To my fiancée, Heather
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<tr>
<td>$a_B$</td>
<td>barge bow crush depth</td>
</tr>
<tr>
<td>$a_{Bf}$</td>
<td>barge bow crush depth after impact</td>
</tr>
<tr>
<td>$a_{Bi}$</td>
<td>barge bow crush depth before impact</td>
</tr>
<tr>
<td>$a_{Bm}$</td>
<td>maximum barge bow crush depth</td>
</tr>
<tr>
<td>$a_{BY}$</td>
<td>barge bow crush depth at yield</td>
</tr>
<tr>
<td>$B_B$</td>
<td>barge width</td>
</tr>
<tr>
<td>$c_{BP}$</td>
<td>barge-pier pseudo-damping coefficient</td>
</tr>
<tr>
<td>$C_H$</td>
<td>hydrodynamic mass coefficient</td>
</tr>
<tr>
<td>DMF</td>
<td>dynamic magnification factor</td>
</tr>
<tr>
<td>DMF$_{ideal}$</td>
<td>ideal dynamic magnification factor</td>
</tr>
<tr>
<td>$f'_c$</td>
<td>compressive strength of concrete</td>
</tr>
<tr>
<td>$f''_c$</td>
<td>effective compressive strength of concrete for FB-MultiPier</td>
</tr>
<tr>
<td>$f_r$</td>
<td>rupture strength of concrete in tension</td>
</tr>
<tr>
<td>$h_P$</td>
<td>clear height of pier columns</td>
</tr>
<tr>
<td>IRF</td>
<td>inertial resistance factor</td>
</tr>
<tr>
<td>IRF$_b$</td>
<td>inertial resistance factor calibrated to total bearing shear</td>
</tr>
<tr>
<td>IRF$_{ideal}^b$</td>
<td>ideal inertial resistance factor calibrated to total bearing shear</td>
</tr>
<tr>
<td>IRF$_{ideal}$</td>
<td>ideal inertial resistance factor (general)</td>
</tr>
<tr>
<td>IRF$_m$</td>
<td>inertial resistance factor calibrated to pier moment</td>
</tr>
<tr>
<td>IRF$_{ideal}^m$</td>
<td>ideal inertial resistance factor calibrated to pier moment</td>
</tr>
<tr>
<td>IRF$_v$</td>
<td>inertial resistance factor calibrated to pier shear</td>
</tr>
<tr>
<td>IRF$_{ideal}^v$</td>
<td>ideal inertial resistance factor calibrated to pier shear</td>
</tr>
<tr>
<td>Symbol</td>
<td>Description</td>
</tr>
<tr>
<td>--------</td>
<td>-------------</td>
</tr>
<tr>
<td>$k_B$</td>
<td>linear barge bow stiffness</td>
</tr>
<tr>
<td>$k_P$</td>
<td>linear pier-soil system stiffness</td>
</tr>
<tr>
<td>$k_S$</td>
<td>series stiffness between barge bow and pier-soil system</td>
</tr>
<tr>
<td>$k_{sup}$</td>
<td>superstructure stiffness</td>
</tr>
<tr>
<td>$KE$</td>
<td>kinetic energy of barge flotilla</td>
</tr>
<tr>
<td>$L_s$</td>
<td>superstructure span length</td>
</tr>
<tr>
<td>$m_B$</td>
<td>mass of barge flotilla</td>
</tr>
<tr>
<td>$M_{\text{max}}^\text{dyn}$</td>
<td>maximum dynamic pier moment</td>
</tr>
<tr>
<td>$M_{\text{max}}^\text{static}$</td>
<td>maximum static pier moment</td>
</tr>
<tr>
<td>$P_B$</td>
<td>barge impact force</td>
</tr>
<tr>
<td>$\bar{P}_B$</td>
<td>static barge impact force</td>
</tr>
<tr>
<td>$P_B^{\text{max}}$</td>
<td>maximum dynamic barge impact force</td>
</tr>
<tr>
<td>$P_{\text{ideal}}$</td>
<td>ideal amplified barge impact force</td>
</tr>
<tr>
<td>$P_{BY}$</td>
<td>barge impact force at yield</td>
</tr>
<tr>
<td>$P_i$</td>
<td>generalized bridge parameter for $i = 1\ldots6$</td>
</tr>
<tr>
<td>$P_1$</td>
<td>inertial force at top of pier</td>
</tr>
<tr>
<td>$r$</td>
<td>Pearson correlation coefficient</td>
</tr>
<tr>
<td>$R_B$</td>
<td>barge width modification factor</td>
</tr>
<tr>
<td>$v_{\text{Bi}}$</td>
<td>barge velocity before impact</td>
</tr>
<tr>
<td>$v_{\text{Br}}$</td>
<td>barge velocity after impact</td>
</tr>
<tr>
<td>$V$</td>
<td>barge impact speed</td>
</tr>
<tr>
<td>$W$</td>
<td>weight of barge flotilla</td>
</tr>
<tr>
<td>$W_P$</td>
<td>weight of pier, including footing (pile cap), shear wall(s), shear strut(s), pier column(s), and pier cap beam</td>
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</table>
$W_{\text{sup}}$ weight of superstructure

$\alpha_i$ parameter exponent for $i = 1 \ldots 6$

$\Delta DE_{\text{if}}$ change in deformation energy from initial state to final state

$\Delta DE_{\text{im}}$ change in deformation energy from initial state to maximum deformation

$\Delta KE_{\text{if}}$ change in kinetic energy from initial state to final state

$\Delta KE_{\text{im}}$ change in kinetic energy from initial state to maximum deformation

$\gamma$ product of bridge parameters (for IRF correlations)

$\theta$ oblique barge impact angle
Current practice with regard to designing bridge structures to resist impact loads associated with barge collisions relies upon the use of the AASHTO bridge design specifications. The AASHTO barge impact design provisions, which were developed from pendulum impact-hammer testing of reduced-scale barge models, employ a static analysis approach. However, recent studies have revealed that significant dynamic amplifications of structural demands (pier design forces) are produced as the result of mass-related inertial forces associated with the bridge superstructure. These same studies have also demonstrated that currently employed static analysis procedures fail to capture or account for such amplification effects.

In the present study, an equivalent-static analysis procedure is developed for use in barge impact design and assessment of bridge structures. In contrast to the AASHTO static analysis procedure, the new method proposed here, called the “static bracketed impact analysis” (SBIA) method, employs static loading conditions and static structural analyses, but produces design forces that conservatively approximate dynamic amplification effects associated with superstructure mass. The SBIA method produces design forces that are equivalent to—or greater than—those that would be predicted using more refined dynamic time-domain methods such as the previously developed “coupled vessel impact analysis” (CVIA) method. Due to its simplicity,
SBIA is particularly appropriate for situations involving preliminary design of bridges or the
design of relatively regular bridge structures for which time-domain dynamic analysis is not
warranted.

In this thesis, a detailed discussion of mass-related dynamic amplifications in bridges
subjected to barge impact loading is presented. Based on insights gained through characterization
of dynamic amplification modes, the static bracketed impact analysis (SBIA) method is
developed and described in detail. A parametric study is then conducted using the SBIA method
to demonstrate its ability to conservatively approximate dynamically amplified bridge design
forces.
CHAPTER 1
INTRODUCTION

Introduction

Design provisions such as the AASHTO *LRFD* Bridge Design Specifications and Commentary (2008) prescribe loading conditions that bridge structures must be adequately designed to resist. For bridges spanning over waterways that are navigable by barge traffic, design and vulnerability assessment calculations must consider the combined effects of lateral barge impact loading and vertical gravity loading. Barge impact loading occurs when a moving barge flotilla (possessing initial kinetic energy) strikes a stationary bridge component (frequently a pier) and is rapidly redirected or brought to a stop. Given that kinetic energy affects the magnitudes of loads generated, barge collision events are fundamentally dynamic in nature. Dynamic sources of structural loading such as barge collision and earthquake loading are frequently assessed through the use equivalent static loading conditions and static structural analysis. In the case of barge collision loading, the AASHTO bridge design provisions permit designers to use a simplified static analysis procedure to assess structural response in lieu of more complex fully dynamic methods.

As detailed in past research reports (Consolazio et al. 2006, Consolazio et al. 2008), the vessel collision components of the AASHTO bridge design provisions include a static barge impact load prediction procedure that is based on tests conducted by Meier-Dörnberg (1983). In the Meier-Dörnberg study, both static and dynamic-drop-hammer tests were performed on reduced scale models of barge bows to quantify impact loads. A key finding of the study was that no significant differences were observed between static and dynamic load tests. However, because the test procedures failed to include a moving barge striking a deformable bridge
structure, dynamic amplification effects related to characteristics of the impacted bridge were
omitted from the study.

To overcome the limitations of the Meier-Dörnberg study (i.e., reduced scale and omission
of pier characteristics) a full-scale barge impact test program (Consolazio et al. 2006) was
carried out on piers of the old St. George Island Causeway Bridge. The St. George Island test
program involved impacting a full-scale tanker barge into three different bridge pier
configurations, each having different structural characteristics. Over a series of eighteen (18)
tests, conducted at varying impact speeds, direct measurements of impact loads and
corresponding bridge responses were made. Subsequently, detailed dynamic structural analyses
were conducted on models the bridge structure using the same impact conditions as those that
were generated experimentally. The experimental test results and dynamic structural analysis
results revealed that significant dynamic amplifications of bridge pier design forces were
produced by mass-related inertial restraint from the bridge superstructure. The study also
demonstrated that currently employed static analysis procedures do not capture amplification
effects caused by the weight (mass) of the bridge superstructure.

In response to the discovery of superstructure-induced dynamic amplification effects, a
follow-up study was conducted (Consolazio et al. 2008) to develop dynamic barge-bridge
collision analysis procedures capable of accounting for such dynamic phenomena. A key result
of this work was the development of a dynamic time-domain analysis procedure called coupled
vessel impact analysis (CVIA) which numerically couples models of a barge, bridge pier,
foundation, soil, and superstructure. Dynamic amplification due to inertial superstructure
restraint is directly accounted for in this method through the inclusion of both superstructure
(bridge deck) mass and stiffness. Used in conjunction with newly formulated barge crush curves
(Consolazio et al. 2009a) and a simplified bridge modeling procedure (Consolazio and Davidson 2008, Davidson et al. 2010), coupled vessel impact analysis permits bridge design forces to be quantified using a rational and accurate dynamic structural analysis procedure.

However, while the CVIA method balances accuracy with numerical efficiency, it is still a time-history analysis procedure. As such, analysis results such as impact loads, bridge displacements, and member forces are all functions of time that must be post-processed in order to identify maximum design values of interest. If a transient dynamic assessment of structural adequacy is needed, CVIA is an ideal solution. However, in many cases, a simpler, yet conservative, analysis method involving a small number of discrete load cases (as opposed to hundreds or thousands of time steps) is more desirable.

**Objectives**

The objectives of this study center on the development of an equivalent static analysis procedure for barge impact design and assessment of bridge structures. In contrast to the AASHTO static barge impact analysis procedure, the equivalent static method developed here employs static loading conditions and static structural analyses, but produces structural demands (design forces) that conservatively approximate dynamic amplification effects associated with superstructure mass. The new method is intended to produce design forces that are equivalent to—or greater than—those that would be predicted using more refined dynamic methods such as CVIA. Key objectives of this study are to ensure that the newly developed equivalent static analysis method is both simple to use, conservative, and capable of accounting for dynamic amplification. Such a method will be particularly appropriate for situations involving preliminary design of bridges (during which few structural details are available) or the design of relatively regular (non-lifeline) bridge structures for which the additional effort involved in conducting a time-history analysis is not warranted. An ideal bridge design process might involve the use of
equivalent static analysis for preliminary design, followed by more refined time-history analysis (e.g., CVIA) to maximize safety and minimize costs.

**Scope of Work**

- Development of bridge models:
  Finite element bridge models are developed for a representative set of bridges sampled from throughout the state of Florida. Each bridge model incorporates all necessary information that is needed to permit nonlinear static and nonlinear dynamic analyses to be performed. Nonlinearities are incorporated into the pier components, piles, soil, and the barge.

- Development of an equivalent static analysis method.
  Using insights gained through characterization of dynamic amplification effects, an equivalent static analysis procedure called static bracketed impact analysis (SBIA) is developed. The SBIA method utilizes two static analysis load cases to bracket (envelope) pier element design forces in such a manner that dynamic amplification effects are conservatively approximated.

- Demonstration parametric study:
  A comprehensive parametric study is conducted using the SBIA method and two types of superstructure modeling. Parametric study results demonstrate the level of conservatism of the simplified static method relative to more refined dynamic CVIA analyses.
CHAPTER 2
BACKGROUND

Review of the Current AASHTO Load Determination Procedure

For bridges that span over navigable waterways, the design specifications used in the United States include the American Association of State Highway and Transportation Officials (AASHTO) Guide Specification and Commentary for Vessel Collision Design of Highway Bridges (AASHTO 2009) and the AASHTO Load and Resistance Factor Design (LRFD) Bridge Design Specifications and Commentary (AASHTO 2008). These documents, collectively referred to as the “AASHTO provisions” in this thesis, use an empirical load calculation procedure based upon an experimental study conducted by Meier-Dörnberg (1983).

Meier-Dörnberg conducted both static and dynamic impact tests on reduced-scale European Type IIa barge bow sections. The European type IIa barge is similar in size and configuration to the jumbo hopper barges widely used throughout the United States. Two dynamic tests, using 2-ton pendulum hammers and two different shapes of impact head, were conducted on 1:4.5-scale stationary (i.e., fixed) barge bows. One dynamic test involved three progressive impacts using a cylindrical hammer with a diameter of 67.0 in., whereas the other involved three progressive impacts using a 90° pointed hammer. A static test was also conducted on a 1:6 scale barge bow using a 90.6 in. hammer. Results obtained from the dynamics tests are shown in Figures 2-1a-b and results from the static test are shown in Figure 2-1c.

Using the experimental data collected, Meier-Dörnberg developed mathematical relationships between kinetic energy (KE), inelastic barge deformation ($a_B$), and dynamic and static force ($P_B$ and $\bar{P}_B$ respectively). These relationships are illustrated in Figure 2-2. As the figure suggests, no major differences were found between the magnitude of dynamic and static impact force. However, this observation was strongly influenced by the stationary barge bow
configuration used in the testing. Omission of a flexible impact target and the corresponding 
barge-pier interaction necessarily precludes the ability to measure and capture dynamic 
amplification effects.

