	40	4.4	8	3.1	58.7	5.27	-3	35.3	60	0.58	3.75	6	284	-199	-6476	545	347	-199	-6476	545	104	41	673	-3000	112	104	673	-3000	1344	0	96	-2033	96	96	1344	-2033	-3000	41	104	673	112	104	673	-3000
I10-Escambia Bay Span Hypothetical Conditions	108	40	4.4	8	3.1	58.7	5.27	-4	35.3	60	0.58	3.75	6	108	94	1504	-1810	143	94	1504	-1810	249	3	2927	-4494	249	193	2625	-4191	3073	7	146	-705	146	90	2771	-403	-4494	3	249	2927	249	193	2625	-4191
I10-Escambia Bay Span Hypothetical Conditions	109	40	4.4	8	3.1	58.7	5.27	-5	35.3	60	0.58	3.75	6	84	211	3190	-4264	227	211	3190	-4264	328	4	4310	-6545	329	298	4143	-6379	4548	13	163	-1232	164	163	4548	-1232	-6545	4	328	4310	329	298	4143	-6379
I10-Escambia Bay Span Hypothetical Conditions	110	40	4.4	8	3.1	58.7	5.27	-6	35.3	60	0.58	3.75	6	66	316	5322	-5841	323	316	5322	-5841	441	5	6548	-8565	441	421	6441	-8458	6588	31	414	-8065	416	414	6588	-8065	-8565	5	441	6548	441	421	6441	-8458
I10-Escambia Bay Span Hypothetical Conditions	111	40	4.4	8	3.1	58.7	5.27	-7	35.3	60	0.58	3.75	6	52	390	6925	-6871	394	390	6925	-6871	552	7	8923	-10082	552	531	8809	-9968	8969	18	524	-9554	524	524	8969	-9554	-10101	-1	524	8436	524	524	8436	-10101
I10-Escambia Bay Span Hypothetical Conditions	112	40	4.4	8	3.1	58.7	5.27	-8	35.3	60	0.58	3.75	6	41	420	7660	-7197	422	420	7660	-7197	599	4	10020	-10614	599	577	9897	-10492	10304	23	550	-9129	550	550	10304	-9129	-10865	-3	558	8883	558	558	8883	-10865
I10-Escambia Bay Span Hypothetical Conditions	113	40	4.4	8	3.1	58.7	5.27	-9	35.3	60	0.58	3.75	6	34	412	7455	-7122	414	412	7455	-7122	584	3	9786	-10273	584	560	9654	-10142	10067	14	541	-9054	541	541	10067	-9054	-10483	-4	543	8727	543	543	8727	-10483
I10-Escambia Bay Span Hypothetical Conditions	114	40	4.4	8	3.1	58.7	5.27	-12	35.3	60	0.58	3.75	6	21	395	6917	-7044	395	395	6917	-7044	549	1	9215	-9481	549	520	9057	-9322	9378	9	502	-8357	502	502	9378	-8357	-9583	-3	507	8345	507	507	8345	-9583
I10-Escambia Bay Span Hypothetical Conditions	115	40	4.4	8	3.1	58.7	5.27	-15	35.3	60	0.58	3.75	6	13	382	6423	-7068	382	382	6423	-7068	526	1	8811	-8937	526	491	8626	-8752	8851	6	479	-8080	479	479	8851	-8080	-8959	-3	482	8088	482	482	8088	-8959

