March 2022

Assessment of Scoured Bridges Subjected to Vessel Impact Using Nonlinear Dynamic Analysis

Amir S. Irhayyim
University of South Florida

Follow this and additional works at: https://digitalcommons.usf.edu/etd

Part of the Civil Engineering Commons

Scholar Commons Citation

This Dissertation is brought to you for free and open access by the USF Graduate Theses and Dissertations at Digital Commons @ University of South Florida. It has been accepted for inclusion in USF Tampa Graduate Theses and Dissertations by an authorized administrator of Digital Commons @ University of South Florida. For more information, please contact scholarcommons@usf.edu.
Assessment of Scoured Bridges Subjected to Vessel Impact Using Nonlinear Dynamic Analysis

by

Amir S. Irhayyim

A dissertation submitted in partial fulfillment
of the requirements for the degree of
Doctor of Philosophy in Civil Engineering
Department of Civil and Environmental Engineering
College of Engineering
University of South Florida

Major Professor: Manjriker Gunaratne, Ph.D.
Rajan Sen, Ph.D.
Mark Ross, Ph.D.
Autar Kaw, Ph.D.
Kandethody Ramachandran, Ph.D.

Date of Approval:
March 10, 2022

Keywords: Ship impact, Nonlinear time history, Scour depth, Dual hazards, Soil structure interaction, Incremental dynamic analysis, Fragility curves

Copyright © 2022, Amir S. Irhayyim
Dedication

To my father, mother, my family members, and friends who were a big inspiration and support to my achievement. To all who encouraged me to make my dream come true especially Dr. Manjriker Gunaratne and Dr. Ameh Fioklou.

I would like to take this opportunity to thank those without whom this work would not have been possible. First and foremost, my sincere gratitude and appreciation to my advisor, Dr. Gunaratne for his academic support, mentorship, and guidance to carry out my research successfully.

I would like to thank Dr. Rajan Sen, Dr. Mark Ross, Dr. Autar Kaw, and Dr. Kandethody Ramachandran for serving on my committee. They have provided encouragement and guidance during this process.

Finally, special thanks go to all USF staff and faculty who have assisted me in completing this research. I am blessed by the immense support you have shown throughout my graduate studies.
# Table of Contents

List of Tables  iii

List of Figures  iv

Abstract  ix

Chapter 1: Introduction and Literature Review  1
  1.1 Background  1
  1.2 Ship Collision Impact Force  2
  1.3 Local Scour Depth  3
  1.4 Concept of Incremental Dynamic Analysis  4
  1.5 Collapse Mechanism of IDA  6
  1.6 Literature Review of Scour and Ship Impact  7
  1.7 Literature Review of IDA  8
  1.8 Literature Review of Fragility Analysis  10
  1.9 Problem Statement and Research Objective  11

Chapter 2: Assessment of Scoured Bridges Subjected to Ship Impact  13
  2.1 Methodology  13
  2.2 Geometry of the Bridge Used for the Presented Case Study  15
  2.3 Development of the Model  17
    2.3.1 Deck  17
    2.3.2 Cap Beam  18
    2.3.3 Column  19
    2.3.4 Pile Cap  20
    2.3.5 Piles  20
    2.3.6 Soil  21
    2.3.7 Abutment  23
  2.4 Analyses  26
    2.4.1 Pushover Analysis  26
    2.4.2 Modal Analysis  27
    2.4.3 Moment-Curvature Analysis  29
    2.4.4 Moment-Curvature Demand Hysteretic Diagram  33
  2.5 Results and Discussion  35
    2.5.1 Effect on Deck Center Displacement  36
    2.5.2 Effect on Base of Column  37
      2.5.2.1 Evaluation of Column Base Shear Force  37
      2.5.2.2 Evaluation of Column Base Moment  38
    2.5.3 Effect on Piles  38
List of Tables

Table 2.1 Modal analysis results of the bridge periods 32
Table 3.1 Bridge configurations 54
Table 3.2 Soil configurations 55
Table 3.3 Summary of materials and elements in OpenSees for bridge components 56
Table 3.4 Bridge column configurations of generalized IDA 96
## List of Figures