Inelastic barge bow deformations measured by Meier-Dörnberg showed that once barge 
bow yielding was initiated, at approximately 4 in. of deformation ($a_B$), the stiffness of the bow 
diminishes significantly (see Figure 2-2). Additionally, Meier-Dörnberg recognized that inelastic 
bow deformations represent a significant form of energy dissipation during collision events.

In the development of the AASHTO barge impact design provisions, the relationships 
between initial barge kinetic energy ($KE$), barge deformation ($a_B$) and equivalent static force ($P_B$) 
developed by Meier-Dörnberg, were adopted with minimal modifications:

$$KE = \frac{C_H W V^2}{29.2} \quad (2-1)$$

$$a_B = \left[ \left(1 + \frac{KE}{5672}\right)^{1/2} - 1 \right] \frac{10.2}{R_B} \quad (2-2)$$

$$P_B = \begin{cases} 
4112a_B R_B & \text{if } a_B < 0.34 \\
(1349 + 110a_B) R_B & \text{if } a_B \geq 0.34
\end{cases} \quad (2-3)$$

In Eqns. 2-1 - 2-2 $KE$ is the barge kinetic energy (kip-ft), $C_H$ is the hydrodynamic mass 
coefficient, $W$ is the vessel weight (in tonnes where 1 tonne = 2205 lbs.), $V$ is the impact speed 
(ft/sec), and $R_B = B_B/35$ where $B_B$ is the width of the barge (ft). The only notable difference 
between the expressions developed by Meier-Dörnberg and the AASHTO expressions, given 
above as Eqn. 2-1 - 2-3, is the use of a barge width correction factor ($R_B$). While the AASHTO 
specification utilizes the $R_B$ term to reflect the influence of barge width, no such factor has been 
included to account for variations in either the size (width) or geometric shape of the bridge pier 
being impacted.
Note that Eqn. 2-3 is a barge force-deformation relationship (i.e., a “crush curve”) that relates static barge impact force $P_B$ to barge deformation $a_B$. The AASHTO barge crush model (Eqn. 2-3) is illustrated graphically in Figure 2-3 for a hopper barge having a width $B_B = 35$ ft and therefore an $R_B = 1$.

**Updated Barge Bow Force-Deformation Relationships**

For dynamic structural analysis purposes, barge response during impact events may be characterized by appropriate force-deformation relationships (crush curves) that describe nonlinear stiffness of the affected vessel portions. As discussed above, development of the AASHTO crush curve (shown in Figure 2-3) relied upon scale model crushing tests. These experiments, however, were carried out using reduced-scale barge models of European, pontoon-style construction, not typical of vessels navigating waterways in the United States.

To address these limitations, studies were conducted (Consolazio et al. 2008; Consolazio et al. 2009a) to characterize barge bow crushing behavior. High-resolution finite element models of the bow sections of two common U.S. barge types—a jumbo hopper barge and an oversize tanker barge—were developed. Over 120,000 elements were used to model each barge bow. During analysis, the barge bow models were subjected to crushing by a wide variety of impactor shapes and sizes. Specifically, both round and flat-faced impact surfaces were employed in the simulations, with impactor sizes ranging from 1 ft to 35 ft.

Force-deformation relationships obtained from a multitude of simulations (a typical case is illustrated in Figure 2-4) were used to form an updated set of design force-deformation relationships for barge bows. The study yielded the following findings:

- For typical design scenarios (head-on impact conditions), barge bow force-deformation can be idealized as an elastic, perfectly-plastic relationship. Recall from Figure 2-3 that the AASHTO crush force continues to increase with increasing deformation. The simulations conducted by Consolazio et al. (2008; 2009a) did not exhibit post-yield hardening
behavior. The AASHTO curve and a typical Consolazio et al. curve are compared in Figure 2-5.

- Due to the high degree of uniformity in barge internal structural configurations (Cameron et al. 2007), impact forces are not sensitive to the width of the barge. The AASHTO provisions employ a barge width correction factor, $R_B$, to account for vessels with widths other than 35 feet. However, for a given pier shape and width, the finite element crushing simulations revealed no substantial differences between forces produced by crushing a 35-foot wide jumbo hopper barge and crushing a 50-foot wide tanker barge.

- Maximum collision force is dependent, in part, on the shape of the impacted pier surface. The AASHTO crush model does not account for impact surface geometry; however, the finite element crushing simulations indicate that rounded pier surfaces, as opposed to flat-faced surfaces, provide an effective means of mitigating the forces generated during barge impact events.

- Maximum barge impact force is related to the width of the impacted pier surface, particularly for flat-faced rectangular piers.

Based on these findings, design barge bow force-deformation relationships were developed [the formulation of these equations and an algorithm for determining the appropriate crush model for a given design scenario are detailed in Consolazio et al. 2008]. Additional simulations that were subsequently conducted by Consolazio et al. (2009a) resulted in minor changes being made to the proposed crush curves. The revised barge crush-model (force-deformation behavior) is shown in Figure 2-6 and is used throughout the remainder of this study.

All barge finite element simulations conducted as part of the 2008 and 2009a Consolazio et al. studies involved directly head-on impact scenarios. However, the probability of perfectly head-on collision between a barge and bridge is likely relatively small. A more likely scenario is that the barge collides with the pier at some small oblique angle. Thus, ongoing research is being conducted by the University of Florida (UF) and the Florida Department of Transportation (FDOT) to characterize barge impact forces corresponding to oblique collision scenarios. Preliminary findings have suggested that oblique collision forces are significantly smaller even when impact occurs at relatively small angles. Specifically, at oblique angles of 2°
or greater, impact forces associated with very wide (near the full barge width of 35 ft) flat-faced impact surfaces (e.g. pile caps placed at the waterline) are at least 30% smaller than head-on impact forces. Thus, for this study, barge collision forces are reduced by 30% for impact scenarios involving waterline pile caps [see Consolazio et al. 2009b for details regarding the 30% reduction].

**Coupled Vessel Impact Analysis (CVIA)**

Coupled vessel impact analysis (CVIA) involves coupling (linking) a single degree of freedom (SDOF) nonlinear dynamic barge model to a multi-degree of freedom (MDOF) nonlinear dynamic bridge analysis code. The term “coupled” refers to the use of a shared contact force between the barge and impacted bridge structure (Figure 2-7). The impacting barge is defined by a mass, initial velocity, and bow force-deformation (crush) relationship. Traveling at a prescribed initial velocity, the barge impacts a specified location on the bridge structure and generates a time-varying impact force in accordance with the crush curve of the barge and the relative displacements of the barge and bridge model at the impact location. The MDOF bridge model (pier, superstructure, and soil) is subjected to the time-varying dynamic impact force and consequently displaces, develops internal forces, and interacts with the SDOF barge model through the shared impact force.

The CVIA algorithm has been documented in detail in a number of previous publications (Consolazio and Cowan 2005; Consolazio and Davidson 2008; Consolazio et al. 2008) and has been implemented in the commercial pier analysis software package FB-MultiPier (2009). Since barge, pier, and superstructure stiffness and mass related forces are all included in CVIA, the method is able to accurately predict pier and substructure design forces under dynamic barge impact conditions. CVIA has been validated against full-scale experimental impact data (Consolazio and Davidson 2008) and presently constitutes a state-of-the-art computational tool.
for barge-bridge collision analysis when numerical efficiency and direct consideration of
dynamic effects are paramount. Because the CVIA procedure can accurately capture dynamic
amplifications of pier design forces due to superstructure inertial effects, it is used throughout
this study when dynamic assessments of structural demand are required.

**Dynamic Amplification of Pier Column Forces**

A recent study by Davidson et al. (2010) quantified dynamic amplification of pier column
internal forces for a wide range of pier and superstructure configurations. Dynamic amplification
was assessed by comparing pier column design forces predicted by dynamic CVIA (Figure 2-8a)
to forces predicted by static analyses in which peak dynamic impact loads $P_{B_{\text{max}}}$ (Figure 2-8b)
were applied. This research uncovered two distinct dynamic amplification modes—
superstructure inertial restraint and superstructure momentum-driven sway. Mixed cases were
also identified in which both modes of amplification were present, i.e. inertial restraint controlled
column moments, while sway controlled column shear forces. Regardless of which dynamic
amplification mode was most prominent, superstructure mass was found to play an important
role in driving bridge response during impact events.

Figure 2-9 illustrates the relative magnitude of dynamic amplification for a range of bridge
pier configurations and impact conditions—28 cases total. Impact cases are denoted using a
three-letter abbreviation corresponding to the bridge (e.g. ACS for the Acosta Bridge), a three
letter abbreviation for the location of the impacted pier (CHA for a pier adjacent to the
navigation channel and OFF for a pier located away from the navigation channel), and a single
letter denoting the impact energy (L, M, H, and S for low, moderate, high, and severe energy,
respectively) [See Davidson et al. 2010 for detailed descriptions of bridge configurations and
impact energy cases].
Data presented in Figure 2-9 are ratios of maximum dynamic pier column moment divided by maximum static pier column moment, obtained as shown in Figure 2-8. Presented in this way, a ratio greater than 1.0 indicates that dynamic analysis predicted a larger member demand than corresponding static analysis. In such cases, inertia-related effects amplify member forces beyond the magnitude that typical static analysis methods can predict. As shown in Figure 2-9, ratios for most cases range from 1.5 to 2.0, indicating that superstructure inertia amplifies column moments at least 50-100% for most impact scenarios, sometimes much more. Furthermore, for typical design situations, the magnitude of amplification for a given bridge configuration is not strongly sensitive to impact energy. For example, moment demands for the Acosta Bridge channel pier (ACS-CHA) are amplified by approximately 500% when pier dynamics are considered, regardless of impact energy. This finding implies that dynamic amplification is primarily a function of structural characteristics such as mass and stiffness distribution.
Figure 2-1. Force-deformation results obtained by Meier-Dörnberg (Adapted from Meier-Dörnberg 1983). A) results from dynamic cylindrical impact hammer test, B) results from dynamic 90° pointed impact hammer test and C) Results from static impact hammer test.
Figure 2-2. Relationships developed from experimental barge impact tests conducted by Meier-Dörnberg (1983) (Adapted from AASHTO 1991).

Figure 2-3. AASHTO relationship between barge crush depth and impact force.
Figure 2-4. Finite element simulation of barge bow crushing. 6 ft diameter round impact with jumbo hopper barge bow. A) 0-in. crush depth, B) 60-in. crush depth, C) 120-in. crush depth and D) 180-in. crush depth.
Figure 2-5. Barge bow force-deformation relationships. A) AASHTO crush curve (independent of impact surface characteristics) and B) Consolazio et al. (2008; 2009a) barge crush curve for 6-foot diameter round impact surface.
Figure 2-6. Barge bow force-deformation flowchart.
Barge and pier-soil system are coupled together through a common contact force $P_B$.

Figure 2-7. Coupling between barge and bridge in CVIA

Figure 2-8. Analyses conducted to quantify dynamic amplification. A) Dynamic CVIA and B) Static (using peak load, $P_B^{\text{max}}$, from CVIA)
Figure 2-9. Maximum pier column dynamic moments relative to static moments. (from Davidson et al. 2010)
CHAPTER 3
BRIDGE MODELING AND STUDY PRELIMINARIES

Introduction

To maximize the accuracy and computational efficiency of the coupled vessel impact analyses conducted in this study, several modeling and analysis procedures were employed, and, in selected cases, new features were developed and implemented into the FB-MultiPier finite element analysis code (FB-MultiPier 2009). FB-MultiPier can be used to perform linear or nonlinear, static or dynamic analyses on single pier models or full bridge (multi-pier, multi-span) models. As illustrated in Figure 3-1, FB-MultiPier bridge models generally contain the following components:

- Superstructure: Modeled using resultant frame elements with linear elastic material behavior.
- Bridge pier: Modeled using cross-section integrated frame elements in conjunction with nonlinear kinematic and constitutive material models.
- Pile cap: Modeled using thick shell elements with linear elastic material behavior.
- Foundation: Modeled using cross-section integrated frame elements in conjunction with nonlinear kinematic and constitutive material models.
- Soil: Modeled using nonlinear discrete spring elements, distributed along each embedded foundation element.

Although FB-MultiPier has the ability to directly analyze full multi-span, multi-pier bridge structures, in this study, one-pier two-span (OPTS) bridge models were used instead to increase computational efficiency (Consolazio and Davidson 2008). Using FB-MultiPier, an inventory was developed, consisting of twelve (12) finite element models of Florida bridges. All static and dynamic analyses conducted as part of this study utilized these twelve (12) models.
**Superstructure**

In an FB-MultiPier bridge model, the superstructure is represented using a series of linear-elastic resultant frame elements. As a result, the girders, deck, and other superstructure features are considered to act compositely during inter-pier force transmission. Rigid elements and multi-degree of freedom springs are used to connect the primary superstructure elements to each pier. The superstructure modeling capabilities of FB-MultiPier enable important static and dynamic interactions between bridge piers.

The use of linear resultant frame elements for superstructure modeling in FB-MultiPier necessitates the inclusion of additional elements to correctly distribute forces to the bearing locations of underlying piers. More specifically, intermediate superstructure elements are employed to approximate the effect of the physical footprint of the superstructure and its interaction with the pier.

As illustrated in Figure 3-1, the superstructure-substructure interface model consists of four primary parts—a rigid vertical link, a rigid horizontal transfer beam, bearing springs, and horizontal bearing offset rigid links. The vertical link is used to produce the correct relative height between the superstructure center of gravity and centerline of the pier cap beam. This distance can be quite large (reaching heights greater than 10 ft), especially for bridges with long-span haunched girders or box-girder superstructures. A horizontal transfer beam, connected to the vertical link, serves to distribute superstructure load to physical bearing pad positions. For piers with two bearing rows, additional horizontal rigid elements act to offset the bearing locations from the pier cap centerline.

Bearing pad elements are modeled as discrete stiffnesses. Arbitrary, nonlinear load-deformation relationships can be associated with each bearing DOF, providing the ability to model complex hyperelastic material behavior or finite-width gapping. For the purposes of this
research, this level of model sophistication was unwarranted. However, the use of non-rigid load-deformation relationships for these elements did provide a more realistic representation of bearing behavior when compared to constraint-based pin or roller supports. Therefore, empirical load-deformation relationships, as opposed to infinitely stiff bearing conditions, were employed for these model components.

In the course of conducting this study, it was observed that constraint-based (pin or roller) bearing conditions led to erratic distribution of bearing loads. For example, when constraints were employed, vertical bearing reactions under gravity loading could consist of widely varying compression and tension forces. For analysis cases where constraint-based bearing conditions were employed, the sum of vertical bearing reactions was consistently found to be statically equal to the superstructure dead load, however, the conspicuous load distribution led to undesired localized member force concentrations and erroneous deformation patterns.