OEA 100% Air

Bridge #	Case#	d_s (ft)	Significant Wave Height (ft)	Maximum Wave Height (ft)	Wave Period (sec)	Wave Length (ft)	Wave Crest Height* (ft)	Y_c (ft)	Span Width (ft)	Span Length (ft)	b (ft)	a (ft)	Number of Girders	Maximum Positive Horizontal Force								Maximum Positive Vertical Force								Largest Positive Moment about the Leading Edge								Largest Negative Moment about the Trailing Edge							
														Maximum Positive Horizontal Force	Associate Vertical Force	Associate Moment about the Leading Edge	Associate Moment about the Trailing Edge	Associate Resultant Force	Associate Vertical Force without Slamming Force	Associate Moment about the Leading Edge without Slamming Force	Associate Moment about the Trailing Edge without Slamming Force	Maximum Positive Vertical Force	Associate Horizontal Force	Associate Moment about the Leading Edge	Associate Moment about the Trailing Edge	Associate Resultant Force	Associate Vertical Force without Slamming Force	Associate Moment about the Leading Edge without Slamming Force	Associate Moment about the Trailing Edge without Slamming Force	Largest Moment about the Leading Edge	Associate Horizontal Force	Associate Vertical Force	Associate Moment about the Trailing Edge	Associate Resultant Force	Associate Vertical Force without Slamming Force	Associate Moment about the Leading Edge without Slamming Force	Associate Moment about the Trailing Edge without Slamming Force	Lowest Moment about the Trailing Edge	Associate Horizontal Force	Associate Vertical Force	Associate Moment about the Leading Edge	Associate Resultant Force	Associate Vertical Force without Slamming Force	Associate Moment about the Leading Edge without Slamming Force	Associate Moment about the Trailing Edge without Slamming Force
I10-Escambia Bay Span and Conditions	123	37.1	4.7	8.2	4.1	92.3	4.95	1.6	35.3	60	0.5833	3.75	6	41	25	706	-193	48	25	706	-193	471	26	4087	-3693	472	109	2139	-1746	4087	26	471	-3693	472	109	2139	-1746	-3693	26	471	4087	472	109	2139	-1746
I10-Escambia Bay Span and Conditions	131	39.5	4.7	8.2	4.1	92.7	4.94	-0.3	35.3	60	0.5833	3.75	6	34	322	5648	-5824	324	322	5648	-5824	769	32	8360	-8296	769	337	6040	-5976	8360	32	769	-8296	769	337	6040	-5976	-8296	32	769	8360	769	337	6040	-5976
I10 - Lake Pontchartrain	W363	22.1	6.5	12.0	5.1	125.8	8.2	1.2	45.5	65	0.54	3.75	6	280	317	1981	-7264	422	192	1221	-6504	548	238	3069	-9189	597	199	940	-7061	3069	238	548	-9189	597	199	940	-7061	-9189	238	548	3069	597	199	940	-7061
I10 - Lake Pontchartrain	W363	22.1	6.5	12.0	5.1	125.8	8.2	4.2	45.5	65	0.54	3.75	6	199	-106	-1032	3252	225	-106	-1032	3252	156	145	812	-2926	213	65	261	-2374	812	145	156	-2926	213	65	261	-2374	-2926	145	156	812	213	65	261	-2374
I10 - Lake Pontchartrain	W382	21.2	6.4	11.7	5.1	124.3	8.1	-0.8	45.5	65	0.54	3.75	6	249	278	3788	-7336	373	278	3788	-7336	765	220	5422	-12280	796	300	2586	-9445	6243	90	155	27	179	155	6243	27	-12280	220	765	5422	796	300	2586	-9445
I10 - Lake Pontchartrain	W382	21.2	6.4	11.7	5.1	124.3	8.1	-3.8	45.5	65	0.54	3.75	6	165	522	11223	-9702	548	522	11223	-9702	577	114	9166	-13319	588	554	9026	-13179	11644	151	491	-8029	514	491	11644	-8029	-14579	33	536	6888	537	536	6888	-14579
I10-Escambia Bay Span Hypothetical Conditions	1	40	4.4	8	3.1	58.7	5.27	3	35.3	60	0.58	3.75	6	194	-14	-253	242	194	-14	-253	242	13	36	36	-437	38	13	36	-437	37	25	13	-405	28	13	37	-405	-450	48	13	22	50	13	22	-450
I10-Escambia Bay Span Hypothetical Conditions	2	40	4.4	8	5	128.3	4.48	3	35.3	60	0.58	3.75	6	23	-71	-807	1708	74	-71	-807	1708	146	14	818	-1245	146	20	141	-568	818	14	146	-1245	146	20	141	-568	-1245	14	146	818	146	20	141	-568
I10-Escambia Bay Span Hypothetical Conditions	3	40	4.4	8	8	254	4.55	3	35.3	60	0.58	3.75	6	40	-1	32	79	40	-1	32	79	279	14	1849	-2135	280	39	557	-844	1849	14	279	-2135	280	39	557	-844	-2135	14	279	1849	280	39	557	-844
I10-Escambia Bay Span Hypothetical Conditions	4	40	4.4	8	3.1	58.7	5.27	0	35.3	60	0.58	3.75	6	146	85	170	-2828	169	85	170	-2828	88	123	221	-2874	151	88	221	-2874	628	79	41	-822	89	41	628	-822	-2916	143	86	141	167	86	141	-2916
I10-Escambia Bay Span Hypothetical Conditions	5	40	4.4	8	5	128.3	4.48	0	35.3	60	0.58	3.75	6	73	68	1646	-770	100	68	1646	-770	570	32	6057	-5473	571	217	4158	-3574	6057	32	570	-5473	571	217	4158	-3574	-5473	32	570	6057	571	217	4158	-3574
I10-Escambia Bay Span Hypothetical Conditions	6	40	4.4	8	8	254	4.55	0	35.3	60	0.58	3.75	6	44	90	2136	-1054	100	90	2136	-1054	903	27	9493	-9501	903	374	6652	-6659	9493	27	903	-9501	903	374	6652	-6659	-9501	27	903	9493	903	374	6652	-6659
I10-Escambia Bay Span Hypothetical Conditions	7	40	4.4	8	3.1	58.7	5.27	-9	35.3	60	0.58	3.75	6	34	579	10185	-10298	580	579	10185	-10298	768	2	12955	-13519	768	740	12803	-13368	13187	14	718	-12220	719	718	13187	-12220	-13668	-4	726	12007	726	726	12007	-13668
I10-Escambia Bay Span Hypothetical Conditions	8	40	4.4	8	5	128.3	4.48	-9	35.3	60	0.58	3.75	6	33	572	10961	-9396	573	572	10961	-9396	753	-7	13364	-13459	753	753	13364	-13459	13366	-6	753	-13440	753	753	13366	-13440	-13558	-1	742	12865	742	742	12865	-13558
I10-Escambia Bay Span Hypothetical Conditions	9	40	4.4	8	8	254	4.55	-9	35.3	60	0.58	3.75	6	35	540	9645	-9567	541	540	9645	-9567	678	2	11970	-12151	678	678	11970	-12151	11991	6	677	-12069	677	677	11991	-12069	-12163	0	677	11924	677	677	11924	-12163
I10-Escambia Bay Span Hypothetical Conditions	10	25	4.4	8	3.1	58.2	5.38	3	35.3	60	0.58	3.75	6	112	-16	-203	362	113	-16	-203	362	11	36	19	-358	38	11	19	-358	33	16	10	-325	19	10	33	-325	-358	36	11	19	38	11	19	-358
I10-Escambia Bay Span Hypothetical Conditions	11	25	4.4	8	5	118.6	4.77	3	35.3	60	0.58	3.75	6	32	-52	-477	1367	61	-52	-477	1367	151	25	852	-1341	153	23	159	-648	852	25	151	-1341	153	23	159	-648	-1341	25	151	852	153	23	159	-648
I10-Escambia Bay Span Hypothetical Conditions	12	25	4.4	8	8	217	5.16	3	35.3	60	0.58	3.75	6	40	-13	-138	336	43	-13	-138	336	316	28	2050	-2625	318	51	621	-1196	2050	28	316	-2625	318	51	621	-1196	-2625	28	316	2050	318	51	621	-1196
I10-Escambia Bay Span Hypothetical Conditions	13	25	4.4	8	3.1	58.2	5.38	0	35.3	60	0.58	3.75	6	129	-91	-915	2316	158	-91	-915	2316	72	91	233	-2314	116	72	233	-2314	350	35	66	-1975	75	66	350	-1975	-2329	88	72	209	114	72	209	-2329
I10-Escambia Bay Span Hypothetical Conditions	14	25	4.4	8	5	118.6	4.77	0	35.3	60	0.58	3.75	6	61	75	1685	-978	97	75	1685	-978	576	44	5923	-5098	578	194	3868	-3043	5923	44	576	-5098	578	194	3868	-3043	-5098	44	576	5923	578	194	3868	-3043
I10-Escambia Bay Span Hypothetical Conditions	15	25	4.4	8	8	217	5.16	0	35.3	60	0.58	3.75	6	54	405	6552	-6830	408	361	6315	-6593	992	52	10050	-9968	993	373	6724	-6642	10050	52	992	-9968	993	373	6724	-6642	-9968	52	992	10050	993	373	6724	-6642
I10-Escambia Bay Span Hypothetical Conditions	16	25	4.4	8	3.1	58.2	5.38	-9	35.3	60	0.58	3.75	6	33	571	9890	-10284	572	571	9890	-10284	759	2	12808	-13343	759	731	12655	-13190	13009	14	709	-12049	709	709	13009	-12049	-13463	-4	718	11916	718	718	11916	-13463
I10-Escambia Bay Span Hypothetical Conditions	17	25	4.4	8	5	118.6	4.77	-9	35.3	60	0.58	3.75	6	41	575	10972	-9495	577	575	10972	-9495	717	-8	12683	-12817	717	717	12683	-12817	12686	-6	716	-12802	716	716	12686	-12802	-12948	-1	707	12197	707	707	12197	-12948
I10-Escambia Bay Span Hypothetical Conditions	18	25	4.4	8	8	217	5.16	-9	35.3	60	0.58	3.75	6	51	538	9879	-9393	541	538	9879	-9393	677	9	12151	-12092	677	677	12151	-12092	12151	9	677	-12092	677	677	12151	-12092	-12144	4	677	12085	677	677	12085	-12144
I10-Escambia Bay Span Hypothetical Conditions	19	40	6.7	12	4	95.1	7.66	3	35.3	60	0.58	3.75	6	195	-111	-1063	2857	224	-111	-1063	2857	77	150	258	-2469	169	77	258	-2469	359	93	74	-2273	119	74	359	-2273	-2516	169	77	207	186	77	207	-2516
I10-Escambia Bay Span Hypothetical Conditions	20	40	6.7	12	6	175.1	7.06	3	35.3	60	0.58	3.75	6	135	-16	-57	522	136	-16	-57	522	448	77	3635	-4440	454	132	1937	-2742	3635	77	448	-4440	454	132	1937	-2742	-4440	77	448	3635	454	132	1937	-2742
I10-Escambia Bay Span Hypothetical Conditions	21	40	6.7	12	8	259	7.25	3	35.3	60	0.58	3.75	6	92	44	1238	-330	102	44	1238	-330	773	67	7100	-6978	775	233	4196	-4074	7100	67	773	-6978	775	233	4196	-4074	-6978	67	773	7100	775	233	4196	-4074
I10-Escambia Bay Span Hypothetical Conditions	22	40	6.7	12	4	95.1	7.66	0	35.3	60	0.58	3.75	6	239	201	1378	-5743	312	201	1378	-5743	207	204	1103	-6245	291	207	1103	-6245	3039	103	98	-433	142	98	3039	-433	-6277	200	207	1070	288	207	1070	-6277
I10-Escambia Bay Span Hypothetical Conditions	23	40	6.7	12	6	175.1	7.06	0	35.3	60	0.58	3.75	6	118	371	6715	-6459	390	371	6715	-6459	918	103	9203	-9929	924	375	6278	-7005	9203	103	918	-9929	924	375	6278	-7005	-9929	103	918	9203	924	375	6278	-7005
I10-Escambia Bay Span Hypothetical Conditions	24	40	6.7	12	8	258.6	7.25	0	35.3	60	0.58	3.75	6	107	450	8016	-7970	462	450	8016	-7970	1257	100	12321	-12797	1261	468	8076	-8553	12321	100	1257	-12797	1261	468	8076	-8553	-12797	100	1257	12321	1261	468	8076	-8553
I10-Escambia Bay Span Hypothetical Conditions	25	40	6.7	12	4	95.1	7.66	-9	35.3	60	0.58	3.75	6	70	580	11792	-8757	584	580	11792	-8757	841	-5	14048	-15246	841	821	13936	-15134	14048	-5	841	-15246	841	821	13936	-15134	-15246	-5	841	14048	841	821	13936	-15134
I10-Escambia Bay Span Hypothetical Conditions	26	40	6.7	12	6	175.1	7.06	-9	35.3	60	0.58	3.75	6	64	469	8740	-7904	474	469	8740	-7904	768	8	13291	-13962	768	768	13291	-13962	13509	16	765	-13623	765	765	13509	-13623	-14086	1	754	12642	754	754	12642	-14086
I10-Escambia Bay Span Hypothetical Conditions	27	40	6.7	12	8	258.6	7.25	-9	35.3	60	0.58	3.75	6	77	492	8740	-8748	498	492	8740	-8748	730	9	12800	-13147	730	730	12800	-13147	12834	14	728	-13038	728	728	12834	-13038	-13195	5	727	12656	727	727	12656	-13195
I10-Escambia Bay Span Hypothetical Conditions	28	25	5.6	10	4	89.1	6.37	3	35.3	60	0.58	3.75	6	197	-67	-649	1738	209	-67	-649	1738	45	71	221	-1387	85	45	221	-1387	225	86	44	-1338	96	44	225	-1338	-1409	84	45	195	96	45	195	-1409
I10-Escambia Bay Span Hypothetical Conditions	29	25	5.6	10	6	154.8	6.35	3	35.3	60	0.58	3.75	6	87	-60	-593	1534	105	-60	-593	1534	371	65	2359	-3586	376	79	790	-2018	2359	65	371	-3586	376	79	790	-2018	-3586	65	371	2359	376	79	790	-2018
I10-Escambia Bay Span Hypothetical Conditions	30	25	5.6	10	8	221.2	6.77	3	35.3	60	0.58	3.75	6	87	19	520	-157	89	19	520	-157	568	73	4197	-4867	572	120	1788	-2458	4197	73	568	-4867	572	120	1788	-2458	-4867	73	568	4197	572	120	1788	-2458
I10-Escambia Bay Span Hypothetical Conditions	31	25	5.6	10	4	89.11	6.37	0	35.3	60	0.58	3.75	6	155	176	1401	-4425	234	160	1315	-4339	218	149	1302	-5050	264	162	1003	-4750	2336	87	99	-1155	132	99	2336	-1155	-5050	149	218	1302	264	162	1003	-4750
I10-Escambia Bay Span Hypothetical Conditions	32	25	5.6	10	6	154.8	6.35	0	35.3	60	0.58	3.75	6	113	89	1821	-1332	143	89	1821	-1332	857	77	8787	-7653	861	290	5738	-4603	8787	77	857	-7653	861	290	5738	-4603	-7653	77	857	8787	861	290	5738	-4603
I10-Escambia Bay Span Hypothetical Conditions	33	25	5.6	10	8	221.2	6.77	0	35.3	60	0.58	3.75	6	116	822	9047	-9198	830	382	6677	-6828	1156	107	11081	-10834	1161	385	6934	-6687	11081	107	1156	-10834	1161	385	6934	-6687	-10834	107	1156	11081	1161	385	6934	-6687
I10-Escambia Bay Span Hypothetical Conditions	34	25	5.6	10	4	89.1	6.37	-9	35.3	60	0.58	3.75	6	56	607	11843	-9674	610	607	11843	-9674	781	-6	13601	-14080	781	781	13601	-14080	13663	-4	780	-13969	780	780	13663	-13969	-14140	-10	779	13450	779	779	13450	-14140
I10-Escambia Bay Span Hypothetical Conditions	35	25	5.6	10	6	154.8	6.35	-9	35.3	60	0.58	3.75	6	60	550	10544	-9022	553	550	10544	-9022	719	2	12638	-12949	719	719	12638	-12949	12698	13	714	-12740	715	714	12698	-12740	-12962	0	718	12603	718	718	12603	-12962
I10-Escambia Bay Span Hypothetical Conditions	36	25	5.6	10	8	221.2	6.77	-9	35.3	60	0.58	3.75	6	76	506	9231	-8666	511	506	9231	-8666	690	11	12148	-12295	691	690	12148	-12295	12198	20	688	-12159	688	688	12198	-12159	-12309	9	689	12086	689	689	12086	-12309
I10-Escambia Bay Span Hypothetical Conditions	101	40	4.4	8	3.1	58.7	5.27	3	35.3	60	0.58	3.75	6	169	-14	-218	265	170	-14	-218	265	13	37	30	-415	39	13	30	-415	38	17	11	-367	20	11	38	-367	-417	51	12	13	53	12	13	-417
I10-Escambia Bay Span Hypothetical Conditions	102	40	4.4	8	3.1	58.7	5.27	2	35.3	60	0.58	3.75	6	105	-34	-1183	15	110	-34	-1183	15	28	50	96	-894	57	28	96	-894	107	40	28	-877	49	28	107	-877	-898	63	28	78	69	28	78	-898
I10-Escambia Bay Span Hypothetical Conditions	103	40	4.4	8	3.1	58.7	5.27	1	35.3	60	0.58	3.75	6	110	-53	-438	1424	122	-53	-438	1424	50	70	201	-1554	86	50	201	-1554	230	49	49	-1500	70	49	230	-1500	-1567	85	49	171	98	49	171	-1567
I10-Escambia Bay Span Hypothetical Conditions	104	40	4.4	8	3.1	58.7	5.27	0	35.3	60	0.58	3.75	6	122	62	348	-1851	137	62	348	-1851	77	98	286	-2437	125	77	286	-2437	418	107	37	-901	114	37	418	-901	-2461	108	77	261	132	77	261	-2461
I10-Escambia Bay Span Hypothetical Conditions	105	40	4.4	8	3.1	58.7	5.27	-1	35.3	60	0.58	3.75	6	144	113	487	-3503	183	113	487	-3503	117	120	499	-3654	168	117	499	-3654	1232	66	63	-996	91	63	1232	-996	-3709	141	116	404	183	116	404	-3709
I10-Escambia Bay Span Hypothetical Conditions	106	40	4.4	8	3.1	58.7	5.27	-2	35.3	60	0.58	3.75	6	147	155	1093	-4405	214	155	1093	-4405	217	96	1325	-5010	237	162	1030	-4715	2719	39	160	-1591	165	105	2424	-1296	-5052	111	213	1228	240	162	957	-4782
I10-Escambia Bay Span Hypothetical Conditions	107	40	4.4	8	3.1	58.7	5.27	-3	35.3	60	0.58	3.75	6	494	12	-1971	-2393	495	12	-1971	-2393	229	38	2180	-5312	232	204	2045	-5177	4414	15	211	-2421	211	186	4280	-2286	-5776	54	216	1862	223	216	1862	-5776
I10-Escambia Bay Span Hypothetical Conditions	108	40	4.4	8	3.1	58.7	5.27	-4	35.3	60	0.58	3.75	6	108	307	4595	-6272	326	307	4595	-6272	413	11	5902	-8205	413	393	5794	-8098	6232	7	310	-4253	310	290	6125	-4146	-8205	11	413	5902	413	393	5794	-8098
I10-Escambia Bay Span Hypothetical Conditions	109	40	4.4	8	3.1	58.7	5.27	-5	35.3	60	0.58	3.75	6	83	411	6412	-8135	420	411	6412	-8135	534	6	8168	-10180	534	512	8052	-10065	8168	6	534	-10180	534	512	8052	-10065	-10180	6	534	8168	534	512	8052	-10065
I10-Escambia Bay Span Hypothetical Conditions	110	40	4.4	8	3.1	58.7	5.27	-6	35.3	60	0.58	3.75	6	66	496	8292	-9260	501	496	8292	-9260	636	4	10037	-11889	636	613	9912	-11765	10086	30	608	-11409	609	608	10086	-11409	-11889	4	636	10037	636	613	9912	-11765
I10-Escambia Bay Span Hypothetical Conditions	111	40	4.4	8	3.1	58.7	5.27	-7	35.3	60	0.58	3.75	6	51	559	9701	-10081	562	559	9701	-10081	739	5	12165	-13342	739	714	12031	-13208	12197	17	705	-12722	705	705	12197	-12722	-13342	5	739	12165	739	714	12031	-13208
I10-Escambia Bay Span Hypothetical Conditions	112	40	4.4	8	3.1	58.7	5.27	-8	35.3	60	0.58	3.75	6	41	586	10368	-10357	588	586	10368	-10357	783	3	13152	-13870	783	756	13009	-13727	13399	22	730	-12424	731	730	13399	-12424	-14039	-3	741	12167	741	741	12167	-14039
I10-Escambia Bay Span Hypothetical Conditions	113	40	4.4	8	3.1	58.7	5.27	-9	35.3	60	0.58	3.75	6	34	579	10185	-10298	580	579	10185	-10298	768	2	12955	-13519	768	740	12803	-13368	13187	14	718	-12220	719	718	13187	-12220	-13668	-4	726	12007	726	726	12007	-13668
I10-Escambia Bay Span Hypothetical Conditions	114	40	4.4	8	3.1	58.7	5.27	-12	35.3	60	0.58	3.75	6	21	563	9658	-10240	563	563	9658	-10240	735	1	12428	-12733	735	701	12250	-12555	12537	9	685	-11670	685	685	12537	-11670	-12789	-3	691	11636	691	691	11636	-12789
I10-Escambia Bay Span Hypothetical Conditions	115	40	4.4	8	3.1	58.7	5.27	-15	35.3	60	0.58	3.75	6	13	547	9090	-10251	547	547	9090	-10251	712	1	12038	-12202	712	674	11833	-11997	12039	6	662	-11384	662	662	12039	-11384	-12202	1	712	12038	712	674	11833	-11997


Maximum Horizontal Force Comparison
Kaplan Method
New Wallingford Method
Douglass Method
Force (kips)
3000
2500
2000
1500
1000
500
0
123
131
W363
W363
W382
W382
1
2
3
4
5
6
7
8
9
10
11
12
13
14
15
16
17
18
19
20
21
22
23
24
25
26
27
28
29
30
31
32
33
34
35
36
101
102
103
104
105
106
107
108
109
110
111
112
113
114
115
Case #