<table>
<thead>
<tr>
<th>Figure</th>
<th>Description</th>
<th>Page</th>
</tr>
</thead>
<tbody>
<tr>
<td>1.1</td>
<td>Elements associated with the ship impact on pier with a scoured foundation</td>
<td>4</td>
</tr>
<tr>
<td>1.2</td>
<td>Conceptual IDA curve</td>
<td>6</td>
</tr>
<tr>
<td>2.1</td>
<td>Time history of ship-impact force exerted by a 5000 DWT vessel at a 2 m/s velocity compared with the corresponding AASHTO specification</td>
<td>14</td>
</tr>
<tr>
<td>2.2</td>
<td>Geometry of the bridge used for the case study</td>
<td>16</td>
</tr>
<tr>
<td>2.3</td>
<td>Schematic diagram of 3D finite element model for the case study bridge</td>
<td>17</td>
</tr>
<tr>
<td>2.4</td>
<td>Elevation view of the geometry model of a three-span bridge</td>
<td>19</td>
</tr>
<tr>
<td>2.5</td>
<td>(a) Schematic diagram of displacement-based element pile model and zero-length element spring soil (Note: Equal DOF indicates compatible displacements of soil and pole nodes); (b) p-y curve for sand; (c) p-y curve for soft clay</td>
<td>22</td>
</tr>
<tr>
<td>2.6</td>
<td>Bridge seat-type abutment components</td>
<td>24</td>
</tr>
<tr>
<td>2.7</td>
<td>Stress-strain material models (a) Concrete01 (b) Concrete04 (c) Steel01 bearing pad material in longitudinal direction; (d) ENT bearing pad material in vertical direction; (e and f) in-parallel material modeling, strains are equal and stresses are additive; (g and h) in-series material modeling, strains are additive and stresses are equal; (i) tension gap; (j) compression gap; (k) impact material (l) impact force</td>
<td>25</td>
</tr>
<tr>
<td>2.8</td>
<td>Modeling of abutment components</td>
<td>26</td>
</tr>
<tr>
<td>2.9</td>
<td>Pushover displacement control at the middle of the bridge column</td>
<td>28</td>
</tr>
<tr>
<td>2.10</td>
<td>Mode shape for scour 0.0 m</td>
<td>31</td>
</tr>
<tr>
<td>2.11</td>
<td>Mode shape for scour 7.0 m</td>
<td>32</td>
</tr>
<tr>
<td>2.12</td>
<td>Fiber discretization for a column section</td>
<td>33</td>
</tr>
<tr>
<td>2.13</td>
<td>Moment curvature of the bridge column</td>
<td>34</td>
</tr>
</tbody>
</table>
Figure 2.14 Moment-curvature hysteretic diagram of the column

Figure 2.15 Deck center displacement due to various scour conditions (0 to 7 m) at different impact locations (3, 5 and 7 m)

Figure 2.16 Base shear of column under various scour conditions (0 to 7 m) at different impact locations (3, 5 and 7 m)

Figure 2.17 Base moment of column under various scour conditions (0 to 7 m) at different impact locations (3, 5, and 7 m)

Figure 2.18 Plan view of pile group under Column 1

Figure 2.19 Maximum absolute shear force of Pile 19 for various scour conditions (0 to 7 m) at different impact locations (3 to 7 m)

Figure 2.20 Maximum absolute moment of Pile 19 for various scour conditions (0 to 7 m) at different impact locations (3 to 7 m)

Figure 2.21 Maximum absolute displacement of Pile 1 for various scour conditions (0 to 7 m) at different impact locations (3 to 7 m)

Figure 2.22 Maximum absolute axial force of Pile 11 for various scour conditions (0 to 7 m) at different impact locations (3 to 7 m)

Figure 2.23 Soil responses of piles 11, 12 and 13 with a scour of 0 m at a ship impact of 3 m

Figure 2.24 Impact point nodal displacement due to ship height 3 m and scour 7 m with different soil configuration (config1 clay- sand, config 2 sand & config 3 clay)

Figure 2.25 Column base moment due to ship height 3 m with different soil configurations

Figure 2.26 Absolute maximum moment on Pile 19 due to ship impact height of 3 m with different soil configurations and pile diameters

Figure 2.27 Pile 1 displacement due to ship height 3 m at scour depth 5 m with different soil configurations

Figure 2.28 Comparison of central deck displacement due to ship impact with scour to that with no scour to column height/pile diameter ratio due to a ship impact at a height of 5 m with 3 m scour

Figure 2.29 Moment curvature fiber comparison for 1.6 m with column height of 10 m