To alleviate this problem, finite stiffness values were prescribed for each bearing spring DOF to simulate more realistic bearing conditions. To accomplish this, bridge models with constrained bearing conditions were first statically subjected to loading conditions similar to those associated with respective vessel impact events of interest. The stiffness of the rigid springs was incrementally reduced until the total bearing reaction was evenly distributed among the bearing elements. During this process, care was taken not to overly soften the bearing springs, as this would reduce the overall rigidity of the superstructure-substructure interface. Reducing the relative stiffness of the superstructure system could cause a larger portion of the vessel impact load to travel through the foundation, as opposed to the superstructure, resulting in unconservative pier column demand predictions. With this concern in mind, it was confirmed that the sum of the “non-rigid” bearing forces compared well with the sum of the
constraint-associated bearing forces. Consequently, it was observed that the softened bearings afforded a uniform distribution of bearing forces to the pier while maintaining comparable superstructure rigidity.

**Substructure**

Bridge substructures, as modeled in this study, consist of one or more pier columns, a pier cap beam, a pile cap, multiple driven piles or drilled shafts, and a numerical representation of the soil. Horizontal and vertical soil stiffness is modeled using nonlinear spring elements. Linear elastic, thick-formulation shell elements are used to model the pile cap.

**Pier Modeling**

Flexural substructure elements—pier columns, pier cap beam, and either piles or drilled shafts—are modeled using nonlinear cross-section integrated frame elements. For relatively light loading, such as that associated with certain service loading conditions, bridge structural components are typically assumed to remain effectively linear. As such, accurate predictions of pier behavior under this type of loading can be obtained using resultant frame elements with gross cross-sectional properties and linear-elastic material models.

However, barge-bridge collision is an extreme event scenario, potentially resulting in permanent damage or even collapse of the bridge structure. As a result, robust nonlinear analytical tools are necessary to capture the effects of widespread concrete cracking, plastic hinge formation, and permanent member deformation. Furthermore, bridge structural elements are commonly composed of multiple materials (i.e., reinforced or prestressed concrete). To address the composite nature of concrete pier elements, FB-MultiPier employs cross-section integrated frame elements that permit the cross-sectional shape and the locations of reinforcing bars or prestressing tendons to be modeled explicitly. When such elements are employed, the cross section is discretized into multiple regions of integration and relevant material models.
(stress-strain relationships) are assigned to each discrete area (integration point) as illustrated in Figure 3-2.

As the section flexes in general biaxial bending and deforms axially, a planar (linear) strain field is generated across the section. Nonlinear stress-strain relationships for each material (Figure 3-3) lead to a nonlinear distribution of stresses over the cross-section. Section axial force and moments are then computed by integrating (summing) the force and moment contributions from each integration point.

FB-MultiPier can employ either built-in stress-strain relationships or user-defined stress-strain relationships for concrete and steel. In Figure 3-3, built-in FB-MultiPier stress-strain relationships for typical bridge pier materials are illustrated. Concrete is modeled using a modified Hogenstead parabola in compression, with strain softening described by a straight line. It is important to note that the maximum compressive stress ($f'_{c}$) is 0.85 times the specified cylinder compressive strength ($f'_c$). In tension, concrete is treated as linear until the cracking stress ($f'_t$) is reached. After cracking, a bilinear model is employed to account for tension stiffening. For 270-ksi high-strength prestressing tendons, the stress-strain model proposed by the Precast/Prestressed Concrete Institute (PCI 2004) is employed. Mild reinforcing steel is treated as elastic, perfectly plastic with a yield stress of 60 ksi. When combined with the cross-section integration scheme, these material models provide accurate predictions of moment-curvature and plastic hinging behavior for reinforced and prestressed concrete members.

**Soil Modeling**

With regard to representing soil-structure interaction, FB-MultiPier utilizes empirical soil-strength models, where these models correlate pertinent soil parameters (e.g., internal friction angle, subgrade modulus) to deformation behavior under loading. Three primary modes
of soil-structure interaction are considered: lateral resistance (P-y), axial skin friction along the pile or shaft (τ-z), and tip bearing resistance (Q-z). For each mode, nonlinear curves define relationships between resistance and displacement. Example soil resistance curves are illustrated in Figure 3-4.

Due to the distributed nature of soil resistance, soil-structure interaction in each orthogonal direction is represented using nonlinear spring elements distributed along the embedded pile (or drilled shaft) length. One vertical spring at each nodal location models the tributary skin friction (τ-z). Additional spring elements are placed at each pile tip node to represent the tip resistance (Q-z).

Modeling lateral (horizontal) soil resistance requires a slightly more refined approach. FB-MultiPier incorporates user-defined pile row multipliers (p-y multipliers) to account for pile group behavior. Each row of piles is assigned a factor by which the lateral resistance is scaled, depending on pile spacing and position within the group (Figure 3-5). However, these factors are dependent on the direction of pile group motion. For example, a pile on the leading row may be assigned a factor of 0.8. However, if the foundation motion reverses direction, that same pile becomes part of a trailing row, for which a row multiplier of 0.3 is applied. Similarly, when group motion occurs in the transverse direction, unique multipliers are used. Consequently, at every node, four horizontal spring elements define the lateral soil resistance in two orthogonal directions. The nonlinear stiffness of each spring is scaled by the applicable row multiplier, providing a realistic representation of lateral pile group behavior.

Behavior of soil-spring elements under cyclic lateral loading can be considered either nonlinear-elastic or nonlinear-inelastic, depending on the soil type. Typically, for cohesionless soils, such as sands, the material exhibits nonlinear-elastic behavior, returning to the original
undeformed state when load is removed. In contrast, cohesive soils (e.g., clay) are often considered nonlinear-inelastic and gaps may form between soil and pile surfaces during cyclic loading. FB-MultiPier soil models can account for this type of behavior; however, a lack of available, dynamic soil data for many of the bridges studied precluded the direct incorporation of such effects in the current study.

**Bridge Inventory and Impact Conditions**

Later in this thesis, a parametric study is conducted to facilitate the development of the proposed equivalent static analysis method and to assess the accuracy of the method when compared to dynamic analysis. The parametric study makes use of FB-MultiPier numerical models (using the features previously described) of Florida bridges of varying age, pier configuration, superstructure type, foundation type, and soil conditions subjected to barge impact scenarios of varying energy. Specific bridge structures and impact conditions that are considered in the parametric study are documented below.

**Bridge Inventory**

Twelve (12) models were developed for various piers and spans of nine (9) different bridges (Table 3-1) located in the state of Florida. For conciseness, and to simplify discussion, each bridge structure is assigned a three-letter identification code. Specific piers within each bridge are further delineated by proximity (with respect to the vessel transit path): the letters “CHA” appended to a bridge identification code indicate that the pier is a channel pier, whereas the letters “OFF” indicate an off-channel pier (a pier not directly adjacent to the channel). Bridges listed in Table 3-1—selected from a larger Florida Department of Transportation (FDOT) catalog of almost two-hundred bridges—constitute a representative cross-section of bridge types currently in service in Florida. The nine (9) selected bridges also vary widely in age, with construction dates spanning from the late 1960s to 2004.
Various past analytical studies (Consolazio et al. 2009a; Yuan et al. 2008) have demonstrated that the geometry (shape and width) of the impacted portion of a bridge pier affects the magnitude of impact loads generated during collision. Hence, in the parametric study, bridges were selected that vary in both pier column shape (flat-faced, round) and pier column width (ranging from 3.5 ft to 20 ft). Pier shape, foundation type (pile-and-cap or drilled shaft), and size data are summarized in Table 3-1.

The extent to which load applied to an impacted pier is transmitted to adjacent piers depends upon the type of superstructure that connects the piers together. Three common superstructure types are included in the bridge inventory employed in the parametric study (Table 3-1): concrete slab on concrete girders; concrete slab on steel girders; and, segmental concrete box girder. Superstructure span lengths included in this study are representative of common, moderate-span bridges that span U.S. inland waterways as opposed to less common, long-span bridges.

**Barge Impact Conditions**

Jumbo hopper barges are the most common type of vessel found in the U.S. barge fleet, and additionally, constitute the baseline design vessel for barge-bridge collision in the AASHTO provisions. For these reasons, a jumbo hopper barge was employed as the design vessel in the parametric study. Prescribed barge impact conditions for bridges in the parametric study were chosen to span the range of collision events that are conceivable for navigable Florida waterways. Vessel weight (flotilla weight) and collision velocity were varied to produce a representative range of impact energy cases: low, moderate, high, and severe (Table 3-2). Based on waterway traffic characteristics, pier location (relative to the navigation channel), and pier strength, one or more suitable impact energy conditions were assigned to each pier in the study.
The AASHTO provisions require that all bridge components located in navigable water depths be designed for, at a minimum, impact from a single empty hopper barge (200 tons) drifting at a velocity equal to the mean water current for the waterway (AASHTO 2009). This low-energy “drifting barge condition” is meant to be representative of a barge that breaks loose during a storm and drifts into a pier. Such a condition is only relevant to the design of piers distant from the navigation channel (since near-channel piers must be designed for greater impact energies, which are associated with errant, tug-propelled barge flotillas). Therefore, the low-energy impact condition was only applied to off-channel piers (Table 3-2). Using water-current data for several waterways in Florida, an approximate average current velocity of 1 knot was determined. Thus, the low-energy case is defined as a 200-ton barge drifting at a velocity of 1 knot.

The majority of impact cases considered in this study fall into the categories of either moderate or high energy (Table 3-2). A moderate-energy impact condition is defined as one fully-loaded hopper barge (2030 tons with tug) traveling at 2.5 knots, and a high-energy impact condition is defined as a flotilla consisting of three fully-loaded hopper barges (5920 tons with tug) traveling at 5.0 knots. These conditions cover the majority of possible impact energies that would be generated by collisions from typical Florida barge traffic (Liu and Wang 2001). For two of the piers considered in this study (channel piers of the Acosta and New St. George bridges), barge traffic and waterway conditions warrant the definition of an additional severe-energy impact condition: a flotilla consisting of four fully-loaded hopper barges (7820 tons with tug) traveling at 7.5 knots.
Table 3-1. Bridge pier configurations.

<table>
<thead>
<tr>
<th>Bridge ID</th>
<th>Bridge name</th>
<th>Cap elevation</th>
<th>Impact surface shape</th>
<th>Impact surface width (ft)</th>
<th>Pier column data</th>
<th>Shaft/pile data</th>
<th>Superstructure type</th>
<th>Span lengths adjacent to pier (ft)</th>
</tr>
</thead>
<tbody>
<tr>
<td>ACS-CHA</td>
<td>Acosta</td>
<td>Waterline</td>
<td>Flat</td>
<td>50.0</td>
<td>1    20</td>
<td>40 Drilled shaft</td>
<td>31 5.0 X</td>
<td>370 620</td>
</tr>
<tr>
<td>BLT-CHA</td>
<td>SR-20 at Blountstown</td>
<td>Waterline</td>
<td>Round</td>
<td>9.0</td>
<td>2    5.5</td>
<td>37 Drilled shaft</td>
<td>2 9.0 X</td>
<td>280 220</td>
</tr>
<tr>
<td>EGB-CHA</td>
<td>Eau Gallie</td>
<td>Mudline</td>
<td>Flat</td>
<td>4.0</td>
<td>4    4.0</td>
<td>69 Steel pile</td>
<td>39 1.2 X</td>
<td>150 150</td>
</tr>
<tr>
<td>MBC-CHA</td>
<td>Melbourne</td>
<td>Mudline</td>
<td>Round</td>
<td>4.5</td>
<td>2    4.5</td>
<td>72 Concrete pile</td>
<td>32 1.5 X</td>
<td>145 110</td>
</tr>
<tr>
<td>NSG-CHA</td>
<td>New St. George Island</td>
<td>Waterline</td>
<td>Flat</td>
<td>28.0</td>
<td>2    6.0</td>
<td>52 Concrete pile</td>
<td>15 4.5 X</td>
<td>250 260</td>
</tr>
<tr>
<td>NSG-OFF</td>
<td>New St. George Island</td>
<td>Waterline</td>
<td>Flat</td>
<td>28.0</td>
<td>2    5.5</td>
<td>52 Concrete pile</td>
<td>9 4.5 X</td>
<td>140 140</td>
</tr>
<tr>
<td>OSG-CHA</td>
<td>Old St. George Island</td>
<td>Mudline</td>
<td>Flat</td>
<td>5.7</td>
<td>2    5.5</td>
<td>47 Steel pile</td>
<td>40 1.2 X</td>
<td>250 180</td>
</tr>
<tr>
<td>OSG-OFF</td>
<td>Old St. George Island</td>
<td>Waterline</td>
<td>Flat</td>
<td>3.5</td>
<td>2    3.5</td>
<td>40 Concrete pile</td>
<td>8 1.7 X</td>
<td>74 74</td>
</tr>
<tr>
<td>PNC-CHA</td>
<td>Pineda</td>
<td>Mudline</td>
<td>Round</td>
<td>4.5</td>
<td>2    4.5</td>
<td>73 Concrete pile</td>
<td>30 1.7 X</td>
<td>120 68</td>
</tr>
<tr>
<td>RNG-CHA</td>
<td>John Ringling</td>
<td>Waterline</td>
<td>Round</td>
<td>13.0</td>
<td>1    13.0</td>
<td>40 Drilled shaft</td>
<td>2 9.0 X</td>
<td>300 300</td>
</tr>
<tr>
<td>RNG-OFF</td>
<td>John Ringling</td>
<td>Waterline</td>
<td>Round</td>
<td>13.0</td>
<td>1    13.0</td>
<td>25 Drilled shaft</td>
<td>2 9.0 X</td>
<td>300 300</td>
</tr>
<tr>
<td>SBZ-CHA</td>
<td>Seabreeze</td>
<td>Mudline</td>
<td>Flat</td>
<td>8.0</td>
<td>1    8.0</td>
<td>58 Concrete pile</td>
<td>32 2.0 X</td>
<td>250 250</td>
</tr>
</tbody>
</table>

\(a\) Waterline footing indicates a foundation top-surface elevation near the mean high water level.

\(b\) Mudline footing indicates a foundation top-surface elevation near the soil surface.

\(c\) Distance from top of foundation to bottom of pier cap.
Table 3-2. Analysis matrix of barge impact energy conditions.