Maximum Vertical Force Comparison
Kaplan Method
New Wallingford Method
Douglass Method
Force (kips)
0
1000
2000
3000
4000
5000
6000
7000
8000
Case #
123
131
W363
W363
W382
W382
1
2
3
4
5
6
7
8
9
10
11
12
13
14
15
16
17
18
19
20
21
22
23
24
25
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27
28
29
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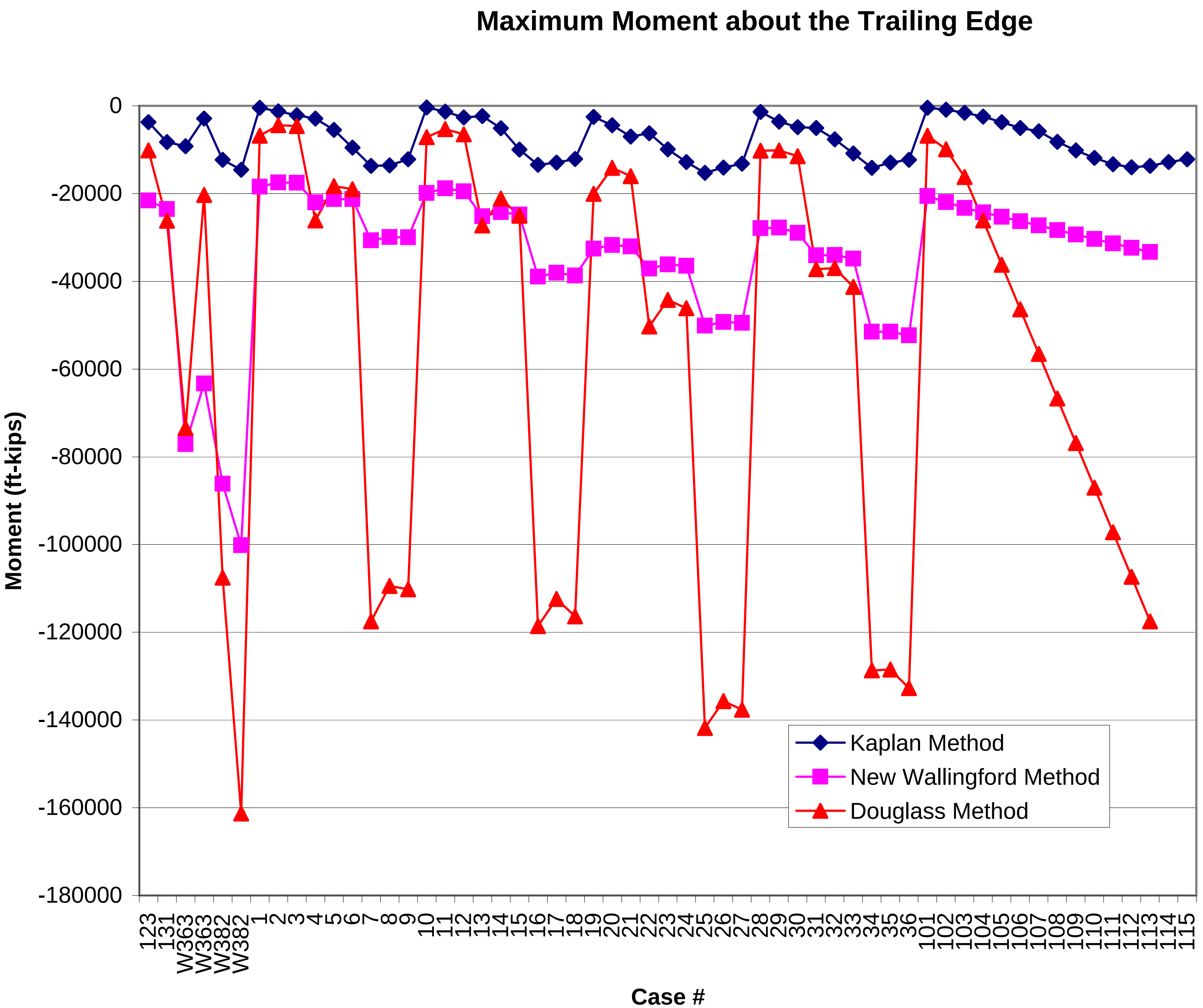
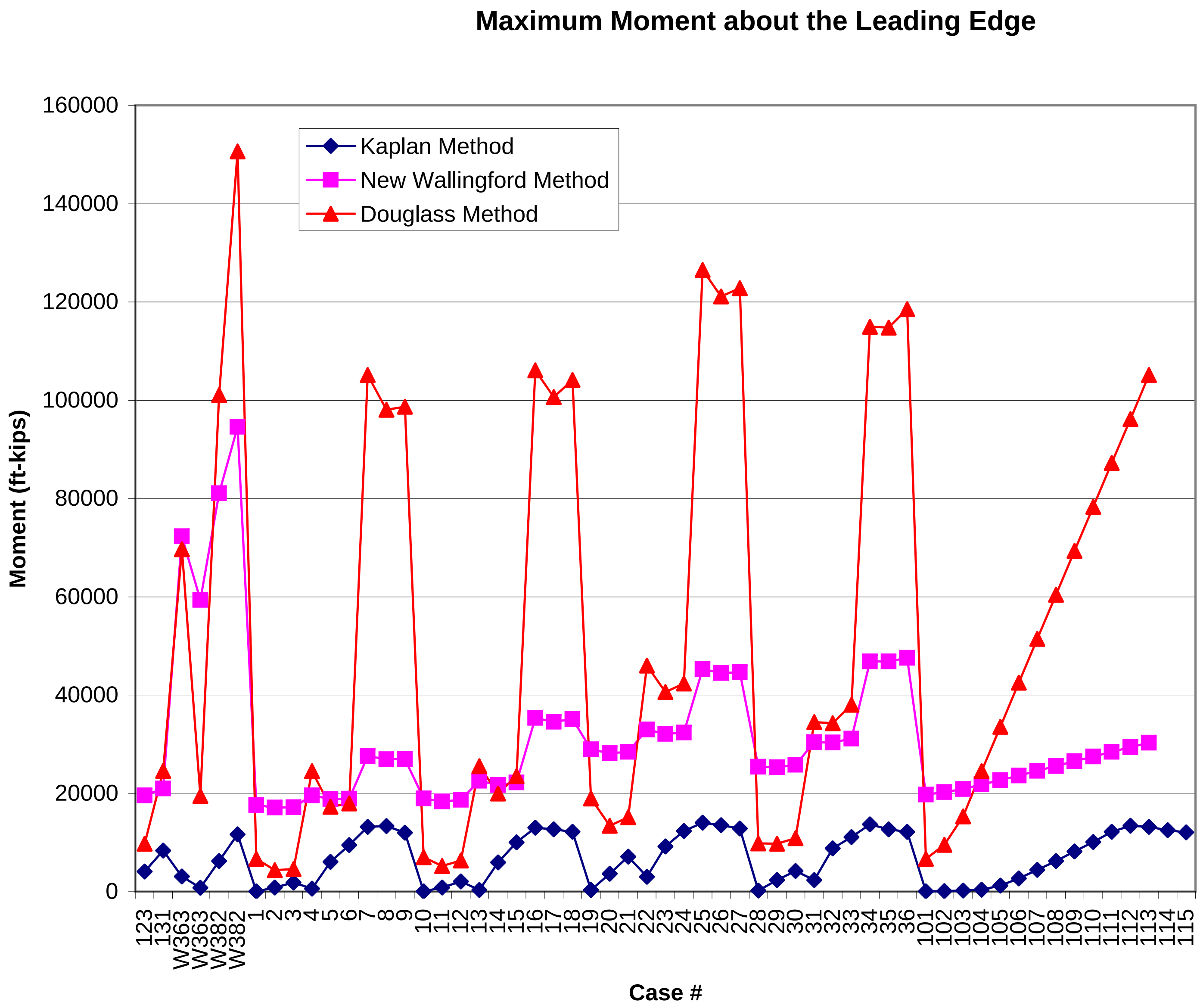
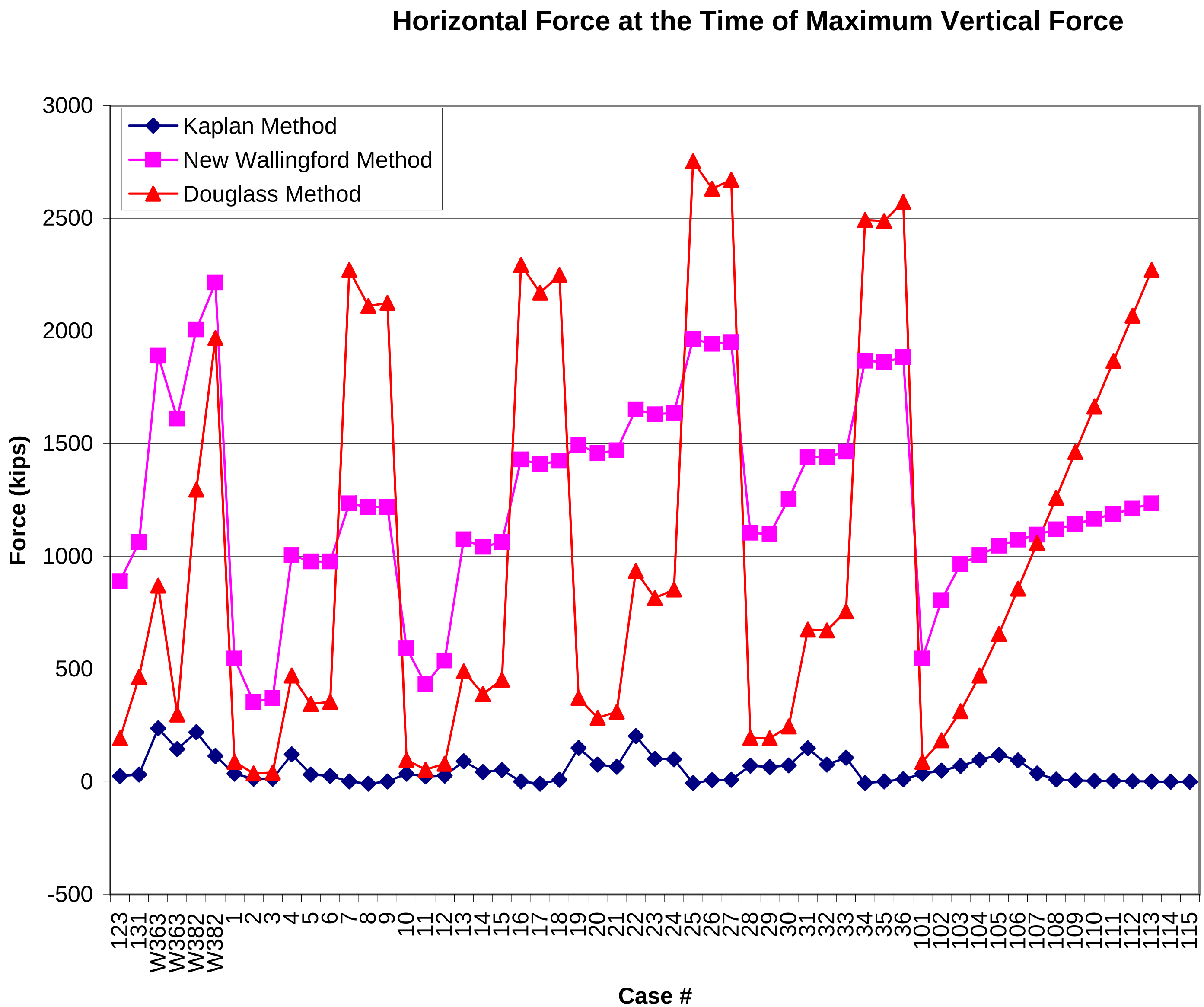
113
114
115


Maximum Moment about the Trailing Edge
Moment (ft-kips)
0
-20000
-40000
-60000
-80000
-100000
-120000
-140000
-160000
-180000
Case #
Kaplan Method
New Wallingford Method
Douglass Method