Figure 2.30 Moment curvature comparison for 1.3, 1.6, 1.9 m diameter column with heights of 7.5, 10.0 and 12.5 m, respectively
Figure 3.1 Steel01 model in OpenSees for barge force displacement relationship

Figure 3.2 Sketch of finite element model for the barge-bridge column collision

Figure 3.3 IDA curve for central deck displacement for five scour cases with incremental barge velocities under barge impact points of (a) 3 m, (b) 4 m, and (c) 5 m above the pile cap

Figure 3.4 IDA curve for column impact point displacement for five scour cases with incremental barge velocities under barge impact points of (a) 3 m, (b) 4 m, and (c) 5 m above the pile cap

Figure 3.5 IDA curve for column impact moment for five scour cases with incremental barge velocities under barge impact points of (a) 3 m, (b) 4 m, and (c) 5 m above the pile cap

Figure 3.6 IDA curve for column impact shear for five scour cases with incremental barge velocities under barge impact points of (a) 3 m, (b) 4 m, and (c) 5 m above the pile cap

Figure 3.7 Traditional IDA curve for column impact point rotation for five scour cases with incremental barge velocities under barge impact points of (a) 3 m, (b) 4 m, and (c) 5 m above the pile cap

Figure 3.8 (a), (b), and (c) IDA curve for maximum absolute displacement of Pile 1

Figure 3.9 (a), (b), and (c) IDA curve for maximum absolute axial force in Pile 1

Figure 3.10 (a), (b), and (c) IDA curve for maximum absolute moment (DM) of Pile 1

Figure 3.11 (a), (b), and (c) IDA curve for maximum absolute shear of Pile 1

Figure 3.12 Fragility curves of drift ratio of bridge columns as functions of mass under (a) 0 m (b) 1 m (c) 2 m (d) and 3 m scour depths

Figure 3.13 Fragility curves of drift ratio of bridge columns as functions of barge velocity under (a) 0 m (b) 1 m (c) 2 m (d) and 3 m scour depths

Figure 3.14 IDA curves for displacement of deck with three soil configurations and bridge configuration 5 at an impact point height of 5 m, and scour depths of (a) 0 m, (b) 2 m, and (c) 4 m

Figure 3.15 IDA curves for displacement of bridge column with three soil configurations and bridge configuration 5 at an impact point height of 5 m, and scour depths of (a) 0 m, (b) 2 m, and (c) 4 m
Figure 3.16 IDA curves for impact point moment of column with three soil configurations and bridge configuration 5 at an impact point height of 5 m, and scour depths of (a) 0 m, (b) 2 m, and (c) 4 m

Figure 3.17 IDA curves for impact point shear of column with three soil configurations and bridge configuration 5 at an impact point height of 5 m, and scour depths of (a) 0 m, (b) 2 m, and (c) 4 m

Figure 3.18 IDA curves for impact point rotation of column with three soil configurations and bridge configuration 5 at an impact point height of 5 m, and scour depths of (a) 0 m, (b) 2 m, and (c) 4 m

Figure 3.19 IDA curves for displacement of Pile 1 with three soil configurations and bridge configuration 5 at an impact point height of 5 m, and scour depths of (a) 0 m, (b) 2 m, and (c) 4 m

Figure 3.20 IDA curves for axial force of Pile 1 with three soil configurations and bridge configuration 5 at an impact point height of 5 m, and scour depths of (a) 0 m, (b) 2 m, and (c) 4 m

Figure 3.21 IDA curves for moment of Pile 1 with three soil configurations and bridge configuration 5 at an impact point height of 5 m, and scour depths of (a) 0 m, (b) 2 m, and (c) 4 m

Figure 3.22 IDA curves for shear of Pile 1 with three soil configurations and bridge configuration 5 at an impact point height of 5 m, and scour depths of (a) 0 m, (b) 2 m, and (c) 4 m

Figure 3.23 Soil profile 1

Figure 3.24 IDA curves for displacement of deck with three bridge column configurations and soil configuration 1 at an impact point height of 3 m, and scour depths of (a) 0 m, (b) 2 m, and (c) 4 m

Figure 3.25 IDA curves for displacement of bridge column with three bridge column configurations and soil configuration 1 at an impact point height of 3 m, and scour depths of (a) 0 m, (b) 2 m, and (c) 4 m

Figure 3.26 IDA curves for moment of bridge column with three bridge column configurations and soil configuration 1 at an impact point height of 3 m, and scour depths of (a) 0 m, (b) 2 m, and (c) 4 m