<table>
<thead>
<tr>
<th>Impact energy condition</th>
<th>Low (L)</th>
<th>Moderate (M)</th>
<th>High (H)</th>
<th>Severe (S)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Impact condition</td>
<td>1.0</td>
<td>2.5</td>
<td>5.0</td>
<td>7.5</td>
</tr>
</tbody>
</table>

<table>
<thead>
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<th>Impact condition characteristics</th>
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<th>Weight (tons)</th>
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<td>EGB-CHA</td>
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<tr>
<td>MBC-CHA</td>
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<td>RNG-OFF</td>
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<tr>
<td>SBZ-CHA</td>
<td>X X</td>
</tr>
</tbody>
</table>
Figure 3-1. Overview of superstructure model configuration in FB-MultiPier.
Each integration point assigned a material model.

Full cross section integrated to obtain section forces:

\[ M_z = \int_A \sigma_y \, dA \approx \sum_i \sigma_i \, Y_i \, dA \]

\[ M_y = \int_A \sigma_z \, dA \approx \sum_i \sigma_i \, Z_i \, dA \]

\[ P = \int_A \sigma \, dA = \sum_i \sigma_i \, dA \]

Figure 3-2. Cross section integration scheme for nonlinear frame elements.
Figure 3-3. Nonlinear material models as implemented in FB-MultiPier. A) concrete, B) mild steel and C) prestressing steel.
Figure 3-4. Typical soil resistance curves employed by FB-MultiPier. A) lateral soil resistance (P-y) curve, B) skin friction (τ-z) curve and C) tip resistance (Q-z) curve.
Figure 3-5. Variation of row multipliers for differing pile group motion.
(Note: Specific multiplier values will vary based on pile spacing)
CHAPTER 4
STATIC BRACKETED IMPACT ANALYSIS (SBIA) METHOD

Introduction

As illustrated by prior research (Consolazio et al. 2006; Consolazio et al. 2008; Davidson et al. 2010), current static analytical methods for barge-bridge collision do not account for dynamic amplification of pier member forces exhibited during impact events. Consequently, design forces obtained using traditional static procedures can be markedly unconservative. The ideal means by which to simulate inertial forces stemming from an impact event—and accurately predict member forces—is to use a fully dynamic time-history structural analysis procedure such as CVIA. However, such simulation procedures can be computationally expensive (relative to static analysis) and require a relatively detailed numerical description of the structure. These issues may be prohibitive in some design situations, particularly during preliminary bridge design when pertinent bridge details are subject to significant variation and frequent revision.

To address the accuracy limitations of current static analysis and the computational requirements of dynamic time-history analysis while still accounting for inertial effects manifested during barge-bridge collision events, an equivalent static analysis method is developed, demonstrated, and verified in this chapter. The newly developed equivalent static analysis method is both simple to use and minimally conservative when compared to dynamic analysis (i.e., it accounts for dynamic amplification).

Conceptual Overview

The primary limitation of static analysis methods is the assumption that lateral pier resistance is provided only by soil and superstructure stiffness (Figure 4-1a). However, acceleration of superstructure mass results in significant inertial restraint at the top of the pier immediately after impact (Figure 4-1b). In many cases, superstructure inertial resistance will
equal or greatly exceed stiffness based resistance. Consequently, for most bridges, momentary maximum pier forces are manifested during this early stage of the impact event (Davidson et al. 2010)

In addition to inertial restraint based dynamic amplification, a second mode of amplification was also identified by Davidson et al. (2010): superstructure momentum-driven sway. In this case, maximum pier forces occur later in time, perhaps after the barge is no longer in contact with the pier. In this mode, oscillation of the superstructure mass acts as a source of loading, ultimately driving maximum pier forces. However, even in cases where superstructure momentum is the dominant dynamic amplification mode, an initial spike in member forces occurs as a result of inertial restraint (resistance) at the pier top. For the sway-controlled cases described in Davidson et al. (2010), this initial restraint based demand was 70-80% of the maximum demands observed later, resulting from superstructure sway. As such, the superstructure inertial resistance phenomenon is present and significant for all structural configurations considered by Davidson et al. (2010) In addition, no simplified means has been identified to predict which dynamic amplification mode will dominate the response of a given bridge, aside from conducting a time-history dynamic analysis. Therefore, focus is given to developing static analysis procedures that approximate superstructure inertial resistance, which can be scaled up to conservatively envelope the sway-driven values as well.

**Superstructure Modeling**

Three primary superstructure stiffness modeling schemes were considered for use in this study:

- Full multiple-pier, multiple-span bridge model
- One-pier, two-span (OPTS) bridge model including impacted pier and adjacent spans
- One-pier model with a single lateral spring representing the superstructure stiffness
However, it has previously been demonstrated (Consolazio and Davidson 2008) that one-pier, two-span (OPTS) bridge descriptions provides accurate approximations of both static and dynamic behavior of full multiple-pier multiple-span bridge models. As such, OPTS modeling was employed in development of the equivalent static analysis method, but full bridge models were not.

To develop and verify the equivalent static analysis method, both OPTS and spring-based superstructure models were considered (Figure 4-2). While the OPTS modeling technique is more accurate, it is common practice in bridge design to represent superstructure stiffness with a single equivalent spring. As such, it is important to also consider this approach when assessing the suitability of proposed analysis methods for design practice.

**Static Impact Load Determination**

A critical step in any impact analysis is quantifying the maximum dynamic load to which the structure is subjected. In the equivalent static method developed here, maximum magnitude dynamic loads are determined, in part, by using the principle of conservation of energy (Consolazio et al. 2008) as follows:

\[
\Delta KE_{if} + \Delta DE_{if} = 0
\]  
(4-1)

where \( \Delta KE_{if} \) is the change in kinetic energy of the barge, and \( \Delta DE_{if} \) is the change in total deformation energy (i.e. the sum of the elastic and plastic deformation energies), associated with the deformation of the barge bow, from the initial state (i) to the final state (f). Conservation of energy is used to define a relationship between the maximum impact load, and the barge parameters.

Assuming that barge mass does not change during the collision, the change in barge kinetic energy can be expressed as:
\[
\Delta KE_{if} = \frac{1}{2} m_B \left( v_{Bf}^2 - v_{Bi}^2 \right) \quad (4-2)
\]
where \( m_B \) is the constant (unchanging) mass of the barge and \( v_{Bf} \) and \( v_{Bi} \) are the magnitudes of the barge velocities at the initial and final states respectively.

In general, the deformation energy for the barge can be described using the following relation:

\[
\Delta DE_{if} = \int_{a_{Bi}}^{a_{Bf}} P_B(a_B) \, da_B \quad (4-3)
\]
where \( P_B(a_B) \) is the impact force as a function of barge crush depth \( (a_B) \), and \( a_{Bi} \) and \( a_{Bf} \) are the barge crush depths at the initial and final states respectively.

To estimate the maximum impact load acting on the pier, the following assumptions were made: 1.) the initial barge crush depth \( (a_{Bi}) \) is assumed to be zero, and 2.) the barge bow force-deformation relationship is assumed to be elastic perfectly-plastic (Figure 4-3), and 3) the pier is initially assumed to be rigid and fixed in space (note that this assumption is ultimately removed, as described later).

The first assumption implies that the initial kinetic energy of the barge is fully converted into deformation energy of the barge bow during loading of the barge (Figure 4-4a). Thus, once all of the barge initial kinetic energy has been converted into deformation of the barge bow (i.e. the barge velocity becomes zero) the barge bow crush depth has reached its maximum value. Additionally, when the barge bow recovers the elastic portion of its deformation energy through unloading, this energy is then converted back into rebound motion of the barge (Figure 4-4b). Final barge kinetic energy can then be determined from the recovered deformation energy.

If the barge bow remains linear and elastic, the conservation of energy up to the point of maximum barge bow deformation can be represented by the following equation:
\[ \Delta KE_{im} + \Delta DE_{im} = -\frac{1}{2} m_B v_{Bi}^2 + \frac{1}{2} P_B a_{Bm} = 0 \]  \hspace{1cm} (4-4)

where \( P_B \) is the maximum impact force observed during the impact, and \( a_{Bm} \) is the maximum barge bow deformation. The maximum impact force and barge bow deformation however, remain undetermined up this point, and thus, an additional equation is required.

If the barge bow remains elastic, the maximum bow deformation can be defined as follows:

\[ a_{Bm} = \frac{P_B}{k_B} = \frac{P_B}{(P_{BY}/a_{BY})} \]  \hspace{1cm} (4-5)

where \( a_{BY} \) and \( P_{BY} \) are the barge bow deformation and force at yield, respectively, and \( k_B \) is the initial elastic stiffness of the barge bow. Combining Eqns. 4-4 and 4-5, and solving for the maximum load produces the following equation:

\[ P_B = v_{Bi} \sqrt{\frac{P_{BY}}{a_{BY}}} m_B = v_{Bi} \sqrt{k_B m_B} \leq P_{BY} \]  \hspace{1cm} (4-6)

Due to the elastic perfectly-plastic assumption for the barge bow force-deformation relationship, the maximum barge impact force is limited to the yield load of the barge bow.

Validation of the analytical model in Eqn. 4-6 revealed that the rigid-pier assumption is overly conservative in cases where the barge deformation remains elastic (Consolazio et al. 2008). This finding necessitates inclusion of the finite stiffness of the impacted pier-soil system into the formulation. The stiffness of the barge and pier-soil system are then combined to form a series stiffness (\( k_S \)).

\[ k_S = \left( \frac{1}{k_B} + \frac{1}{k_p} \right)^{-1} = \left( \frac{a_{BY}}{P_{BY}} + \frac{1}{k_p} \right)^{-1} \]  \hspace{1cm} (4-7)
where $k_P$ is the linear stiffness pier-soil system. Replacing the initial elastic barge stiffness ($k_B$) in Eqn. 4-6 with the effective barge-pier-soil series spring stiffness ($k_S$) (Eqn. 4-7) produces the following equation:

$$P_B = v_{Bi} \sqrt{k_S m_B} = v_{Bi} \cdot c_{BP} \leq P_{BY}$$

(4-8)

where $c_{BP}$ is the barge-pier pseudo-damping coefficient, defined as follows:

$$c_{BP} = \sqrt{k_S m_B}$$

(4-9)

By incorporating the series stiffness ($k_S$) in place of the barge stiffness ($k_B$), the accuracy of the load prediction model is greatly enhanced, providing a reliable means of estimating the peak vessel impact load. See Consolazio et al. (2008) for additional details and verification of the procedure.

**Potential Static Approximations of Superstructure Inertial Resistance**

In previous research (Consolazio et al. 2006; Consolazio et al 2008; Davidson et al. 2010), superstructure inertial resistance was identified as the dominant source of dynamic pier force amplification during vessel impact events. Current static analytical methods (e.g., AASHTO) do not account for such dynamic effects. To address this important omission, three analytical schemes were assessed to statically approximate the additional source of superstructure resistance attributable to inertia:

- Restrain the pier top with an infinitely stiff lateral boundary condition (Figure 4-5a)
- Amplify the lateral superstructure stiffness (Figure 4-5b)
- Directly apply an inertial load at the superstructure level (Figure 4-5c)

Each of these approaches involved modifying one of three components common to all static analyses: boundary conditions, stiffness, and loads.
Approximating superstructure inertial resistance by means of boundary conditions (i.e. fixing the pier-top elevation) generally proved overly conservative; static predictions of pier demands greatly exceeded those quantified by dynamic simulations. In addition, the level of conservatism varied greatly between various structural configurations. This result is, in part, predictable, as a fixed boundary condition allows no lateral deflection at the pier-top elevation. Dynamic simulations consistently show positive, non-zero displacements (in the direction of impact) at this elevation during times at which maximum structural demands occur. As such, a fixed pier-top boundary condition provides an unreasonable level of restraint.

Given these findings, use of a finite-stiffness boundary condition at the pier-top elevation was considered. As noted previously, static superstructure stiffness alone does not provide sufficient resistance to adequately predict dynamic amplification effects. Thus, the lateral superstructure stiffness was amplified to account for both static superstructure resistance and superstructure inertial resistance. This approach proved reasonably effective in producing displaced pier shapes and member demand predictions that were consistent with dynamic analysis. However, a large degree of variability in the stiffness magnification factors was observed across a range of structural configurations. Furthermore, no correlation was observed between these magnification factors and corresponding bridge parameters. Consequently, amplifying superstructure stiffness was deemed an impractical means of approximating of superstructure inertial resistance.

Given the significant conservatism and variability associated with boundary condition and stiffness-based approximations of inertial resistance, a third major static approach was considered—approximating the superstructure inertial resisting force and applying it as a static load (Figure 4-5c). As was the case with magnified superstructure stiffnesses, static inertial
forces varied substantially among differing structural configurations. However, correlations were identified between the static inertial loads and corresponding bridge parameters, thus providing an accurate static approximation of dynamic pier behavior.

**Static Approximation of Inertial Resistance by Direct Load Application**

An empirical approach was employed to develop equivalent static loading conditions that provide accurate predictions of both maximum dynamic member forces and dynamic pier displacements. Ideal static loading conditions were determined for twelve (12) Florida bridge piers, incorporating a wide range of common pier styles. In addition, one or more representative impact energy conditions were considered for each bridge, resulting in twenty (20) pier/energy cases. Load prediction equations were then developed that correlate statically equivalent inertial loads to readily available bridge parameters.

**Factored Impact Load**

Static superstructure inertial forces were found to be strongly sensitive to the choice of impact load. For any given impact-point load (within a reasonable range), a corresponding pier-top load can be identified that will provide an adequate prediction of dynamic member forces (shears and moments). However, if the impact point load is too small, unrealistic pier displacements result. Specifically, if the maximum dynamic impact load is applied, as calculated from Eqn. 4-8, the corresponding pier-top loads must be very large to obtain member forces similar to those predicted dynamically. In many cases, the large pier-top load causes the pier to deflect into the negative range (opposite of the impact direction). Negative displacements were not observed from dynamic analysis, thus this static response is undesirable.

To counteract this tendency, a factored impact load was employed. Use of an amplified impact load causes piers, as a whole, to deflect in the impact direction, avoiding possible unrealistic negative pier displacements. Through an iterative process, an impact load factor of
1.45 (Figure 4-6) was found to be reasonably ideal (optimal) in terms of producing realistic displaced shapes, while still providing accurate estimations of dynamic member forces.

**Determination of Ideal Pier-Top Loads**

Having factored the peak impact load ($P_B$) by 1.45, ideal static pier-top inertial loads ($P_{BI}$) were determined by iteration. An ideal pier-top load is defined as the load that generates a static column moment equal to the maximum dynamic column moment. For each of the twenty (20) pier/energy cases studied, ideal pier-top loads were determined utilizing both OPTS and spring-based superstructure models (recall Figure 4-2). Ideal pier-top inertial loads were normalized by corresponding impact loads ($P_B$) to form ideal inertial resistance factors ($IRF_{ideal}$), as shown in Figure 4-7. Two additional sets of ideal inertial resistance factors (IRF) were analogously developed, corresponding to pier column shear and total bearing shear.