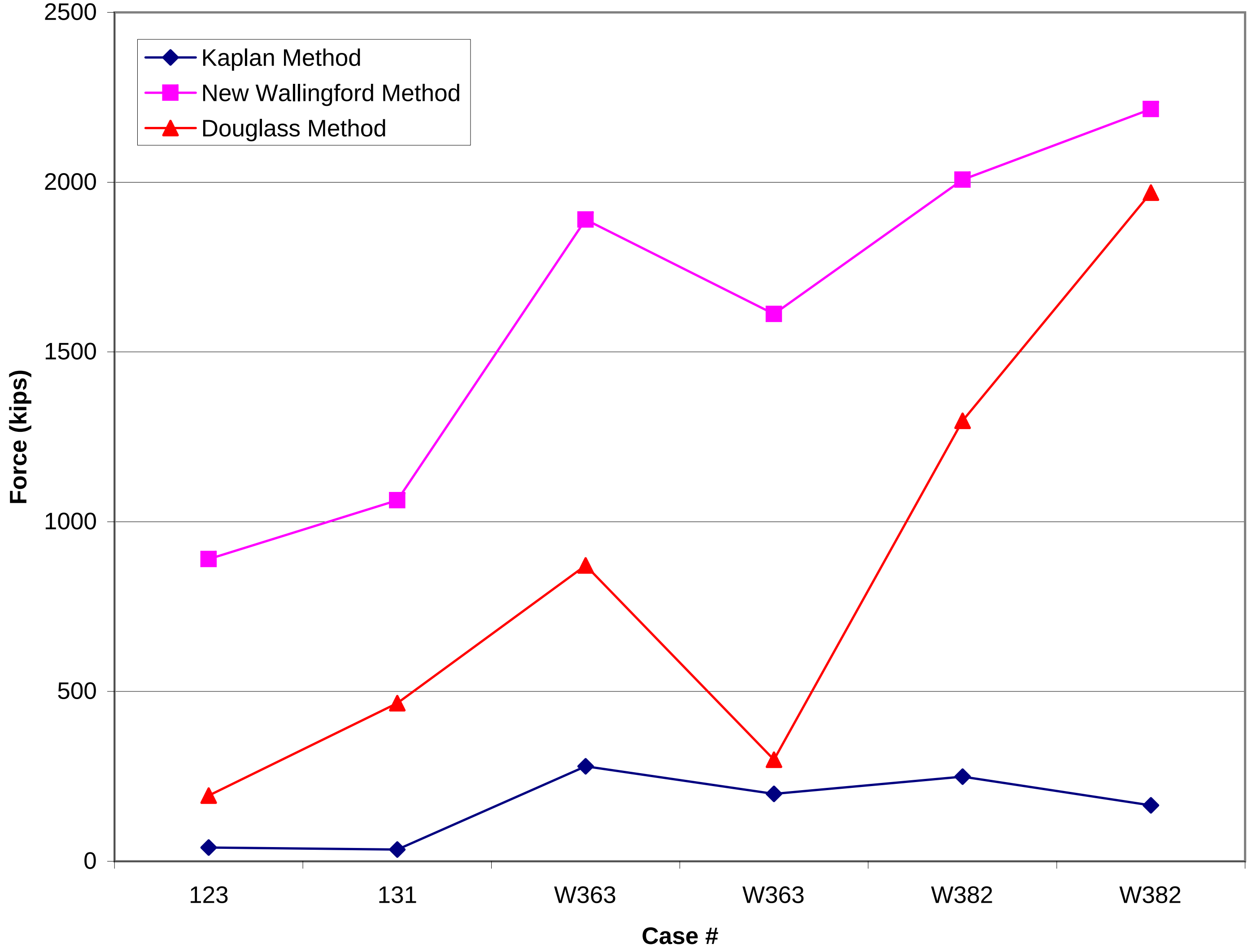


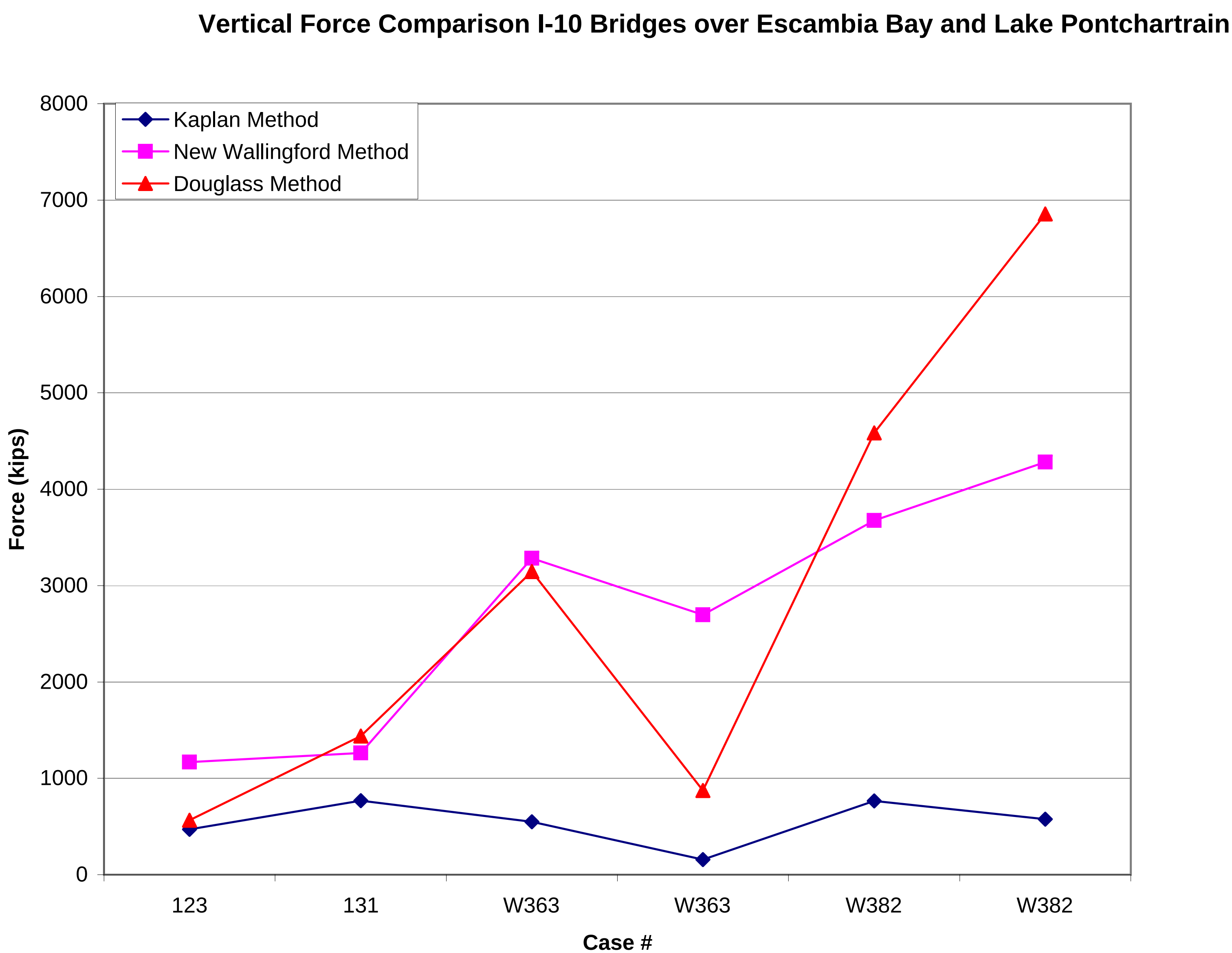
Maximum Moment about the Leading Edge
Kaplan Method
New Wallingford Method
Douglass Method
Moment (ft-kips)
160000
140000
120000
100000
80000
60000
40000
20000
0
Case #

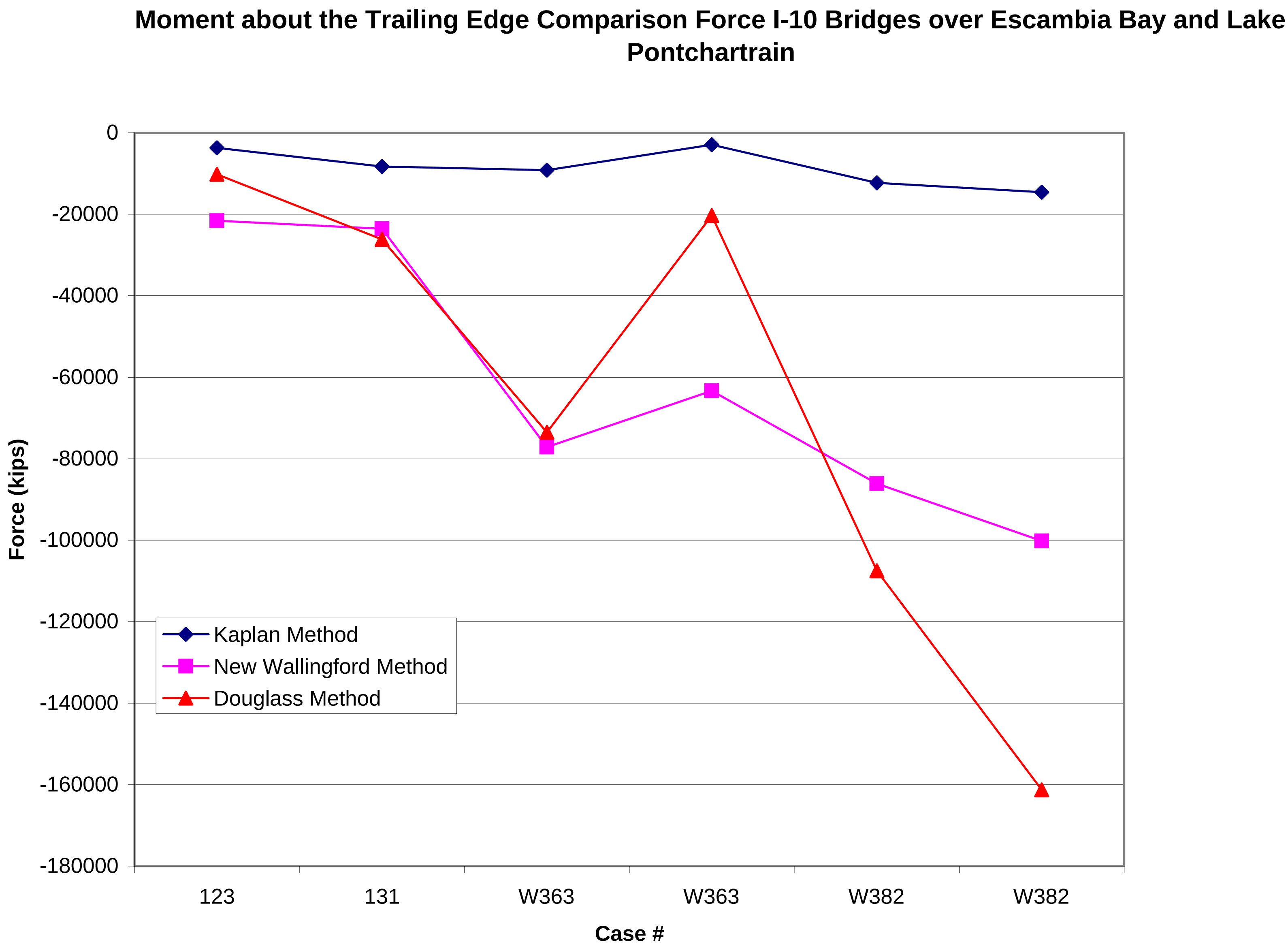


Horizontal Force at the Time of Maximum Vertical Force
Kaplan Method
New Wallingford Method
Douglass Method
Force (kips)
3000
2500
2000
1500
1000
500
0
-500
Case #

Horizontal Comparison Force I-10 Bridges over Escambia Bay and Lake Pontchartrain

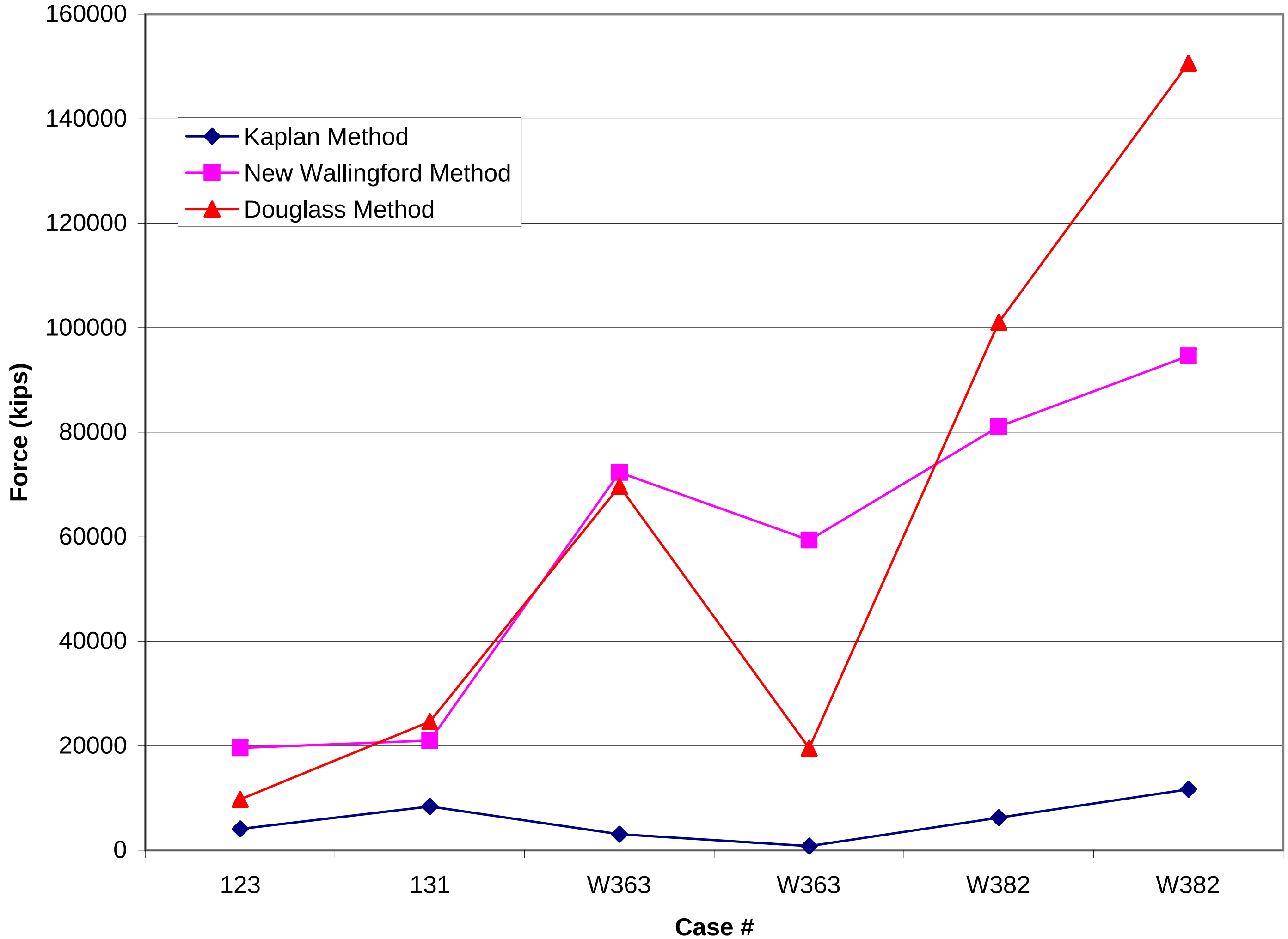



Vertical Force Comparison I-10 Bridges over Escambia Bay and Lake Pontchartrain
Kaplan Method
New Wallingford Method
Douglass Method
Force (kips)
0
1000
2000
3000
4000
5000
6000
7000
8000
123
131
W363
W363
W382
W382
Case #



Moment about the Trailing Edge Comparison Force I-10 Bridges over Escambia Bay and Lake Pontchartrain
0
-20000
-40000
-60000
-80000
-100000
-120000
-140000
-160000
-180000
Force (kips)
Kaplan Method
New Wallingford Method
Douglass Method
123
131
W363
W363
W382
W382
Case #

Moment about the Leading Edge Comparison Force I-10 Bridges over Escambia Bay and Lake Pontchartrain



Attachment C

Analysis Results
for One Span of I-10 Escambia Bay Bridge
Under Varying Storm Water Levels
and Different Wave Periods and Wave Heights

Effect of changing Y_c for 8 ft wave height

Maximum Wave Height (ft)	Y_c (ft)	OEA Maximum Vertical Force (kips)	OEA Maximum Moment about the Leading Edge (ft-kips)	OEA Maximum Moment about the Trailing Edge (ft-kips)	Wallingford Method 2 Maximum Vertical Force (kips)	Wallingford Method 2 Associated Moment about the Leading Edge (ft-kips)	Wallingford Method 2 Associated Moment about the Trailing Edge (ft-kips)	Douglas Method Maximum Vertical Force (kips)	Douglas Method Maximum Moment about the Leading Edge (ft-kips)	Douglas Method Maximum Moment about the Trailing Edge (ft-kips)
8	3	13	38	-417	1143	19739	-20567	383	6679	-6814
8	2	28	107	-898	1198	20235	-21992	551	9517	-9920
8	1	50	230	-1567	1252	20879	-23258	895	15337	-16227
8	0	77	418	-2461	1307	21814	-24268	1437	24526	-26141
8	-1	117	1232	-3709	1362	22681	-25322	1979	33574	-36197
8	-2	217	2719	-5052	1417	23628	-26320	2522	42528	-46367
8	-3	229	4414	-5776	1471	24575	-27293	3064	51468	-56529
8	-4	413	6232	-8205	1531	25598	-28357	3606	60409	-66690
8	-5	534	8168	-10180	1586	26543	-29356	4148	69351	-76851
8	-6	636	10086	-11889	1640	27480	-30340	4689	78291	-87013
8	-7	739	12197	-13342	1695	28426	-31340	5231	87232	-97174
8	-8	783	13399	-14039	1752	29394	-32363	5773	96173	-107335
8	-9	768	13187	-13668	1804	30282	-33315	6315	105113	-117498
8	-12	735	12537	-12789						
8	-15	712	12039	-12202						

Effect of changing Y_c for different wave periods and wave height of 12 ft.

Wave Period (sec)	Maximum Wave Height (ft)	Y_c (ft)	OEA Maximum Vertical Force (kips)	OEA Maximum Moment about the Leading Edge (ft-kips)	OEA Maximum Moment about the Trailing Edge (ft-kips)	Wallingford Method 2 Maximum Vertical Force (kips)	Wallingford Method 2 Associated Moment about the Leading Edge (ft-kips)	Wallingford Method 2 Associated Moment about the Trailing Edge (ft-kips)	Douglas Method Maximum Vertical Force (kips)	Douglas Method Maximum Moment about the Leading Edge (ft-kips)	Douglas Method Maximum Moment about the Trailing Edge (ft-kips)
4	12	3	77	359	-2516	1745	28944	-32584	1107	18941	-20083
6	12	3	448	3635	-4440	1700	28146	-31770	782	13393	-14162
8	12	3	773	7100	-6978	1716	28436	-32055	885	15159	-16037
4	12	0	207	3039	-6277	1988	32983	-37100	2733	46007	-50323
6	12	0	918	9203	-9929	1937	32100	-36194	2407	40639	-44222
8	12	0	1257	12321	-12797	1954	32385	-36485	2511	42344	-46159
4	12	-9	841	14048	-15246	2706	45269	-50101	7611	126488	-141787
6	12	-9	768	13509	-14086	2660	44494	-49264	7285	121119	-135687
8	12	-9	730	12834	-13195	2671	44677	-49469	7389	122824	-137624

Effect of changing Y_c for wave period 4 sec. and wave height of 12 ft.

Wave Period (sec)	Maximum Wave Height (ft)	Y_c (ft)	OEA Maximum Vertical Force T=4sec (kips)	OEA Maximum Moment about the Leading Edge T=4sec (ft-kips)	OEA Maximum Moment about the Trailing Edge T=4sec (ft-kips)	Wallingford Method 2 Maximum Vertical Force T=4sec (kips)	Wallingford Method 2 Associated Moment about the Leading Edge T=4sec (ft-kips)	Wallingford Method 2 Associated Moment about the Trailing Edge T=4sec (ft-kips)	Douglas Method Maximum Vertical Force T=4sec (kips)	Douglas Method Associated Moment about the Leading Edge T=4sec (ft-kips)	Douglas Method Associated Moment about the Trailing Edge T=4sec (ft-kips)
4	12	3	77	359	-2516	1745.4817	28944	-32584	1107	18941	-20083
4	12	0	207	3039	-6277	1988.16681	32983	-37100	2733	46007	-50323
4	12	-9	841	14048	-15246	2705.53362	45269	-50101	7611	126488	-141787

Effect of changing Y_c for wave period 6 sec. and wave height of 12 ft.

Wave Period (sec)	Maximum Wave Height (ft)	Y_c (ft)	OEA Maximum Vertical Force T=6sec (kips)	OEA Maximum Moment about the Leading Edge T=6sec (ft-kips)	OEA Maximum Moment about the Trailing Edge T=6sec (ft-kips)	Wallingford Method 2 Maximum Vertical Force T=6sec (kips)	Wallingford Method 2 Associated Moment about the Leading Edge T=6sec (ft-kips)	Wallingford Method 2 Associated Moment about the Trailing Edge T=6sec (ft-kips)	Douglas Method Maximum Vertical Force T=6sec (kips)	Douglas Method Associated Moment about the Leading Edge T=6sec (ft-kips)	Douglas Method Associated Moment about the Trailing Edge T=6sec (ft-kips)
6	12	3	448	3635	-4440	1699.744681	28146	-31770	782	13393	-14162
6	12	0	918	9203	-9929	1937.429787	32100	-36194	2407	40639	-44222
6	12	-9	768	13509	-14086	2659.796596	44494	-49264	7285	121119	-135687

Effect of changing Y_c for wave period 8 sec. and wave height of 12 ft.

Wave Period (sec)	Maximum Wave Height (ft)	Y_c (ft)	OEA Maximum Vertical Force T=8sec (kips)	OEA Maximum Moment about the Leading Edge T=8sec (ft-kips)	OEA Maximum Moment about the Trailing Edge T=8sec (ft-kips)	Wallingford Method 2 Maximum Vertical Force T=8sec (kips)	Wallingford Method 2 Associated Moment about the Leading Edge T=8sec (ft-kips)	Wallingford Method 2 Associated Moment about the Trailing Edge T=8sec (ft-kips)	Douglas Method Maximum Vertical Force T=8sec (kips)	Douglas Method Associated Moment about the Leading Edge T=8sec (ft-kips)	Douglas Method Associated Moment about the Trailing Edge T=8sec (ft-kips)
8	12	3	773	7100	-6978	1716.07234	28436	-32055	885	15159	-16037
8	12	0	1257	12321	-12797	1953.757447	32385	-36485	2511	42344	-46159
8	12	-9	730	12834	-13195	2670.812766	44677	-49469	7389	122824	-137624

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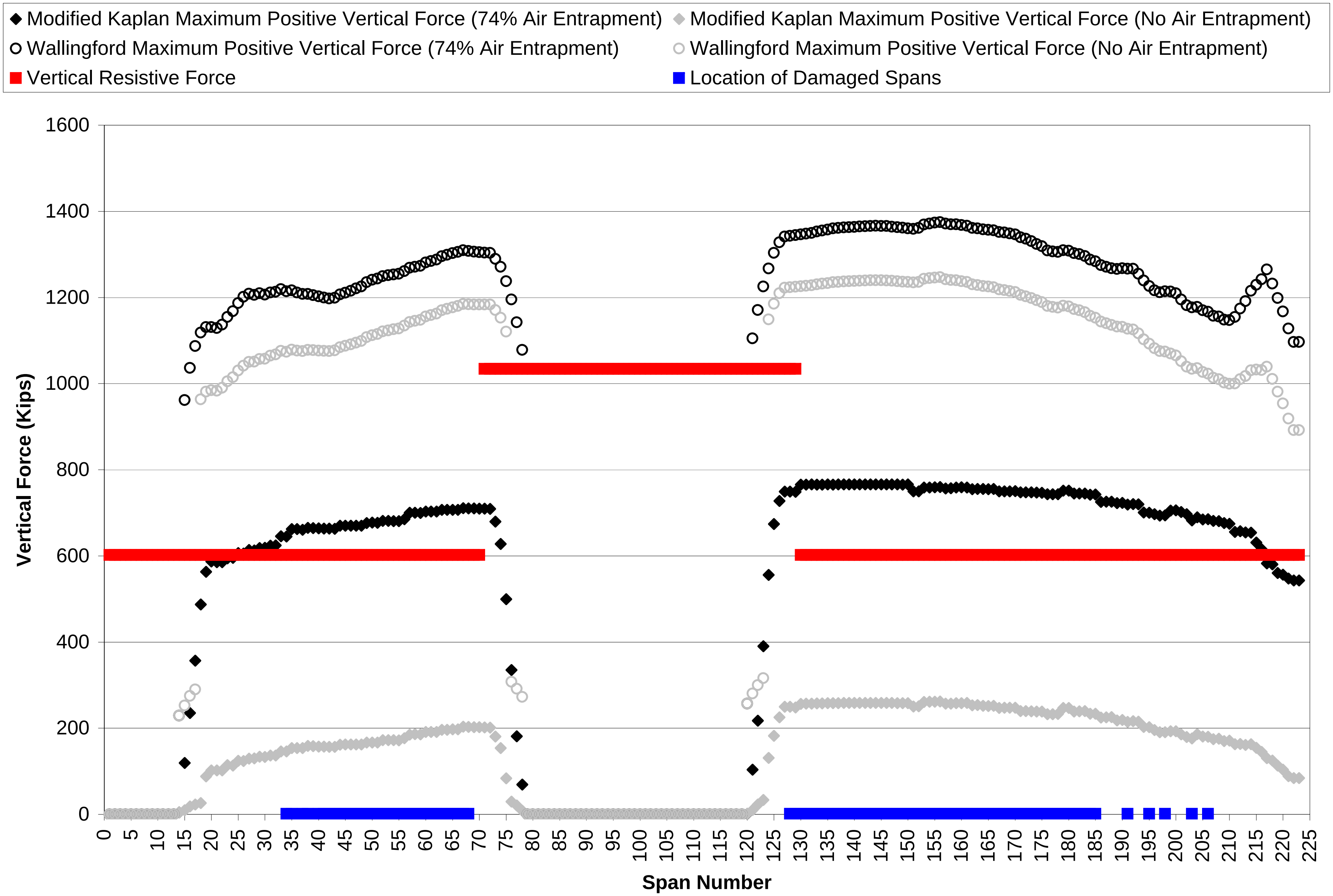
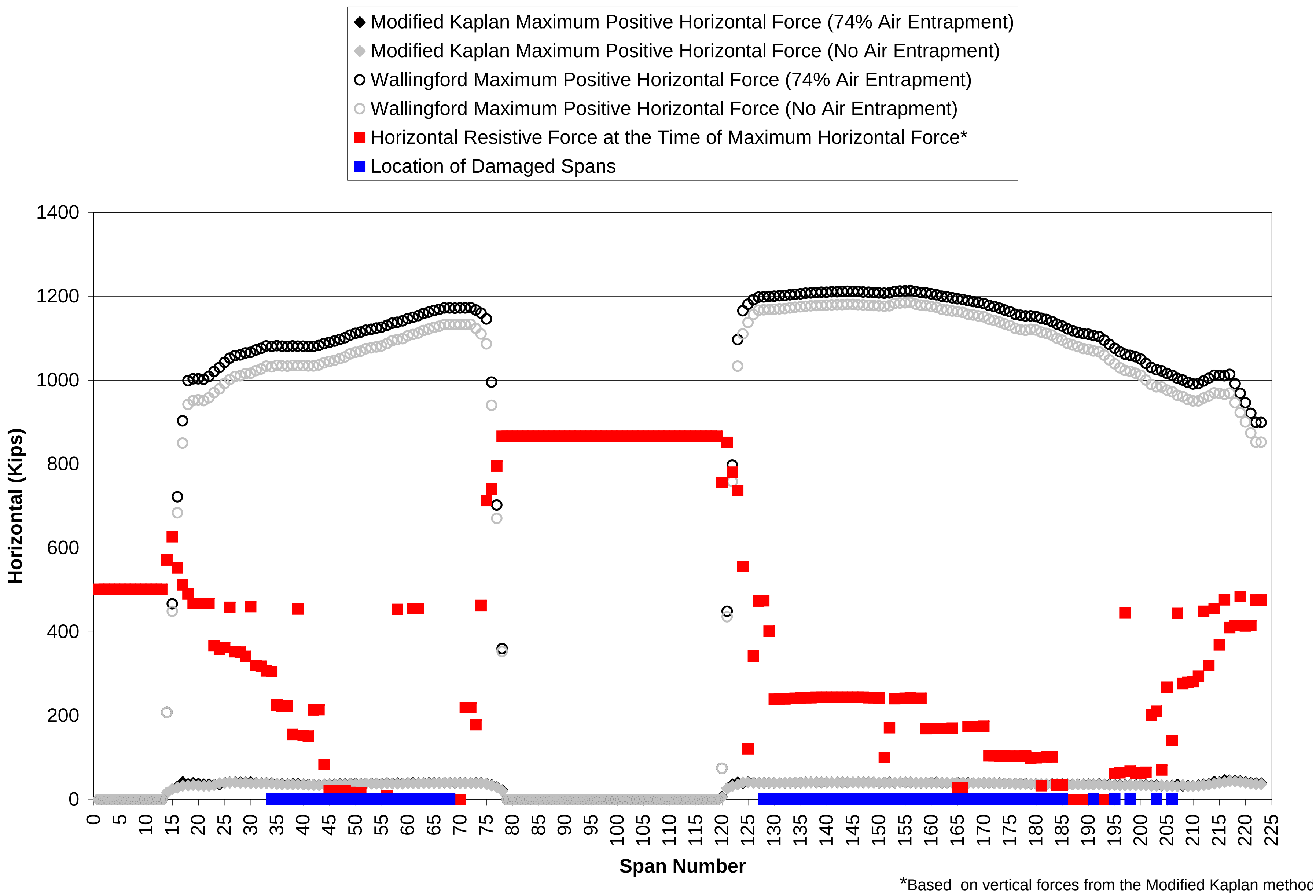
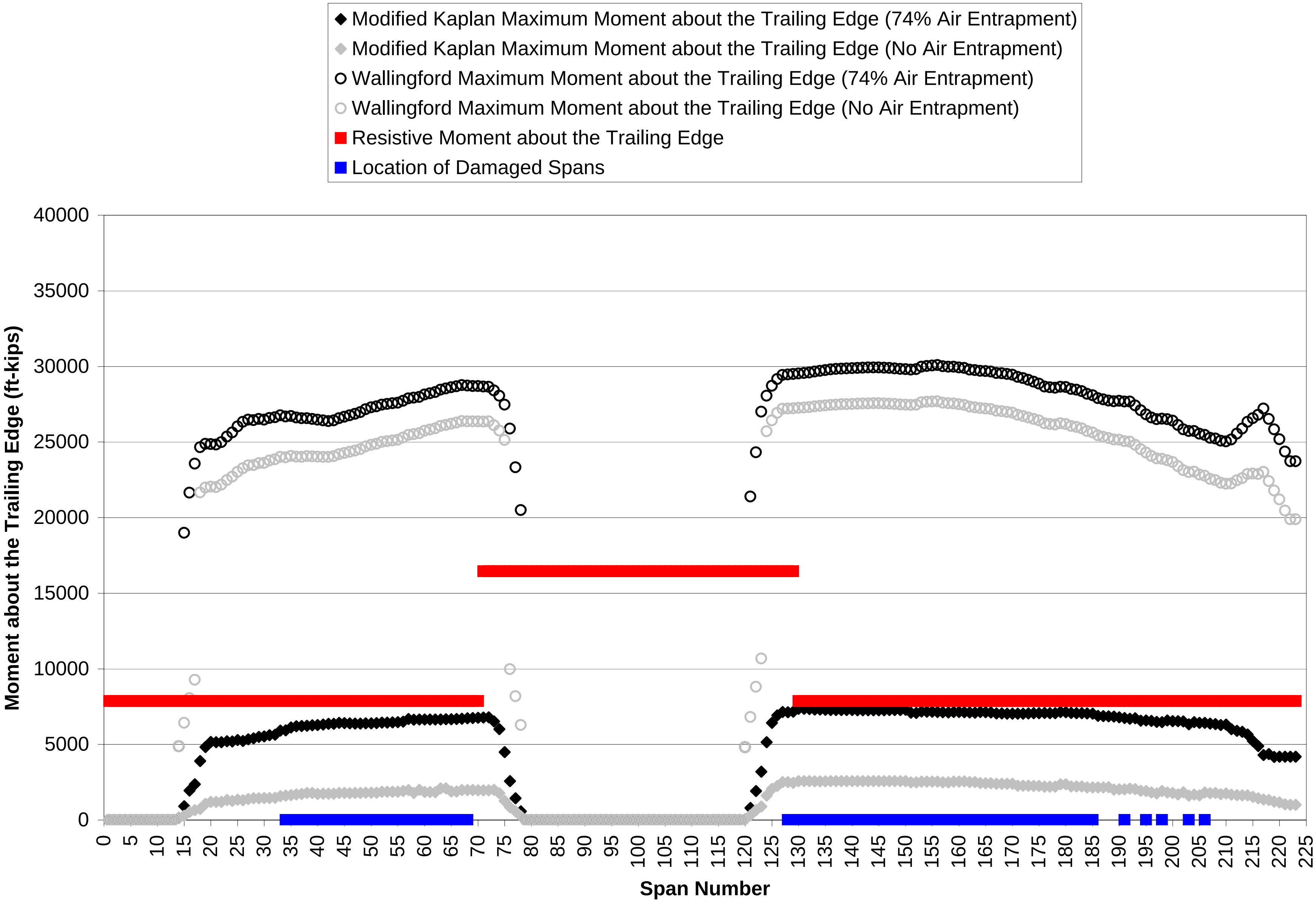
Attachment D