**Correlation of Pier-Top Inertial Resistance Factors (IRFs) to Bridge Parameters**

In the interest of developing a universal loading condition for all bridge configurations, one possible approach is to use the maximum IRF value observed among all bridges types. However, significant variation in ideal IRF magnitudes was observed between differing structural configurations—IRF values ranged from a minimum of 0.02 to a maximum of 1.17. Thus, applying an equivalent inertial load equal to $1.17 \cdot P_B$ to a bridge for which the ideal IRF is 0.02 is unreasonably conservative. To mitigate excessive conservatism, ideal IRF values were correlated to key structural parameters, to form empirical IRF prediction equations. These equations allow for computation of bridge-specific IRFs, providing a more accurate result than simply utilizing a uniform load factor for all bridges. Additionally, IRF prediction equations were calibrated separately to each major pier demand type—column moment, column shear, and total bearing shear—so that conservatism was minimized.
Several bridge parameters were considered for possible correlation to inertial resistance. Quantities such as natural frequency or superstructure acceleration generally control dynamic pier behavior. However, quantities of this nature are not readily determined without first conducting a dynamic analysis. Furthermore, during the early portions of the bridge design process, sufficient information may simply not be available to estimate such dynamic properties. Instead, ideal IRF values were correlated to parameters that can be easily quantified or estimated at any practical stage of design. Parameters were considered that involve geometry, mass distribution, and stiffness of the various bridge components—all of which influence dynamic pier behavior. The following quantities were considered:

- Pier height \( h_p \)
- Span length \( L_s \)
- Superstructure weight \( W_{sup} \)
- Superstructure stiffness \( k_{sup} \)
- Pier weight \( W_p \)
- Pier stiffness \( k_p \)

Because these parameters were identified as likely predictors of inertial bridge response, it was expected that a combination of these parameters would correlate well with computed ideal IRF values. To assess this expectation, an algorithm was developed that systematically evaluated each of several thousand possible combinations of the six (6) bridge parameters listed above \( (p_1 \ldots p_6) \), subject to the following functional form:

\[
\gamma = \left( p_1^{\alpha_1} \right) \left( p_2^{\alpha_2} \right) \left( p_3^{\alpha_3} \right) \left( p_4^{\alpha_4} \right) \left( p_5^{\alpha_5} \right) \left( p_6^{\alpha_6} \right)
\]

where, \( \alpha_1 \ldots \alpha_6 = +1, -1, +\frac{1}{2}, -\frac{1}{2}, +\frac{1}{4}, -\frac{1}{4} \)
For each possible combination, the product of bridge parameters ($\gamma$) was computed for each bridge considered in the study. Linear regression was used to correlate $\gamma$ values to observed ideal IRF factors for each pier/energy condition. To quantify the level of dependency between $\gamma$ and IRF, correlation coefficients ($r$) were computed for each of the several thousand trial correlations. This combinatorial analysis yielded viable relationships between the six bridge parameters and IRF. Specifically, the following parameter combination was most meaningfully correlated with IRF:

$$\gamma = \frac{1}{h_p} \sqrt[3]{\frac{k_{sup}}{W_p}}$$

(4-11)

By correlating each of the three sets of ideal IRF values to $\gamma$ (as computed in Eqn. 4-11), the following linear regression equations result (Figure 4-8):

$$\text{IRF}_{m}^{\text{ideal}} = 0.12 + 3.4 \cdot \gamma = 0.12 + \frac{3.4}{h_p} \sqrt[3]{\frac{k_{sup}}{W_p}}$$

(4-12)

$$\text{IRF}_{v}^{\text{ideal}} = 0.24 + 1.6 \cdot \gamma = 0.24 + \frac{1.6}{h_p} \sqrt[3]{\frac{k_{sup}}{W_p}}$$

(4-13)

$$\text{IRF}_{b}^{\text{ideal}} = 0.20 + 5.1 \cdot \gamma = 0.20 + \frac{5.1}{h_p} \sqrt[3]{\frac{k_{sup}}{W_p}}$$

(4-14)

where $\text{IRF}_{m}^{\text{ideal}}$ (Figure 4-8a), $\text{IRF}_{v}^{\text{ideal}}$ (Figure 4-8b), and $\text{IRF}_{b}^{\text{ideal}}$ (Figure 4-8c) are ideal IRF factors calibrated to pier moment, pier shear, and total bearing shear, respectively.

Correlation coefficients ($r$) were computed for the correlations defined by Eqns. 4-12, 4-13, and 4-14 (0.88, 0.59, and 0.85 respectively). Each of the correlation coefficients exceed the Pearson’s critical $r$ value of 0.561 (Pearson and Hartley 1958)—for a sample size of 20 and 0.01 significance—implying that the correlations are statistically meaningful.
Note that Eqns. 4-12, 4-13, and 4-14 do not constitute a static analytical method that is conservative when compared to dynamic analysis. These regression equations predict conservative values of IRF (and corresponding pier-top forces) for only about 50% of conceivable cases. Consequently, an upper bound envelope is needed that has a greatly increased likelihood of conservatism. Thus, envelopes were developed for each correlation, corresponding to a 99% confidence level (using the Student’s t-distribution)—implying that the resulting static pier demands are 99% likely to exceed the corresponding dynamic demands. Envelopes of this form for each demand type are described by the following equations (Figure 4-8):

\[
\text{IRF}_m = 0.22 + \frac{4.5}{h_p} \sqrt{\frac{k_{\text{sup}}}{W_p}}
\]

\(\text{(4-15)}\)

\[
\text{IRF}_v = 0.36 + \frac{3.0}{h_p} \sqrt{\frac{k_{\text{sup}}}{W_p}}
\]

\(\text{(4-16)}\)

\[
\text{IRF}_b = 0.37 + \frac{7.0}{h_p} \sqrt{\frac{k_{\text{sup}}}{W_p}}
\]

\(\text{(4-17)}\)

Thus, dynamic column moment, column shear, and total bearing shear demands may be conservatively estimated by statically analyzing the impacted pier under the load conditions illustrated in Figure 4-9. Note that three distinct static analyses must be conducted, each used to predict the maximum demand corresponding to the chosen IRF. For example, when the structure is analyzed using the IRF corresponding to pier moment (IRF\(_m\)), the column shears and bearing shears predicted by this analysis are not utilized.

The static loading scheme illustrated in Figure 4-9 constitutes a conservative—but minimally so—static analysis procedure for predicting both direct load effects of barge impact and indirect dynamic amplifications caused by inertial superstructure resistance. Prediction
equations for pier-top inertial resistance factors (Eqns. 4-15, 4-16, and 4-17) include structural parameters that are important to dynamic amplification yet are readily quantified during design. Furthermore, conservatism—relative to dynamic analysis—is minimized by separately analyzing for pier moment, pier shear, and total bearing shear.

**Substructure Considerations**

While the static loading conditions shown in Figure 4-9 provide adequate predictions of pier column and bearing shear demands, dynamic substructure forces (pile/shaft moments and shears) are consistently underestimated by this procedure. Because the static inertial load \( (\text{IRF} \cdot P_B) \) opposes the amplified impact load \( (1.45 \cdot P_B) \), loads transmitted through the foundation to the soil are relieved. Thus, foundation demands must be considered separately, excluding a pier-top inertial load. An empirical static loading condition was developed for predicting pile/shaft design forces, implementing similar methodology to that described above.

**Determination of Ideal Impact-Point Loads**

To develop a load model for predicting dynamic foundation demands, ideal impact-point loads were developed for each of the twenty (20) pier/impact energies considered. As illustrated in Figure 4-10, the ideal amplified impact load \( (P_{\text{ideal}})^\text{amp} \) is defined as the load for which the static foundation moment \( (M_{\text{static}}^{\text{max}}) \) equals the maximum moment predicted by dynamic analysis \( (M_{\text{dyn}}^{\text{max}}) \). For each case, ideal amplified impact loads were normalized by corresponding approximate impact loads \( (P_B) \) to form ideal impact-point dynamic magnification factors \( (\text{DMF}_{\text{ideal}}) \).

**Correlation of Impact-Point DMF to Bridge Parameters**

Ideal impact-point DMF values for the cases considered ranged from 0.93 to 1.6—implying that the impact force \( (P_B, \text{calculated from Eqn. 4-8}) \) is too small to provide conservative
estimates of dynamic pile/shaft demands, in most cases. Thus, a dynamic magnification factor must be considered for impact loads.

A combinatorial study was conducted using the methodology described previously to identify correlations between bridge structural parameters and impact-point DMF. As before, several-thousand trial correlations were computed and assessed by means of correlation coefficients (r). In contrast to the previous investigation, no statistically meaningful correlations were observed between bridge parameters and impact-point DMF. Consequently, the impact-point DMF was treated as uniform across all structural configurations and impact conditions. Among the twenty (20) cases studied, the mean DMF was 1.34. However, an upper bound envelope was desired to greatly increase the probability of conservatism. Thus, a uniform envelope of 1.85 was established (Figure 4-11)—corresponding to a 99% confidence upper bound—using the Student’s t-distribution. Thus, by amplifying the barge impact load (\(P_B\)) by a factor of 1.85, conservative estimates of foundation design forces are produced.

**Static Bracketed Impact Analysis (SBIA) Method**

The static bracketed impact analysis (SBIA) method consists of bracketing (or enveloping) member design forces using the two static loading conditions described above. In this manner, pier column, bearing shear, and foundation design forces are conservatively quantified with regard to dynamic amplifications. The SBIA method is summarized and demonstrated in this section.

**SBIA Overview**

The proposed SBIA procedure (Figure 4-12) consists of two primary load cases. Load Case 1 involves statically applying both an amplified impact load equal to \(1.45 \cdot P_B\) and a statically equivalent superstructure inertial load equal to \(\text{IRF} \cdot P_B\). The magnitude of IRF depends
on both bridge structural parameters ($h_p$, $k_{sup}$, and $W_p$) and the desired pier demand type (pier moment, pier shear, or total bearing shear). Typically, Load Case 1 controls the design of pier columns and bearing connections to resist vessel collision forces. Load Case 2 consists of a single amplified impact load equal to $1.85 \cdot P_B$ and typically controls the design of foundation elements such as piles or drilled shafts.

It should be noted that feasible structural configurations and impact conditions exist for which the “typical” controlling load case does not control for a given member type. Consequently, maximum pier demands obtained between both load cases should be considered for design (i.e. the results must be bracketed by both load cases). This bracketing approach is consistent with widely accepted structural design practice concerning multiple loading conditions. Figure 4-12 provides a summary of the entire SBIA procedure as well as specific definitions for the quantities $h_p$, $k_{sup}$, and $W_p$ needed in calculating IRF values.

**SBIA Demonstration**

In this section, the SBIA method is demonstrated in detail for one of the twenty (20) pier/energy cases considered in this study—the Blountstown Bridge channel-pier high-energy case (BLT-CHA-H). The bridge is analyzed using both OPTS and spring superstructure models (recall Figure 4-2). Refer to Table 3-1 and Table 3-2, in the previous chapter, for additional details of the structural configuration for this bridge and for a definition of the impact condition. For the Blountstown Bridge—formally known as the New Trammell Bridge—high-energy case (BLT-CHA-H), barge impact occurs on the channel pier near the top of a 30.5 ft tall shear wall that connects two 9 ft diameter drilled shafts (Figure 4-13). Two circular pier columns (5.5 ft diameter), which are axially collinear with each foundation shaft, span from the foundation elements to the top of the pier.
Prior to constructing the SBIA load cases, the barge impact force must be computed. For this bridge, impact occurs near the top of the shear wall, which has a 9-ft diameter round impact surface ($w_p = 9 \text{ ft}$). Thus, the barge yield force is calculated as:

$$P_{BY} = 1500 + 30 \cdot w_p = 1500 + 30 \cdot (9) = 1770 \text{ kips}$$

(4-18)

According to Consolazio et al. (2009a), this yield force is expected to occur at a barge crush depth ($a_{BY}$) of 2 in.

With the yield force quantified, the impact force corresponding to the high-energy barge collision (5920-ton flotilla, traveling at 5.0 knots) is computed. First, the series stiffness of the barge and pier/soil system ($k_s$) is calculated per Eqn. 4-7. Note that the stiffness of the pier/soil system for this bridge ($k_p$) is 963 kip/in.

$$k_s = \left( \frac{a_{BY}}{P_{BY}} + \frac{1}{k_p} \right)^{-1} = \left( \frac{2}{1770} + \frac{1}{963} \right)^{-1} = 461 \text{ kip/in.}$$

(4-19)

Thus, the high-energy crush force is computed given the barge tow velocity ($v_{Bi}$) of 5.0 knots (101 in/s) and mass ($m_B$) of 5920 tons (30.7 kip/in/s²):

$$P_B = v_{Bi} \cdot \sqrt{k_s \cdot m_B} = (101) \cdot \sqrt{(461) \cdot (30.7)} = 12,015 \text{ kips}$$

$$12,015 \text{ kips} > P_{BY}$$

$$\therefore P_B = P_{BY} = 1,770 \text{ kips}$$

(4-20)

This calculation illustrates that the initial kinetic energy of the barge tow is more than sufficient to yield the barge bow, generating the maximum crush force for this pier (1770 kips).

With the barge impact load ($P_B$) quantified, the two SBIA load cases are constructed. For Load Case 1, the amplified static impact load is computed:

$$1.45 \cdot P_B = 1.45 \cdot (1770) = 2567 \text{ kips}$$

(4-21)

This amplified impact load is used for each part of Load Case 1, regardless of the demand type of interest. However, unique pier-top loads are computed, corresponding to pier moment, pier...
shear, and total bearing shear (recall Figure 4-12). To quantify these loads, corresponding IRFs are calculated, based on bridge structural parameters. For this bridge, the height of the pier (h_p) is 37 ft, the lateral superstructure stiffness (k_{sup}) is 199 kip/in, and the total weight of the pier (W_p) is 1815 kips. Thus,

\[
\text{IRF}_m = 0.22 + \frac{4.5}{h_p} \sqrt{\frac{k_{sup}}{W_p}} = 0.22 + \frac{4.5}{37} \sqrt{\frac{199}{1815}} = 0.48
\]  
\[
\text{IRF}_v = 0.36 + \frac{3.0}{h_p} \sqrt{\frac{k_{sup}}{W_p}} = 0.36 + \frac{3.0}{37} \sqrt{\frac{199}{1815}} = 0.54
\]  
\[
\text{IRF}_b = 0.37 + \frac{7.0}{h_p} \sqrt{\frac{k_{sup}}{W_p}} = 0.37 + \frac{7.0}{37} \sqrt{\frac{199}{1815}} = 0.78
\]  

Lastly, the amplified impact load for Load Case 2 (Figure 4-12) is calculated as:

\[
1.85 \cdot P_B = 1.85 \cdot (1770) = 3275 \text{ kips}
\]  

With the SBIA load factors formed, two approaches to statically analyzing bridge response to barge impact loading are now considered.