Wave Force Calculations for All Spans
of
I-10 Escambia Bay Bridge using different analysis methods



Modified Kaplan Maximum Positive Vertical Force (74% Air Entrapment)
Modified Kaplan Maximum Positive Vertical Force (No Air Entrapment)
Wallingford Maximum Positive Vertical Force (74% Air Entrapment)
Wallingford Maximum Positive Vertical Force (No Air Entrapment)
Vertical Resistive Force
Location of Damaged Spans
Vertical Force (Kips)
0
200
400
600
800
1000
1200
1400
1600
Span Number
0 5 10 15 20 25 30 35 40 45 50 55 60 65 70 75 80 85 90 95 100 105 110 115 120 125 130 135 140 145 150 155 160 165 170 175 180 185 190 195 200 205 210 215 220 225


Modified Kaplan Maximum Positive Horizontal Force (74% Air Entrapment)
Modified Kaplan Maximum Positive Horizontal Force (No Air Entrapment)
Wallingford Maximum Positive Horizontal Force (74% Air Entrapment)
Wallingford Maximum Positive Horizontal Force (No Air Entrapment)
Horizontal Resistive Force at the Time of Maximum Horizontal Force*
Location of Damaged Spans
Horizontal (Kips)
Span Number
*Based on vertical forces from the Modified Kaplan method


Modified Kaplan Maximum Moment about the Trailing Edge (74% Air Entrapment)
Modified Kaplan Maximum Moment about the Trailing Edge (No Air Entrapment)
Wallingford Maximum Moment about the Trailing Edge (74% Air Entrapment)
Wallingford Maximum Moment about the Trailing Edge (No Air Entrapment)
Resistive Moment about the Trailing Edge
Location of Damaged Spans
Moment about the Trailing Edge (ft-kips)
Span Number
0
5000
10000
15000
20000
25000
30000
35000
40000
0 5 10 15 20 25 30 35 40 45 50 55 60 65 70 75 80 85 90 95 100 105 110 115 120 125 130 135 140 145 150 155 160 165 170 175 180 185 190 195 200 205 210 215 220 225

Attachment E

A Method for Estimating Bridge Span Resistance
to
Storm Surge and Wave Loading
for
Girder Type Bridge Superstructures

TASK ORDER DTFH61-06-T-70006

FOR THE DEVELOPMENT OF

GUIDE SPECIFICATIONS FOR BRIDGES VULNERABLE TO COASTAL STORMS AND

HANDBOOK OF RETROFIT OPTIONS FOR BRIDGES VULNERABLE TO COASTAL STORMS

A Method for Estimating Bridge Span Resistance to Storm Surge and Wave Loading for Girder Type Bridge Superstructures

D. Max Sheppard
Phil Dompe
OEA, Inc.
February 2007

When bridge decks encounter elevated water levels and wind waves they can be subjected to complex horizontal and vertical loads. The loads are dynamic and are composed of drag, inertia, change in added mass and impulse (slamming) forces. In the vertical direction there is also a buoyancy force. As used here the reactive forces consist of the weight of the superstructure (beams, deck, railings, etc.) and the frictional forces between the beams and the pile cap.

The analyses presented here makes the assumption that the forces and moments due to constraints (e.g. anchor bolts and tie-downs) are not initiated until the reaction forces/moments (as defined above) have been exceeded. Once the reaction forces/moments have been exceeded the constraint forces/moments are initiated and increase until failure. A number of additional assumptions have also been made including:

- Static analysis of the following three cases is sufficient to capture the state of forces: 1) maximum vertical force and associated horizontal force and moment about the trailing girder, 2) maximum horizontal force and associated vertical force and moment about the trailing girder, and 3) maximum moment and associated horizontal and vertical forces.
- Superstructure assumed to be a rigid body with no deflections
- The span itself has sufficient strength to withstand the applied loads (i.e. the span does not fracture). Since bridge spans are not normally designed for upward

vertical forces this could be a problem and should be investigated by a separate analysis.

The horizontal and vertical forces are out of phase as the wave propagates past the structure. For this reason the forces/moment are computed for three different times, 1) at the time of maximum vertical force (with associated horizontal force and moment about the trailing edge (see Figures 1 and 2), 2) at the time of maximum horizontal force (with associated vertical force and moment about the origin) and 3) at the time of maximum moment about the origin (with associated horizontal and vertical forces). For the spans analyzed to date the most critical situation has been when the vertical force is a maximum. Even though this may be true for most spans and wave conditions the other cases should be checked.

Figure 1 is a definition sketch showing the location and orientation of the coordinate system, the reactive forces and the notation used in the analysis. Figure 2 is another definition sketch showing the constraint forces and moments that occur once the reaction forces and moments are exceeded.

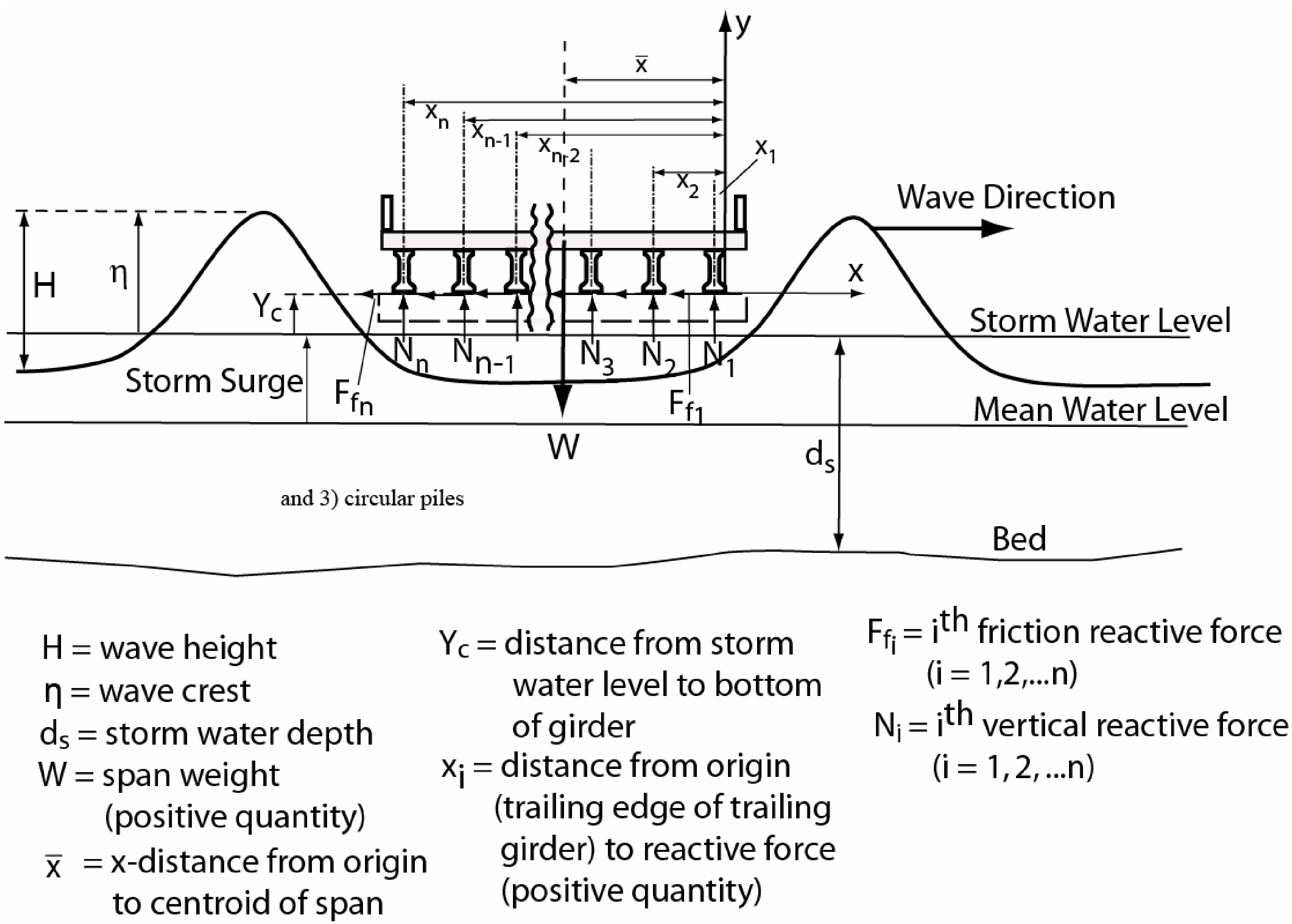


Figure 1 Definition sketch showing coordinate system, reactive forces, span weight, waves and nomenclature used in analysis.