**SBIA demonstration with OPTS superstructure model**

The SBIA method is first demonstrated using a one-pier, two-span (OPTS) bridge representation (recall Figure 4-13). Load Case 1 is analyzed by conducting three separate static analyses (Figure 4-14). The amplified impact load (as computed in Eqn. 4-21) is applied at the impact location for all three analyses. For each of the three analyses, the corresponding IRF (as calculated in Eqns. 4-22, 4-23, and 4-24) is multiplied by the impact force (P_B), and applied at the superstructure center of gravity, in the opposite direction of impact (Figure 4-14).

Having formed the complete loading condition for Load Case 1, the structure is statically analyzed. Predictions of pier member (column and foundation) moment, pier member (column
and foundation) shear, and total bearing shear demands are quantified using the respective analyses (shown in Figure 4-14). Note that beam-column axial forces are obtained from the Load Case 1 moment analysis (Figure 4-14a), for use in load-moment interaction calculations. SBIA Load Case 2 is also analyzed as shown in Figure 4-15. From this analysis, all pertinent member forces are quantified—pier member (column and foundation) moments, pier member (column and foundation) shears, and total bearing shear.

Design forces predicted by Load Cases 1 and 2 are summarized in Table 4-3. For each demand type, the maximum is selected for design. In this example, Load Case 1 controlled pier column and bearing design forces, while Load Case 2 controlled foundation design forces. This pattern is typical of the SBIA procedure; however, it is possible, given specific pier configurations and loading conditions, for either load case to dominate a given demand. Thus, the maximum demand predicted between both load cases must be considered for design.

The SBIA results for the Blountstown Bridge are additionally compared to dynamic predictions of design forces. Specifically, a corresponding dynamic analysis was conducted using the coupled vessel impact analysis (CVIA) method. The bridge was analyzed considering identical impact conditions—a 5920-ton barge tow, impacting the pier at 5.0 knots. Design forces obtained from CVIA are compared to SBIA predictions in Table 4-4.

The SBIA method conservatively predicts all relevant design forces, when compared to CVIA. For this example, SBIA predictions of pier column and bearing shear forces are within 2 to 8% of corresponding dynamic forces. The disparity is larger for foundation forces, at 27 to 35%. However, this level of conservatism is deemed acceptable, given the relative simplicity of the analysis procedure.
SBIA demonstration with spring superstructure model

In addition to demonstrating the SBIA method using an OPTS bridge model, the procedure and results are assessed using a more simplistic spring superstructure model. Specifically, the superstructure is removed and replaced by an equivalent 199 kip/in lateral spring. This modeling approach is common in bridge design practice; thus, compatibility is assessed between OPTS and spring-based superstructure modeling.

As before, SBIA loads are calculated per Eqns. 4-21, 4-22, 4-23, and 4-24. While load magnitudes are identical to those applied to the OPTS model, placement of the pier-top load differs. Using an OPTS model, this load is applied at the superstructure center of gravity. However, without adding additional connecting elements and increasing model complexity, this load-application location is not available in the simplified numerical model. Instead, the pier-top load is applied at the pier cap beam center of gravity (Figure 4-16). Load Case 2 involves only replacing the OPTS superstructure with an equivalent spring. The amplified barge load is then applied at the impact location (Figure 4-17).

As before, maximum demands obtained between the two load cases are selected as design forces (Table 4-5). Again, Load Case 1 controlled pier and bearing demands, while Load Case 2 controlled foundation demands.

Static (SBIA) and dynamic (CVIA) demand predictions are compared in Table 4-6. Using an equivalent spring superstructure model, SBIA column and bearing forces differ from CVIA by 2 to 9%, while foundation forces differ by 30 to 35%.

Comparison of SBIA using OPTS vs. spring superstructure model

Predictions of static (SBIA) design forces are compared in Table 4-7 for both OPTS and spring superstructure models. Observed discrepancies are small—less than 4%—across all demand types. Additionally, forces predicted by the spring superstructure model were always
conservative relative to the OPTS model, though this finding is specific to the demonstration case. In the next section, it will be shown that it is possible for spring superstructure models to produce lower force predictions than OTPS models. In general, the small relative difference between OPTS and spring-based design forces confirms the compatibility of either superstructure modeling technique for use with the SBIA procedure.

In addition to quantifying maximum forces, deflected pier shapes and structural demand profiles (shear and moment) are compared in Figure 4-18. Displacement and force profiles are shown for CVIA (dynamic analysis), SBIA using an OPTS model, and SBIA using a spring superstructure model. Note that the SBIA displacement profiles shown in Figure 4-18 were obtained from the Load Case 1 analysis calibrated to pier moment (recall Figure 4-14 and Figure 4-16).

The displacement profile for the Blountstown Bridge is representative of many of the cases studied in that the overall shape of the profile matches well with CVIA, but the pier displacements are smaller in magnitude, and sometimes negative. Negative pier displacements indicate that the net static loading pushed portions of the structure in the opposite direction of impact (typically near the pier-top). Negative displacements are not indicative of dynamic inertial restraint behavior, making this prediction somewhat unrealistic.

These unrealistic deflections are primarily a consequence of the conservative nature of SBIA. As previously discussed, this tendency is somewhat mitigated by scaling the impact load by 1.45. Using a scaled impact load, the ideal inertial resistance static loading conditions did not produce negative pier displacements. However, when using the conservative 99% upper bound inertial resistance loads (recall Figure 4-8), negative pier displacements occur in ten of the twenty pier/energy cases considered (see Appendix A for displacement profiles for all cases).
With regard to pier forces, SBIA profiles match well with CVIA (Figure 4-18). In this case, the overall shape of the profiles is consistent with CVIA, though SBIA generally predicts larger magnitudes—as is desired. Furthermore, in this case, all maximum SBIA pier demands occur in the same physical locations as predicted dynamically. However, this is not universal for all bridges (see Appendix A) therefore it is recommended that the maximum SBIA force predictions be used to design an entire member (column or foundation) as opposed to using the detailed shear or moment profile.

**Parametric Study Results**

In addition to the demonstration case described in the previous section, the SBIA method was assessed for a broad range of structural configurations and barge impact conditions by means of a parametric study. Specifically, the SBIA procedure was used to quantify pier design forces—pier column moments and shears, foundation moments and shears, and total bearing shear forces—for twenty (20) pier/impact energy conditions. As discussed previously, SBIA was additionally assessed using both OPTS and spring superstructure models.

In the results summary plots (Figure 4-19 through Figure 4-23), recall that each pier/energy case is denoted by a three part identifier. The first portion is an abbreviation of the bridge name (e.g. ACS refers to the Acosta Bridge). The second portion refers to the impacted pier location—CHA for a channel pier, and OFF for a pier located away from the navigation channel. The third portion refers to the impact energy condition—L, M, H, and S for low, moderate, high, and severe impact energy, respectively. See Chapter 3 for additional details regarding the bridges and energy conditions considered.

For comparison, corresponding fully dynamic simulations were conducted using the CVIA method, from which dynamic design force predictions were developed. To assess the suitability of SBIA for conservatively estimating dynamic design forces, SBIA demands are normalized by
corresponding CVIA demands to form demand ratios. If a given demand ratio is equal to 1.0, this indicates that the SBIA demand exactly matches that obtained from CVIA. Ratios greater than 1.0 imply that SBIA is conservative compared to CVIA, and ratios less than 1.0 imply that SBIA is unconservative.

Based on the data presented Figure 4-19 through Figure 4-23, it is observed that SBIA is always conservative relative to CVIA for the cases studied. This is a natural consequence of utilizing upper bound envelopes (99% confidence) for the two SBIA load cases. Not only is SBIA conservative relative to CVIA, the overall level of conservatism is reasonable—with mean values ranging from 1.3 to 1.6, depending on the demand type. Furthermore, nearly every relative demand is less than 2.0. Lastly, Figure 4-19 through Figure 4-23 illustrate that using a spring-based superstructure model is compatible with SBIA. Specifically, observed discrepancies between OPTS and spring demands are typically small—ranging from 0.01% to 36%, with an average of 7.0%. In general, the SBIA method provides adequate estimates of dynamic design forces across a wide spectrum of bridge configurations and impact conditions.
Table 4-3. SBIA demand prediction summary (OPTS superstructure).

<table>
<thead>
<tr>
<th></th>
<th>Load Case 1</th>
<th>Calibrated to pier moment</th>
<th>Calibrated to pier shear</th>
<th>Calibrated to bearing shear</th>
<th>Load Case 2</th>
<th>Maximum</th>
</tr>
</thead>
<tbody>
<tr>
<td>Column moment (kip-ft)</td>
<td>7,826</td>
<td>--</td>
<td>--</td>
<td>5,559</td>
<td>7,826</td>
<td></td>
</tr>
<tr>
<td>Column shear (kips)</td>
<td>411</td>
<td>--</td>
<td>--</td>
<td>345</td>
<td>411</td>
<td></td>
</tr>
<tr>
<td>Foundation moment (kip-ft)</td>
<td>11,310</td>
<td>--</td>
<td>--</td>
<td>22,324</td>
<td>22,324</td>
<td></td>
</tr>
<tr>
<td>Foundation shear (kips)</td>
<td>904</td>
<td>--</td>
<td>--</td>
<td>1,638</td>
<td>1,638</td>
<td></td>
</tr>
<tr>
<td>Total bearing shear (kips)</td>
<td>--</td>
<td>--</td>
<td>891</td>
<td>640</td>
<td>891</td>
<td></td>
</tr>
</tbody>
</table>

Table 4-4. Comparison of demand predictions—CVIA vs. SBIA (OPTS superstructure).

<table>
<thead>
<tr>
<th></th>
<th>Dynamic (CVIA)</th>
<th>Equiv. static (SBIA)</th>
<th>Percent difference</th>
</tr>
</thead>
<tbody>
<tr>
<td>Column moment (kip-ft)</td>
<td>7,281</td>
<td>7,826</td>
<td>7.5%</td>
</tr>
<tr>
<td>Column shear (kips)</td>
<td>404</td>
<td>411</td>
<td>1.7%</td>
</tr>
<tr>
<td>Foundation moment (kip-ft)</td>
<td>17,569</td>
<td>22,324</td>
<td>27.1%</td>
</tr>
<tr>
<td>Foundation shear (kips)</td>
<td>1,216</td>
<td>1,638</td>
<td>34.7%</td>
</tr>
<tr>
<td>Total bearing shear (kips)</td>
<td>883</td>
<td>891</td>
<td>0.9%</td>
</tr>
</tbody>
</table>

Table 4-5. SBIA demand prediction summary (spring superstructure).

<table>
<thead>
<tr>
<th></th>
<th>Load Case 1</th>
<th>Calibrated to pier moment</th>
<th>Calibrated to pier shear</th>
<th>Calibrated to bearing shear</th>
<th>Load Case 2</th>
<th>Maximum</th>
</tr>
</thead>
<tbody>
<tr>
<td>Column moment (kip-ft)</td>
<td>7,934</td>
<td>--</td>
<td>--</td>
<td>6,688</td>
<td>7,934</td>
<td></td>
</tr>
<tr>
<td>Column shear (kips)</td>
<td>413</td>
<td>--</td>
<td>--</td>
<td>343</td>
<td>413</td>
<td></td>
</tr>
<tr>
<td>Foundation moment (kip-ft)</td>
<td>11,596</td>
<td>--</td>
<td>--</td>
<td>22,753</td>
<td>22,753</td>
<td></td>
</tr>
<tr>
<td>Foundation shear (kips)</td>
<td>886</td>
<td>--</td>
<td>--</td>
<td>1,644</td>
<td>1,644</td>
<td></td>
</tr>
<tr>
<td>Total bearing shear (kips)</td>
<td>--</td>
<td>--</td>
<td>922</td>
<td>657</td>
<td>922</td>
<td></td>
</tr>
</tbody>
</table>

Table 4-6. Comparison of demand predictions—CVIA vs. SBIA (spring superstructure).

<table>
<thead>
<tr>
<th></th>
<th>Dynamic (CVIA)</th>
<th>Equiv. static (SBIA)</th>
<th>Percent difference</th>
</tr>
</thead>
<tbody>
<tr>
<td>Column moment (kip-ft)</td>
<td>7,281</td>
<td>7,934</td>
<td>9.0%</td>
</tr>
<tr>
<td>Column shear (kips)</td>
<td>404</td>
<td>413</td>
<td>2.2%</td>
</tr>
<tr>
<td>Foundation moment (kip-ft)</td>
<td>17,569</td>
<td>22,753</td>
<td>29.5%</td>
</tr>
<tr>
<td>Foundation shear (kips)</td>
<td>1,216</td>
<td>1,644</td>
<td>35.2%</td>
</tr>
<tr>
<td>Total bearing shear (kips)</td>
<td>883</td>
<td>922</td>
<td>4.4%</td>
</tr>
</tbody>
</table>
Table 4-7. Comparison of SBIA demand predictions—OPTS vs. spring superstructure.

<table>
<thead>
<tr>
<th></th>
<th>SBIA (OPTS)</th>
<th>SBIA (spring)</th>
<th>Percent difference</th>
</tr>
</thead>
<tbody>
<tr>
<td>Column moment (kip-ft)</td>
<td>7,826</td>
<td>7,934</td>
<td>1.4%</td>
</tr>
<tr>
<td>Column shear (kips)</td>
<td>411</td>
<td>413</td>
<td>0.5%</td>
</tr>
<tr>
<td>Foundation moment (kip-ft)</td>
<td>22,324</td>
<td>22,753</td>
<td>1.9%</td>
</tr>
<tr>
<td>Foundation shear (kips)</td>
<td>1,638</td>
<td>1,644</td>
<td>0.4%</td>
</tr>
<tr>
<td>Total bearing shear (kips)</td>
<td>891</td>
<td>922</td>
<td>3.5%</td>
</tr>
</tbody>
</table>

Figure 4-1. Static barge impact analysis. A) using existing methods (AASHTO 1991) and B) accounting for superstructure inertial resistance.
Figure 4-2. Superstructure modeling techniques considered. A) OPTS superstructure model and B) spring superstructure model.
Figure 4-3. Barge bow force-deformation relationship.

Figure 4-4. Inelastic barge bow deformation energy. A) loading and B) unloading.
Figure 4-5. Statically approximating superstructure inertial resistance by: A) restraining the pier top with boundary conditions; B) amplifying the superstructure stiffness and C) directly applying an inertial load.
Figure 4-6. Impact load magnified by a factor of 1.45.

Figure 4-7. Determination of ideal pier-top load and IRF for a given bridge pier (calibrated to column moment).
Figure 4-8. Correlation between inertial resistance factor (IRF) and bridge parameters. Calibrated to A) pier moment, B) pier shear and C) total bearing shear.
To obtain pier moments:

\[
\text{IRF}_m = 0.22 + 4.5 \frac{k_{\text{sup}}}{h_p} \sqrt{\frac{W_p}{W_p}}
\]

To obtain pier shears:

\[
\text{IRF}_v = 0.36 + 3.0 \frac{k_{\text{sup}}}{h_p} \sqrt{\frac{W_p}{W_p}}
\]

To obtain bearing shears:

\[
\text{IRF}_b = 0.37 + 7.0 \frac{k_{\text{sup}}}{h_p} \sqrt{\frac{W_p}{W_p}}
\]

Figure 4-9. Static loading to approximate superstructure dynamic amplification.

Given: \( M_{\text{max}}^{\text{dyn}} \) (maximum pile moment from dynamic analysis)

Iterate: \( P_{\text{Bamp}} \)

when \( M_{\text{static}}^{\text{max}} = M_{\text{dyn}}^{\text{max}} \Rightarrow P_{\text{Bamp}} = P_{\text{Bamp}}^{\text{ideal}} \)

thus \( \text{DMF}_{\text{ideal}} = \frac{P_{\text{Bamp}}^{\text{ideal}}}{P_B} \)

Figure 4-10. Determination of ideal amplified impact load and DMF for a bridge pier (calibrated to pile moment).
Figure 4-11. Mean value and envelope of impact-point DMF.
Figure 4-12. Static bracketed impact analysis (SBIA) method.
Figure 4-13. Structural configuration for Blountstown Bridge channel pier (BLT-CHA).

Figure 4-14. Loading conditions and maximum demand predictions for Load Case 1 with OPTS superstructure model. Calibrated to A) pier moment, B) pier shear and C) total bearing shear.
Figure 4-15. Loading conditions and maximum demand predictions for Load Case 2 with OPTS superstructure model.

Figure 4-16. Loading conditions and maximum demand predictions for Load Case 1 with spring superstructure model. Calibrated to A) pier moment, B) pier shear and C) total bearing shear.
Figure 4-17. Loading conditions and maximum demand predictions for Load Case 2 with spring superstructure model.
Figure 4-18. Comparison of CVIA and SBIA bridge responses for New Trammel Bridge at Blountstown (BLT-CHA). A) pier column profiles and B) foundation profiles.
Figure 4-19. SBIA pier column moment demands relative to CVIA.

Figure 4-20. SBIA pier column shear demands relative to CVIA.
Figure 4-21. SBIA foundation moment demands relative to CVIA.

Figure 4-22. SBIA foundation shear demands relative to CVIA.
Figure 4-23. SBIA total bearing shear demands relative to CVIA.
CHAPTER 5
CONCLUSIONS AND RECOMMENDATIONS

Summary and Conclusions

Previously conducted experimental and analytical research has highlighted the importance of dynamic effects that are present during barge-bridge collision events. Specifically, these studies identified superstructure inertia as a critical component of dynamic bridge pier behavior. Furthermore, current static analysis methods do not account for dynamic amplification of pier forces generated by inertial effects. Thus, dynamic analytical methods were developed, as part of prior research, which directly consider the effects of bridge mass distribution and bridge motions developed during barge impact.

While time-domain dynamic analysis procedures—notably the coupled vessel impact analysis (CVIA) procedure—provide accurate predictions of bridge response to barge collision loads, the computational requirements of such methods can be prohibitive in some design situations. In addition, the structural characteristics of a bridge in preliminary design may not be sufficiently well-defined to permit a detailed dynamic analysis. Thus, a need was identified to develop an equivalent static analysis procedure that simply and conservatively accounts for dynamic amplifications present during barge impact events. The research presented in this thesis has been carried out to address this need.

An equivalent static analysis procedure was developed that statically emulates superstructure inertial restraint. The proposed static bracketed impact analysis (SBIA) method consists of two static load cases. The first load case addresses inertial resistance by means of a static inertial load applied at the superstructure elevation, in addition to the barge impact load. This inertial load is quantified using empirical expressions that relate inertial force to readily available structural parameters. The second load case excludes the inertial load, and provides a
reliable means of quantifying maximum foundation forces. By bracketing pier forces between both load cases, reasonably conservative estimates of pier demands are assessed, including dynamic amplifications.

**Recommendations**

- **Consideration of superstructure inertial effects:**
  It is recommended that superstructure inertia be considered as part of the bridge design process for barge collision. Superstructure inertia can greatly amplify pier forces, and such amplifications are not considered in existing static analysis procedures. Dynamically amplified design forces should be quantified using either the time-domain CVIA method or the newly developed equivalent static SBIA method.

- **Use of static bracketed impact analysis (SBIA) procedure:**
  For preliminary design, or in cases where time-domain dynamic analysis is not warranted, the SBIA procedure is recommended to statically approximate dynamic amplification effects. This empirical method provides a conservative but simple static approach to impact-resistant bridge design. This thesis demonstrates that SBIA predicts conservative estimates of dynamic pier forces for a wide range of structural configurations and impact conditions.

- **Use of maximum member design forces from SBIA:**
  To maximize the simplicity of the SBIA method, certain limitations in terms of accuracy had to be imposed. While the static load cases employed by SBIA are able to conservatively bracket maximum design forces arising in major structural bridge components (pier columns, piles, drilled shafts), the detailed vertical profiles of structural demand (moment, shear) produced by the SBIA load cases are not necessarily conservative at every elevation. Hence, it is recommended that each major bridge component be designed with uniform capacity along its entire length such that every section of the component possesses sufficient capacity to resist the maximum forces predicted by the SBIA method.
APPENDIX A
COMPARISON OF SBIA AND CVIA RESULTS

In this appendix, SBIA (equivalent static) analysis results are compared to corresponding CVIA (dynamic) simulation results. For each bridge in the parametric study, three analyses were conducted: 1) Fully dynamic CVIA, 2) SBIA using one-pier, two-span (OPTS) models, and 3) SBIA using spring-based superstructure models. The data presented in this appendix form the basis for the relative demand ratios presented in Figures 4-19 through 4-23.

Figures presented in this appendix show the maximum magnitudes of displacement, shear force, and moment at each vertical elevation, for all three analyses. Elevation data have been adjusted such that the elevation datum (0 ft.) corresponds to the midplane elevation of the pile cap. In each figure, a gray rectangle is used to represent the pile cap vertical thickness. Each figure represents one impact energy condition applied to a respective bridge-pier configuration.
Figure A-1. Comparison of CVIA, SBIA-OPTS, and SBIA-spring results. Bridge: Acosta Bridge (ACS) channel pier. Impact condition: 2030 tons at 2.5 knots.

* SBIA displacement profiles shown were obtained using SBIA Load Case 1 with the moment IRF equation (Equation 5.15)
SBIA displacement profiles shown were obtained using SBIA Load Case 1 with the moment IRF equation (Equation 5.15).

Figure A-2. Comparison of CVIA, SBIA-OPTS, and SBIA-spring results. Bridge: Acosta Bridge (ACS) channel pier. Impact condition: 5920 tons at 5.0 knots.

### Peak Pier Demands

<table>
<thead>
<tr>
<th></th>
<th>CVIA</th>
<th>SBIA OPTS</th>
<th>SBIA SPRING</th>
</tr>
</thead>
<tbody>
<tr>
<td>Column moment:</td>
<td>63873 kip-ft</td>
<td>97201 kip-ft</td>
<td>83425 kip-ft</td>
</tr>
<tr>
<td>Column shear:</td>
<td>1145 kips</td>
<td>2146 kips</td>
<td>2202 kips</td>
</tr>
<tr>
<td>Pile Moment:</td>
<td>2797 kip-ft</td>
<td>4110 kip-ft</td>
<td>4018 kip-ft</td>
</tr>
<tr>
<td>Pile Shear:</td>
<td>272 kips</td>
<td>385 kips</td>
<td>369 kips</td>
</tr>
<tr>
<td>Total Bearing Shear:</td>
<td>1223 kips</td>
<td>2841 kips</td>
<td>2925 kips</td>
</tr>
</tbody>
</table>
Impact condition: 7820 tons at 5.0 knots.

Figure A-3. Comparison of CVIA, SBIA-OPTS, and SBIA-spring results. Bridge: Acosta Bridge (ACS) channel pier.
Peak Pier Demands

**Column moment:**
- CVIA: 7165 kip-ft
- SBIA OPTS: 7826 kip-ft
- SBIA SPRING: 7934 kip-ft

**Column shear:**
- CVIA: 394 kips
- SBIA OPTS: 411 kips
- SBIA SPRING: 413 kips

**Pile Moment:**
- CVIA: 14124 kip-ft
- SBIA OPTS: 22324 kip-ft
- SBIA SPRING: 22753 kip-ft

**Pile Shear:**
- CVIA: 1123 kips
- SBIA OPTS: 1638 kips
- SBIA SPRING: 1644 kips

**Total Bearing Shear:**
- CVIA: 849 kips
- SBIA OPTS: 891 kips
- SBIA SPRING: 922 kips

Impact condition: 2030 tons at 2.5 knots.
Impact condition: 5920 tons at 5.0 knots.  

Figure A-5. Comparison of CVIA, SBIA-OPTS, and SBIA-spring results. Bridge: SR-20 at Blountstown (BLT) channel pier.  

<table>
<thead>
<tr>
<th>Peak Pier Demands</th>
<th>CVIA</th>
<th>SBIA OPTS</th>
<th>SBIA SPRING</th>
</tr>
</thead>
<tbody>
<tr>
<td>Column moment:</td>
<td>7281 kip-ft</td>
<td>7826 kip-ft</td>
<td>7934 kip-ft</td>
</tr>
<tr>
<td>Column shear:</td>
<td>404 kips</td>
<td>411 kips</td>
<td>413 kips</td>
</tr>
<tr>
<td>Pile Moment:</td>
<td>17569 kip-ft</td>
<td>22324 kip-ft</td>
<td>22753 kip-ft</td>
</tr>
<tr>
<td>Pile Shear:</td>
<td>1216 kips</td>
<td>1638 kips</td>
<td>1644 kips</td>
</tr>
<tr>
<td>Total Bearing Shear:</td>
<td>883 kips</td>
<td>892 kips</td>
<td>922 kips</td>
</tr>
</tbody>
</table>

* SBIA displacement profiles shown were obtained using SBIA Load Case 1 with the moment IRF equation (Equation 5.15)
Peak Pier Demands

<table>
<thead>
<tr>
<th>Demand</th>
<th>CVIA</th>
<th>SBIA OPTS</th>
<th>SBIA SPRING</th>
</tr>
</thead>
<tbody>
<tr>
<td>Column moment</td>
<td>▼ 2589 kip-ft</td>
<td>▼ 2940 kip-ft</td>
<td>▼ 3057 kip-ft</td>
</tr>
<tr>
<td>Column shear</td>
<td>■ 106 kips</td>
<td>□ 113 kips</td>
<td>□ 117 kips</td>
</tr>
<tr>
<td>Pile Moment</td>
<td>◆ 106 kip-ft</td>
<td>◆ 158 kip-ft</td>
<td>◆ 156 kip-ft</td>
</tr>
<tr>
<td>Pile Shear</td>
<td>◇ 79 kips</td>
<td>◇ 91 kips</td>
<td>◇ 90 kips</td>
</tr>
<tr>
<td>Total Bearing Shear</td>
<td>438 kips</td>
<td>461 kips</td>
<td>480 kips</td>
</tr>
</tbody>
</table>

* SBIA displacement profiles shown were obtained using SBIA Load Case 1 with the moment IRF equation (Equation 5.15)

Figure A-6. Comparison of CVIA, SBIA-OPTS, and SBIA-spring results. Bridge: Eau Gallie Bridge (EGB) channel pier. Impact condition: 2030 tons at 2.5 knots.
**Peak Pier Demands**

<table>
<thead>
<tr>
<th></th>
<th>CVIA</th>
<th>SBIA OPTS</th>
<th>SBIA SPRING</th>
</tr>
</thead>
<tbody>
<tr>
<td>Column moment:</td>
<td>3331 kip-ft</td>
<td>3722 kip-ft</td>
<td>3825 kip-ft</td>
</tr>
<tr>
<td>Column shear:</td>
<td>203 kips</td>
<td>266 kips</td>
<td>293 kips</td>
</tr>
<tr>
<td>Pile Moment:</td>
<td>123 kip-ft</td>
<td>158 kip-ft</td>
<td>159 kip-ft</td>
</tr>
<tr>
<td>Pile Shear:</td>
<td>75 kips</td>
<td>90 kips</td>
<td>90 kips</td>
</tr>
<tr>
<td>Total Bearing Shear:</td>
<td>410 kips</td>
<td>459 kips</td>
<td>505 kips</td>
</tr>
</tbody>
</table>

*SBIA displacement profiles shown were obtained using SBIA Load Case 1 with the moment IRF equation (Equation 5.15)*

Figure A-7. Comparison of CVIA, SBIA-OPTS, and SBIA-spring results. Bridge: Melbourne Causeway (MBC) channel pier. Impact condition: 2030 tons at 2.5 knots.
Figure A-8. Comparison of CVIA, SBIA-OPTS, and SBIA-spring results. Bridge: new St. George Island (NSG) channel pier. Impact condition: 2030 tons at 2.5 knots.