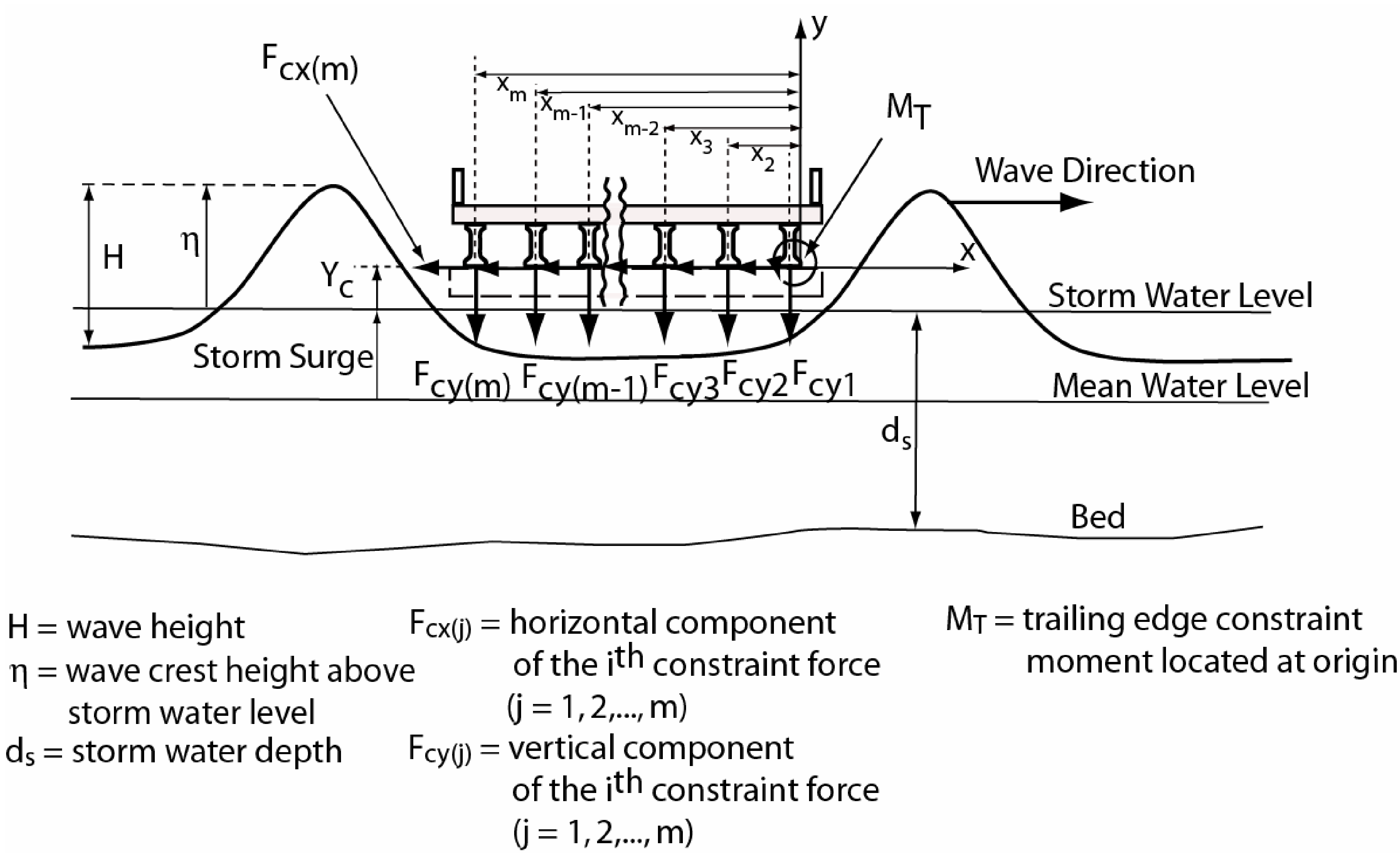


Figure 2 Definition sketch showing constraint forces and moments.

UNCONSTRAINED ANALYSIS

If the wave forces are not enough to overcome the reactive forces, the structure is in static equilibrium and the following equilibrium equations apply:

Note that the

$$\text{Total Reaction Force in the x-direction} = 2\sum_{i=1}^{n} F_{f(i)}\ . \qquad (0.1)$$

The assumption is that the friction force is the same on both ends of the girder (and therefore the 2 in Equation (0.1).

$$\sum_{i=1}^{n} F_x = F_{x(wave)} - 2\sum_{i=1}^{n} F_{f(i)} \qquad (0.2)$$

$$\sum F_y = F_{y(wave)} - W + 2\sum_{i=1}^{n} N_i = 0 \qquad (0.3)$$

$$\sum M_0 = M_{0(wave)} + W\bar{x} - 2\sum_{i=2}^{n} N_i\, x_i = 0 \qquad (0.4)$$

where

$F_{f(i)} \equiv$ frictional force on one end of the i^{th} girder

$N_i \equiv$ vertical component of the reactive force on one end of i^{th} girder

$\bar{x} \equiv$ x distance from origin to span center of gravity (positive scalar)

$x_i \equiv$ x distance from origin to center of the i^{th} girder (positive scalar)

$W \equiv$ weight of superstructure (positive scalar)

$F_{x(wave)} \equiv$ x component of wave force (positive or negative)

$F_{y(wave)} \equiv$ y component of wave force (positive or negative)

$M_{0(wave)} \equiv$ moment of wave loading about origin (positive CCW)

$n \equiv$ number of girders

The friction force between the bottom of one end of the i^{th} girder and the pile cap is given in Equation (0.5)

$$F_{f(i)} = \mu\ N_i \tag{0.5}$$

where

$$\mu \equiv \text{ static friction coefficient} \tag{0.6}$$

Substituting Equation (0.5) into Equations (0.3) and (0.4) results in

$$\text{Reaction Force in X - direction } = \begin{cases} \mu\, N & \text{for } W > F_{y(wave)} \\ 0 & \text{for } W \le F_{y(wave)} \end{cases}, \tag{0.7}$$

where

$$N = 2\sum_{i=1}^{n} N_i = W - F_{y(wave)} \tag{0.8}$$

Note that N cannot be negative.

$$\text{Reaction Force in y - direction } = \begin{cases} W - F_{y(wave)} & \text{for } W > F_{y(wave)} \\ 0 & \text{for } W \le F_{y(wave)} \end{cases} \tag{0.9}$$

$$\text{Stabilizing Moment about origin} = W\ \bar{x} \tag{0.10}$$

We can now see if any of the maximum reaction forces are exceeded.

If

$$F_{x(wave)} > \mu N = \mu \left(W - F_{y(wave)}\right) \tag{0.11}$$

the horizontal wave force exceeds the horizontal friction force.

If

$$F_{y(wave)} > W \tag{0.12}$$

the vertical wave forces exceeds the vertical reactive force.

If

$M_{0(wave)} < 0$ (i.e.$M_{0(wave)}$ is in CW direction, and;

$$\left|M_{0(wave)}\right| > W\,\bar{x}, \tag{0.13}$$

the moment about the origin created by the wave forces exceeds the stabilizing moment.

If any one of the above described reactive forces/moments are exceeded and the span has constraints (i.e. bolts, tie downs, etc.) then the following analysis must be performed.

The following equations assume that at least one of the reaction forces and/or moments (due to the weight of the structure in the vertical direction and the friction forces in the horizontal direction) have been exceeded. The constraint forces and moments in the following analysis are the best estimates of their maximum value. The total number of constraints is m and the location of the j^{th} constraint relative to the origin of the coordinate system is x_j as shown in Figure 2.

The excess forces and moment are defined as follows. These forces/moment represent the amount the wave/surge forces exceed the reactive forces. Note that some of these could be negative (i.e. one or more may not have exceeded the reactive forces/moments for the unconstrained span).

$$F_{x(excess)} = F_{x(wave)} - \mu N = F_{x(wave)} - \mu\left(W - F_{y(wave)}\right) \tag{0.14}$$

$$F_{y(excess)} = \left(F_{y(wave)} - W\right) \tag{0.15}$$

$$M_{0(excess)} = \left(W\,\bar{x} + M_{0(wave)}\right) \tag{0.16}$$

If $M_{0(excess)} \geq 0.0$, unconstrained stabilizing moment has not been exceeded

If $M_{0(excess)} < 0.0$, unconstrained stabilizing moment has been exceeded

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CONSTRAINED ANALYSIS

If

$$2\sum_{j=1}^{m} F_{cx(j)} > F_{x(excess)} \tag{0.17}$$

the horizontal constraint forces exceed the excess wave force in the x direction.

If

$$2\sum_{j=1}^{m} F_{cx(j)} \leq F_{x(excess)} \tag{0.18}$$

the horizontal constraint forces are exceeded by the excess horizontal force and the span will fail.

If

$$2\sum_{j=1}^{m} F_{cy(j)} > F_{y(excess)} \tag{0.19}$$

the vertical constraint forces exceed the excess wave force in the y direction.

If

$$2\sum_{j=1}^{m} F_{cy(j)} \leq F_{y(excess)} \tag{0.20}$$

the vertical constraint forces are exceeded by the excess vertical force and the span will fail.

If $M_{0(excess)} < 0.0$, i.e. $M_{0(excess)}$ is in CW direction, and;

$$2\sum_{j=1}^{m} F_{cy(j)}\, x_j + M_T > \left|M_{0(excess)}\right| \tag{0.21}$$

the constraint moment exceeds the excess moment about the origin.

If

$$2\sum_{j=1}^{m} F_{cy(j)}\, x_J + M_T \leq \left|M_{0(excess)}\right| \tag{0.22}$$

the constraint moment is exceeded by the excess moment and the span will fail.

Where

M_T ≡ the moment due to the constraint at the trailing edge (this term is included to account for the rare situation where the constraint is located at the origin of the coordinate system (thus a zero moment arm) but exerts a reactive moment.

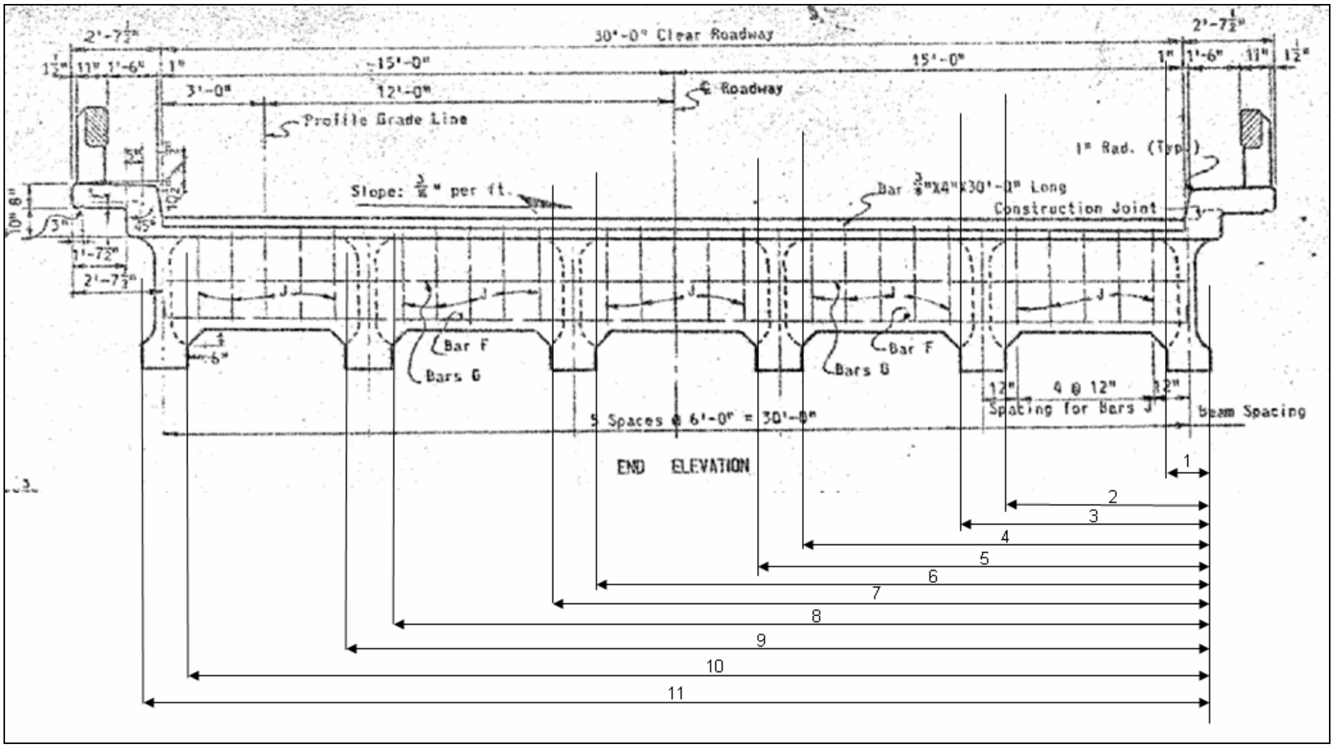
m ≡ number of constraints

$F_{cx(j)}$ ≡ j^{th} component of the horizontal constraint forces at each end of the girders

$F_{cy(j)}$ ≡ j^{th} component of the vertical constraint forces at each end of the girders

x_j ≡ horizontal distance from the origin of the j^{th} component of the vertical constraint forces

Example Problem 1: Given the Storm Surge and Wave Forces Determine if Span 131 on the East Bound I-10 Bridge over Escambia Bay Failed during Hurricane Ivan.