* SBIA displacement profiles shown were obtained using SBIA Load Case 1 with the moment IRF equation (Equation 5.15)
Figure A-9. Comparison of CVIA, SBIA-OPTS, and SBIA-spring results. Bridge: new St. George Island (NSG) channel pier. Impact condition: 5920 tons at 5.0 knots.
**Peak Pier Demands**

<table>
<thead>
<tr>
<th></th>
<th>CVIA</th>
<th>SBIA OPTS</th>
<th>SBIA SPRING</th>
</tr>
</thead>
<tbody>
<tr>
<td>Column moment:</td>
<td>✓ 7233 kip-ft</td>
<td>✓ 8312 kip-ft</td>
<td>✓ 8308 kip-ft</td>
</tr>
<tr>
<td>Column shear:</td>
<td>□ 644 kips</td>
<td>□ 742 kips</td>
<td>□ 821 kips</td>
</tr>
<tr>
<td>Pile Moment:</td>
<td>● 3076 kip-ft</td>
<td>○ 3775 kip-ft</td>
<td>● 3724 kip-ft</td>
</tr>
<tr>
<td>Pile Shear:</td>
<td>▲ 384 kips</td>
<td>○ 459 kips</td>
<td>○ 454 kips</td>
</tr>
<tr>
<td>Total Bearing Shear:</td>
<td>1180 kips</td>
<td>1376 kips</td>
<td>1400 kips</td>
</tr>
</tbody>
</table>

* SBIA displacement profiles shown were obtained using SBIA Load Case 1 with the moment IRF equation (Equation 5.15)

Figure A-10. Comparison of CVIA, SBIA-OPTS, and SBIA-spring results. Bridge: new St. George Island (NSG) channel pier. Impact condition: 7820 tons at 7.5 knots.
Figure A-11. Comparison of CVIA, SBIA-OPTS, and SBIA-spring results. Bridge: new St. George Island (NSG) off-channel pier. Impact condition: 200 tons at 1.0 knots.

<table>
<thead>
<tr>
<th>Peak Pier Demands</th>
<th>CVIA Column moment: 1518 kip-ft</th>
<th>SBIA-OPTS Column moment: 2671 kip-ft</th>
<th>SBIA SPRING Column moment: 2898 kip-ft</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>CVIA Column shear: 61 kips</td>
<td>SBIA-OPTS Column shear: 95 kips</td>
<td>SBIA SPRING Column shear: 103 kips</td>
</tr>
<tr>
<td>Pile Moment:</td>
<td>CVIA 521 kip-ft</td>
<td>SBIA-OPTS 1060 kip-ft</td>
<td>SBIA SPRING 1039 kip-ft</td>
</tr>
<tr>
<td>Pile Shear:</td>
<td>CVIA 71 kips</td>
<td>SBIA-OPTS 141 kips</td>
<td>SBIA SPRING 140 kips</td>
</tr>
<tr>
<td>Total Bearing Shear</td>
<td>CVIA 128 kips</td>
<td>SBIA-OPTS 218 kips</td>
<td>SBIA SPRING 231 kips</td>
</tr>
</tbody>
</table>

* SBIA displacement profiles shown were obtained using SBIA Load Case 1 with the moment IRF equation (Equation 5.15)
Peak Pier Demands

<table>
<thead>
<tr>
<th></th>
<th>CVIA</th>
<th>SBIA OPTS</th>
<th>SBIA SPRING</th>
</tr>
</thead>
<tbody>
<tr>
<td>Column moment:</td>
<td>6710 kip-ft</td>
<td>8025 kip-ft</td>
<td>7689 kip-ft</td>
</tr>
<tr>
<td>Column shear:</td>
<td>405 kips</td>
<td>490 kips</td>
<td>500 kips</td>
</tr>
<tr>
<td>Pile Moment:</td>
<td>107 kip-ft</td>
<td>189 kip-ft</td>
<td>184 kip-ft</td>
</tr>
<tr>
<td>Pile Shear:</td>
<td>73 kips</td>
<td>88 kips</td>
<td>87 kips</td>
</tr>
<tr>
<td>Total Bearing Shear:</td>
<td>721 kips</td>
<td>860 kips</td>
<td>973 kips</td>
</tr>
</tbody>
</table>

SBIA displacement profiles shown were obtained using SBIA Load Case 1 with the moment IRF equation (Equation 5.15).

Figure A-12. Comparison of CVIA, SBIA-OPTS, and SBIA-spring results. Bridge: old St. George Island (OSG) channel pier. Impact condition: 2030 tons at 2.5 knots.
Figure A-13. Comparison of CVIA, SBIA-OPTS, and SBIA-spring results. Bridge: old St. George Island (OSG) off-channel pier. Impact condition: 200 tons at 1.0 knots.
Figure A-14. Comparison of CVIA, SBIA-OPTS, and SBIA-spring results. Bridge: Pineda Causeway (PNC) channel pier. Impact condition: 2030 tons at 2.5 knots.
Figure A-15. Comparison of CVIA, SBIA-OPTS, and SBIA-spring results. Bridge: Ringling (RNG) channel pier.
Impact condition: 2030 tons at 2.5 knots.

* SBIA displacement profiles shown were obtained using SBIA Load Case 1 with the moment IRF equation (Equation 5.15)
Impact condition: 5920 tons at 5.0 knots.

Figure A-16. Comparison of CVIA, SBIA-OPTS, and SBIA-spring results. Bridge: Ringling (RNG) channel pier.
Figure A-17. Comparison of CVIA, SBIA-OPTS, and SBIA-spring results. Bridge: Ringling (RNG) off-channel pier. Impact condition: 200 tons at 1.0 knots.

<table>
<thead>
<tr>
<th>Peak Pier Demands</th>
<th>CVIA</th>
<th>SBIA OPTS</th>
<th>SBIA SPRING</th>
</tr>
</thead>
<tbody>
<tr>
<td>Column moment:</td>
<td>12678 kip-ft</td>
<td>19346 kip-ft</td>
<td>16233 kip-ft</td>
</tr>
<tr>
<td>Column shear:</td>
<td>282 kips</td>
<td>412 kips</td>
<td>528 kips</td>
</tr>
<tr>
<td>Pile Moment:</td>
<td>7134 kip-ft</td>
<td>8476 kip-ft</td>
<td>7766 kip-ft</td>
</tr>
<tr>
<td>Pile Shear:</td>
<td>410 kips</td>
<td>526 kips</td>
<td>500 kips</td>
</tr>
<tr>
<td>Total Bearing Shear:</td>
<td>428 kips</td>
<td>573 kips</td>
<td>759 kips</td>
</tr>
</tbody>
</table>
SBIA displacement profiles shown were obtained using SBIA Load Case 1 with the moment IRF equation (Equation 5.15).

Figure A-18. Comparison of CVIA, SBIA-OPTS, and SBIA-spring results. Bridge: Ringling (RNG) off-channel pier. Impact condition: 2030 tons at 2.5 knots
Figure A-19. Comparison of CVIA, SBIA-OPTS, and SBIA-spring results. Bridge: Seabreeze (SBZ) channel pier. Impact condition: 2030 tons at 2.5 knots.

* SBIA displacement profiles shown were obtained using SBIA Load Case 1 with the moment IRF equation (Equation 5.15)
Impact condition: 5920 tons at 5.0 knots

Figure A-20. Comparison of CVIA, SBIA-OPTS, and SBIA-spring results. Bridge: Seabreeze (SBZ) channel pier.
APPENDIX B
DEMONSTRATION OF SBIA METHOD

Introduction

In this appendix, the SBIA method is demonstrated for the New Trammel Bridge, in northwestern Florida. For this example, a three-barge flotilla (5920 tons with tug) collides with the channel pier at 5.0 knots. Barge impact occurs near the top of a 30.5 ft tall shear wall that connects two 9 ft diameter drilled shafts (Figure B-1). Two circular pier columns (5.5 ft diameter), which are axially collinear with each foundation shafts, span from the foundation elements to the top of the pier.

Figure B-1. Structural configuration for New Trammel Bridge.

Demonstration of SBIA Procedure

Prior to constructing the SBIA load cases, the vessel impact force must be computed. For this bridge, impact occurs near the top of the shear wall, which has a 9-ft diameter round impact
surface (\( w_p = 9 \text{ ft} \)). Thus, the barge yield force is determined in accordance with Figure B-2 and Eqn. B-1. Note that this yield force occurs at a crush depth (\( a_{BY} \)) of 2 in.

![Figure B-2](image.png)

Figure B-2. Barge yield load determination for 9-ft round impact surface.

\[
P_{BY} = 1500 + 30 \cdot w_p = 1500 + 30 \cdot (9) = 1770 \text{kips} \quad (B-1)
\]

With the yield force quantified, the impact force corresponding to the high-energy barge collision (5920-ton flotilla, traveling at 5.0 knots) is computed. First, the series stiffness of the barge and pier/soil system (\( k_S \)) is calculated per Eqn. B-3. Note that the stiffness of the pier/soil system for this bridge (\( k_P \)) must be quantified as shown in

Figure B-3 and Eqn. B-2. For this example, \( P_{BY} \) is applied to the pier to quantify \( k_P \); however, if the calculated impact load (\( P_B \)) is found to be less than \( P_{BY} \), then this process should be repeated to obtain a more accurate estimate of \( k_P \).
Thus, the high-energy crush force is computed given the barge tow velocity \(v_{Bi}\) of 5.0 knots (101 in/s) and mass \(m_B\) of 5920 tons (30.7 kip/in/s):

\[
P_B = v_{Bi} \cdot \sqrt{k_s \cdot m_B} = (101) \cdot \sqrt{(461) \cdot (30.7)} = 12,015 \text{ kips}
\]

\[
12,015 \text{ kips} > P_{BY}
\]

\[
\therefore P_B = P_{BY} = 1,770 \text{ kips}
\]

This calculation illustrates that the incoming kinetic energy of the barge tow is sufficient to yield the barge bow, generating the maximum crush force for this pier (1770 kips).

**Load Case 1**

With the barge impact load \(P_B\) quantified, the SBIA load cases are constructed. For Load Case 1, the amplified static impact load is computed:

\[
1.45 \cdot P_B = 1.45 \cdot (1770) = 2567 \text{ kips}
\]
This amplified impact load is used for each part of Load Case 1, regardless of the demand type of interest. However, unique pier-top loads are computed, corresponding to pier moment, pier shear, and total bearing shear. To quantify these loads, corresponding IRFs are calculated, based on the bridge structural parameters illustrated in Figure B-4 and Figure B-5.

Figure B-4. Determination of pier weight (W_p) and pier height (h_p).

Remove impact pier and apply load equal to 0.25P_B at impact pier location

Measure superstructure deflection (Δ_sup) at impact pier location

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Figure B-5. Determination of superstructure stiffness ($k_{\text{sup}}$).

As illustrated in Figure B-4, the total weight of this pier ($W_p$) is 1815 kips, and the height of the pier ($h_p$) is 37 ft. The lateral superstructure stiffness ($k_{\text{sup}}$) is 199 kip/in, as determined using the process shown in Figure B-5. Due to the empirical nature of the IRF equations, the units shown above must be used. Thus,

\[
\text{IRF}_m = 0.22 + \frac{4.5}{h_p} \sqrt{\frac{k_{\text{sup}}}{W_p}} = 0.22 + \frac{4.5}{37} \sqrt{\frac{199}{1815}} = 0.48 \quad \text{(B-6)}
\]

\[
\text{IRF}_v = 0.36 + \frac{3.0}{h_p} \sqrt{\frac{k_{\text{sup}}}{W_p}} = 0.36 + \frac{3.0}{37} \sqrt{\frac{199}{1815}} = 0.54 \quad \text{(B-7)}
\]

\[
\text{IRF}_b = 0.37 + \frac{7.0}{h_p} \sqrt{\frac{k_{\text{sup}}}{W_p}} = 0.37 + \frac{7.0}{37} \sqrt{\frac{199}{1815}} = 0.78 \quad \text{(B-8)}
\]

The amplified impact load (as computed in Eqn. B-5) is applied at the impact location for all three analyses. For each of the three analyses, the corresponding IRF (as calculated in Eqns. B-6, B-7, and B-8) is multiplied by the impact force ($P_B$), and this load is applied at the pier cap beam center of gravity, in the opposite direction of impact (Figure B-6).
Figure B-6. Loading conditions and maximum demand predictions for Load Case 1.

With the loading conditions for Load Case 1 developed, the structure is statically analyzed. Predictions of pier (column and foundation) moment, pier (column and foundation) shear, and total bearing shear demands are quantified using the respective analyses (Figure B-6). These design forces are additionally summarized in Table B-1.

Load Case 2

SBIA Load Case 2 is also analyzed as shown in Figure B-7. From this single analysis, all pertinent member forces are quantified—pier moments, pier shears, and total bearing shear. These demands are compared to those obtained from Load Case 1 in Table B-1. The amplified impact load for Load Case 2 is calculated as:

\[ 1.85 \cdot P_B = 1.85 \cdot (1770) = 3275 \text{kips} \] (B-9)
Figure B-7. Loading conditions and maximum demand predictions for Load Case 2.

Results summary

Design forces predicted by Load Cases 1 and 2 are summarized in Table B-1. For each demand type, the maximum is selected for design. In this example, Load Case 1 controlled pier column and bearing design forces, while Load Case 2 controlled foundation design forces. This pattern is typical of the SBIA procedure; however, it is possible, given specific pier configurations and loading conditions, for either load case to dominate a given demand. Thus, the maximum demand predicted between both load cases must be considered for design.

Table B-1. SBIA demand prediction summary.

<table>
<thead>
<tr>
<th></th>
<th>Load Case 1</th>
<th>Load Case 2</th>
<th>Maximum</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Calibrated</td>
<td>Calibrated</td>
<td></td>
</tr>
<tr>
<td></td>
<td>to</td>
<td>to</td>
<td></td>
</tr>
<tr>
<td></td>
<td>pier moment</td>
<td>pier shear</td>
<td>bearing shear</td>
</tr>
<tr>
<td>Column moment (kip-ft)</td>
<td>7,934</td>
<td>--</td>
<td>--</td>
</tr>
<tr>
<td>Column shear (kips)</td>
<td>--</td>
<td>413</td>
<td>--</td>
</tr>
<tr>
<td>Foundation moment (kip-ft)</td>
<td>11,596</td>
<td>--</td>
<td>--</td>
</tr>
<tr>
<td>Foundation shear (kips)</td>
<td>--</td>
<td>886</td>
<td>--</td>
</tr>
<tr>
<td>Total bearing shear (kips)</td>
<td>--</td>
<td>--</td>
<td>922</td>
</tr>
</tbody>
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LIST OF REFERENCES


BIOGRAPHICAL SKETCH

The author was born in Ogden, Utah, in 1981. In September 2002, he began attending Daytona Beach Community College where he later earned an Associate of Arts degree in August 2005. Subsequently, he began attending the University of Florida, where he received the degree of Bachelor of Science in Civil Engineering in May 2008. The author enrolled in graduate school at the University of Florida in September 2008, where he anticipates receiving the degree of Master of Engineering. The author plans to continue work toward a Doctor of Philosophy degree at the University of Florida upon graduation.