Moment Arms		
#	Length (ft)	
1	1.833 =	1.833
2	6-(1.833/2)+(1.833/2) =	6.000
3	6+(1.833/2)+(1.833/2) =	7.833
4	12-(1.833/2)+(1.833/2) =	12.000
5	12+(1.833/2)+(1.833/2) =	13.833
6	18-(1.833/2)+(1.833/2) =	18.000
7	18+(1.833/2)+(1.833/2) =	19.833
8	24-(1.833/2)+(1.833/2) =	24.000
9	24+(1.833/2)+(1.833/2) =	25.833
10	30-(1.833/2)+(1.833/2) =	30.000
11	30+(1.833/2)+(1.833/2) =	31.833

Step 1 – Determine wave and surge forces

Wave and surge forces computed using the Modified Kaplan Method

Maximum Positive Vertical Force (kips)	Horizontal Force (kips) at time of Maximum Vertical Force	Moment (ft-kips) at time of Maximum Vertical Force
760	40	-7207

Step 2 – UNCONSTRAINED ANALYSIS

Maximum Reactive forces in the y-direction

$$\text{Maximum Reactive Force in y - direction for unconstrained span} = W = 386 \text{ kips}$$

Maximum possible Reaction forces in the x-direction

$$\text{Maximum Possible reaction Force in x - direction for unconstrained span} = \mu N$$

$$= \mu W = 0.4 \times 386 \text{ kips} = 154 \text{ kips}$$

Maximum stabilizing moment about the origin

$$\text{Maximum Stabilizing Moment about origin for unconstrained span} = W\,\bar{x}$$

$$\text{Maximum Stabilizing Moment about origin for unconstrained span} = 386 \text{ kips} \left[\frac{30\text{ft}+1.833\text{ft}}{2}\right]$$

$$\text{Maximum Reactive Stabilizing about origin for unconstrained span} = 6{,}144 \text{ ft-kips}$$

Reactive forces with surge/wave loading in the y-direction

$$N = 2\sum_{i=1}^{n} N_i = W - F_{y(wave)}$$

$$N = 2\sum_{i=1}^{n} N_i = 386 \text{ kips} - 760 \text{ kips} = -374 \text{ kips}$$

Since the vertical reaction forces for the unconstrained case (i.e. the normal forces) must be positive or zero

$$N = 0 \text{ kips}$$

Step 3 – Check to see if surge wave forces and moments exceed reactive forces and moments

Check x-direction:

Is $F_{x(wave)} > \mu N$?

40 kips > 0.4 x 0

40 kips > 0 kips

Therefore, horizontal wave force exceeds horizontal friction force for unconstrained span.

Check y-direction

Is $F_{y(wave)} > W$?

760 > 386

Therefore, vertical wave force exceeds vertical reactive force for unconstrained span

Check moment about the origin

Is $\begin{array}{l} M_{0(wave)} < 0 \text{ and} \\ |M_{0(wave)}| > W\bar{x} \end{array}$?

-7207 ft-kips < 0

|-7207| > 6144

Therefore, moment due to wave forces exceeds moment due to reactive forces for unconstrained span.

Step 4 – Calculate excess force

Excess force in x-direction

$$F_{x(excess)} = F_{x(wave)} - \mu N = F_{x(wave)} - \mu\left(W - F_{y(wave)}\right)$$

$$F_{x(excess)} = 40 \text{ kips} - 0 \text{ kips} = 40 \text{ kips}$$

Excess force in y-direction

$$F_{y(excess)} = \left(F_{y(wave)} - W\right)$$

$$F_{y(excess)} = \left(760 - 386\right) = 374 \text{ kips}$$

Excess moment about the origin

$M_{0(wave)} < 0$ and

$M_{0(excess)} = \left(W\bar{x} + M_{0(wave)}\right)$

-7207 ft-kips < 0

$M_{0(excess)} = \left(6144 \text{ ft-kips} - 7207 \text{ ft-kips}\right) = -1063 \text{ ft-kips}$

Step 5 – CONSTRAINED ANALYSIS

Eight 7/8 in diameter anchor bolts connect the span to the substructure (two bolts at the ends of the outer girders). The plans specify A307 bolts which have a shear strength of 24 ksi and a tensile strength of 45 ksi. The bolt section is 0.6013 in^2 resulting in bolt shear and axial resistances of 14.4 kips and 27 kips, respectively. Calculating the bolt resistance using design specifications other than those used in designing the bridge and/or making different assumptions regarding the presence of threads in the shear planes or assuming some corrosion will result in different values of the calculated resistance.

Check x-direction constraints

$$\sum_{j=1}^{m} F_{cx(j)} = 8 \text{ bolts x } 14.4 \text{ kips/bolt} = 115 \text{ kips} > F_{x(excess)}$$

Therefore, constrained span does not fail due to wave forces in the x-direction.

Check y-direction constraints

$$\sum_{j=1}^{m} F_{cy(j)} = 8 \text{ bolts x } 27 \text{ kips/bolt} = 217 \text{ kips} < F_{y(excess)} \ \left(\text{i.e. } 217 \text{ kips} < 374 \text{ kips}\right)$$

Therefore, the constrained span **DOES FAIL** due to surge/wave forces in y direction.

Check the constraining moment about the origin

$$\sum_{j=1}^{m} F_{cy(j)}\, x_j + M_T = 2 \text{ x } (1.833\text{ft x } 27 \text{ kips} + 30\text{ft x } 27 \text{ kips} + 31.833\text{ft x } 27 \text{ kips}) + 0$$

$$= 3{,}438 \text{ ft-kips} > \left|M_{0(excess)}\right|$$

Therefore, the constrained span does not fail due to moments due to surge/wave forces.

According to these calculations the span will fail due to the upward vertical surge and wave force. An additional 157 kips of vertical constraint is required to equal the vertical forces.

Attachment F

Task 4 Submission

Attachment G

Discussion of Coefficients of Variation (COV's) to use in Reliability Analysis

TASK ORDER DTFH61-06-T-70006

FOR THE DEVELOPMENT OF
GUIDE SPECIFICATIONS FOR BRIDGES VULNERABLE TO COASTAL STORMS
AND
HANDBOOK OF RETROFIT OPTIONS FOR BRIDGES VULNERABLE TO COASTAL STORMS

Discussion of Coefficients of Variation (COV's) to use in Reliability Analysis

Paraphrasing from "Marketing Uncertainty" by J. William Kamphuis;

Uncertainties are inherent in basic coastal data and in derived data. Physical and numerical modeling contain their own uncertainties. Interpretation to produce quantitative output relies on measurements of output variables (calibration and verification data) and interpretation of model results requires coastal engineering expertise and must be done very carefully.

Coastal variables and the resulting computations contain substantial uncertainties. In reality, engineering is working with such uncertain parameters to derive best possible solutions to practical problems by careful study, application of basic principles, use of the appropriate tools and eventually introduction of a large measure of ingenuity and experience.

Regarding the development of guide specifications for bridges subject to coastal storm events, there are four basic parameters in the wave force prediction equations which all have varying degrees of uncertainty. They are:

1) Coefficients from laboratory tests
2) Wave height (including crest elevation)
3) Wave period
4) Storm water level (composed of astronomical tide, storm surge created by reduced atmospheric pressure and wind stress on water surface, wave setup, and local wind set-up/set-down)

For most physical quantities, errors increase with the magnitude of the quantity. For example, the absolute error in measuring a wave height of 0.5m will be less than the absolute error in measuring a wave height of 5m. Thus, the uncertainty is defined as a coefficient of variation (COV) which is equal to the standard deviation divided by the mean value. Thus, COV's for each of the parameters listed above must be determined for use in the reliability analysis. The primary reference used for this effort is the Report of PIANC Working Group No. 12, "Uncertainty Related to Environmental Data and Estimated Extreme Events." It was decided that COVs should be developed independently for both Level I and Level III degrees of analysis and load factors developed through calibration using Monte Carlo Simulation for both cases. Based on these results, an estimate of the load factors for a Level II analysis would be made.

A discussion of how each of the COV's was developed follows.

1) <u>Coefficients from Laboratory Tests</u> – These coefficients are based on curve fitting the design equations to the results of the physical laboratory tests. As such, COV's can be determined directly from the data and resulting equations. They were calculated to be (TBD) for coefficient "A" and (TBD) for coefficient "B"

2) <u>Wave Height</u> - With respect to the determination of the design wave height and crest elevation, four primary areas of uncertainties apply:
   - Errors in the calculation methods to determine the wave height - these uncertainties pertain to the validity of any analytical or numerical methods used to calculate wave heights as well as the reliability of the underlying data used in these methods. The PIANC report presents a range of COV's for nearshore wave heights of 0.25 – 0.35 for manual calculations and of 0.1-0.2 for numerical methods (noting, thought, that it could be much larger in some cases). The upper limits of these ranges were chosen; 0.35 for a Level I analysis which is based on manual analytical calculations and 0.2 for a Level III analysis which requires the use of advanced numerical models.
   - Extreme Value Analysis to determine the 100-year design event – two issues are involved in this item. The first is the error induced by the lack of knowledge about the true extremal distribution and the second concerns the influence of choice of threshold level in a Peak over Threshold, POT, analysis which is typically used for coastal storm events as these are discrete events which could occur any number of times in a given year (or not at all), and thus an annual maximum approach isn't valid. For the former issue, the PIANC report states the COV is in the order of 0.05-0.1 for events of 50 to 100 year return periods. For the latter issue, COV's on the order of 0.18-0.22 were calculated. Given that the extremal value analysis methods will be similar for Level I and Level III analyses, a combined COV value of 0.20 was chosen for uncertainties related to Extreme Value Analyses.
   - Distribution of wave heights within the spectrum, i.e. the magnitude of H_{max} versus H_s. Wave heights are typically assumed to follow a Rayleigh distribution. However, in reality, the wave heights observed in the sea tend to indicate a distribution slightly narrower than the Rayleighan due to the spread of a wave spectra over a wide frequency range, contrary to the assumption of the narrow-banded spectrum in the derivation of the Rayleigh distribution (Goda, 2000). A COV of 0.03 for both levels of analysis was chosen.
   - Calculation of η versus H, i.e., the wave crest elevation above the still water elevation – the shape of a wave is impacted by the water depth and wave period and this in turn affects the relative elevation of the wave crest above the still water elevation. Different theories have been developed to describe a wave profile and its particle movements depending upon such parameters as the wave height, water depth and wave period. A COV of 0.03 for both levels of analysis was chosen.

Based on the above values, total COV's for wave height / crest elevation of 0.4 and 0.3 were calculated for Level I and III analyses respectively. The following table summarizes these preliminary COVs for the wave height for review and concurrence by the Task Force committee members.

	Level I	Level III
H_s calculation	0.35	0.20
Extreme Value Analysis	0.20	0.20
H_{max}/H_s factor	0.03	0.03
η / H factor	0.03	0.03
Total ($\sqrt{}$sum of squares)	0.4	0.3

3) <u>Wave Period</u> - With respect to the determination of the design wave period, two primary areas of uncertainties apply:
    - Errors in the calculation methods to determine the wave height - these uncertainties pertain to the validity of any analytical or numerical methods used to calculate wave periods as well as the reliability of the underlying data used in these methods. The same COV's as chosen for wave heights were chosen for wave periods.
    - Extreme Value Analysis to determine the 100-year design event – similar issues as discussed for wave height above. Given that the extremal value analysis methods will be similar for Level I and Level III analyses, a combined COV value of 0.20 was chosen for uncertainties related to Extreme Value Analyses

The following table of preliminary COVs was developed for review by the Task Force committee members.

	Level I	Level III
Tp calculation	0.35	0.20
Extreme Value Analysis	0.20	0.20
Total ($\sqrt{}$sum of squares)	0.4	0.3

4) <u>Water Level</u> - With respect to the determination of the design water level three primary areas of uncertainties apply:
    - Errors in the calculation methods to determine the storm surge - these uncertainties pertain to the validity of any analytical or numerical methods used to calculate water levels as well as the reliability of the underlying data used in these methods. The PIANC report presents a range of COV's for estimates of storm surge of 0.1 – 0.25 for numerical methods. The upper limit of this range was chosen, 0.25 for a Level I analysis and the lower limit, 0.1 for a Level III analysis.
    - Astronomical Tides – The PIANC report provides estimates of COV's ranging from 0.001 to 0.07 for predictions from constants. A mid-range value of 0.03 was chosen for both levels of analysis.
    - Extreme Value Analysis to determine the 100-year design event – similar issues as discussed above for wave heights. Given that the extremal value analysis methods will be similar for Level I and Level III analyses, a combined COV value of 0.20 was chosen for uncertainties related to Extreme Value Analyses.
    - Errors in the calculation methods to determine the local wind set-up / set-down - these uncertainties pertain to the validity of any analytical or numerical methods used to calculate local wind set-up / set-down as well as the reliability of the underlying data used in these methods. A COV of 0.35 was chosen for a Level I analysis and 0.2 for a Level III analysis.

The following table of preliminary COVs was developed for review by the Task Force committee members.

	Level I	Level III
Surge calculation	0.25	0.10
Astronomical Tide	0.03	0.03
Extreme Value Analysis	0.20	0.20
Local Wind Set-up / -down	0.35	0.20
Total (√sum of squares)	0.5	0.3