Figure 3.27 IDA curves for shear of bridge column with three bridge column configurations and soil configuration 1 at an impact point height of 3 m, and scour depths of (a) 0 m, (b) 2 m, and (c) 4 m
Figure 3.28 IDA curves for rotation of bridge column with three bridge column configurations and soil configuration 1 at an impact point height of 3 m, and scour depths of (a) 0 m, (b) 2 m, and (c) 4 m 102

Figure 3.29 IDA curves for displacement of Pile 1 with three bridge column configurations and soil configuration 1 at an impact point height of 3 m, and scour depths of (a) 0 m, (b) 2 m, and (c) 4 m 103

Figure 3.30 IDA curves for axial force of Pile 1 with three bridge column configurations and soil configuration 1 at an impact point height of 3 m, and scour depths of (a) 0 m, (b) 2 m, and (c) 4 m 104

Figure 3.31 IDA curves for moment of Pile 1 with three bridge column configurations and soil configuration 1 at an impact point height of 3 m, and scour depths of (a) 0 m, (b) 2 m, and (c) 4 m 105

Figure 3.32 IDA curves for shear of Pile 1 with three bridge column configurations and soil configuration 1 at an impact point height of 3 m, and scour depths of (a) 0 m, (b) 2 m, and (c) 4 m 106

Figure B.1 Screenshot of Fragility function Generator from The University of Toledo, OH, USA 123
Abstract

Scour has been the number one cause of bridge failure in the United States with an average of 22 bridges collapsing or being closed owing to severe deformation each year [1]. This work attempts to deal with two important issues: (1) Potential bridge failures during the co-occurrence of scour and ship impact; (2) Incorporating IDA analysis to predict bridge responses based on barge and collision parameters, and to generate fragility curves to predict the probability of exceedance damage states.

For bridges, flooding is considered the most threatening hazard. According to the National Bridge Inventory (NBI) [2], 500,000 out of 615,000 bridges that cross waterways are exposed to floods, and 26,000 bridges in the U.S. are deemed scour critical (the bridge foundation is not stable). Bridge substructure component design is based on 100-year and 500-year floods.

Scour is not force effect, but scour is the loss of foundation lateral support. This loss of support affects the stability of the foundation and has significant change on forces acting on the bridge structure. The American Association of State Highway Transportation officials (AASHTO) specification [3], Load Resistance Factor Design (LRFD) methodology, does not combine probability of two extreme events: vessel collision and the bridge scour. AASHTO believes the probability of those events is very low; therefore, those events are determined separately.

The two most common causes of bridge failure are hydraulic failure due to scour of the bridge foundation and collision of vessels. Scour, the loss of soil caused by high-velocity flowing water, adversely affects the stiffness of bridges. Scour has been the predominant cause of bridge failures in the U.S., accounting for 60% of them, and it may also result in excessive rotation of the
column and displacement of the deck. The second most common cause of bridge failure in the U.S. is vessel collision, responsible for 12% of them. The above hazards are typically treated independently as single extreme events according to the AASHTO design guidelines. However, vessel collision with scoured bridge piers and piles can co-occur, and there is a lack of assessment methodologies to address this synchronous dual-hazard scenario. Based on past statistics, narrow and congested waterways are more prone to collisions between ships and bridges. The U.S. and the world are projected to experience an increase in ship sizes and higher frequencies of large vessel navigation in waterways due to global economic growth. Therefore, ship impact scenarios need closer scrutiny in the future. The aim of this research is to highlight and investigate potential bridge failures during the co-occurrence of scour and ship impact. The results of this research will help engineers to address such risks. The results illustrate that depending on the surrounding soil properties, ship impact locations, nonlinear dynamic load time history, and scour depth conditions, shear demand of the column base decreases due to increased scour at lower impact locations with clay soil configuration. In addition, the moment demand on the bridge column increases with the scour depth. The results of the parametric studies show larger displacement in piles under increased scour and ship impact locations close to pile cap, especially for clayey soil foundations.

In addition to analyzing aforementioned multi-hazardous events, this dissertation examined Incremental Dynamic Analysis (IDA) [4], a method of parametric analysis used in nonlinear dynamic systems for estimating the seismic capacities of bridges. IDA is usually applied to estimate the performance of structures under extreme events. In this research, IDA was used for evaluating the capacities of bridge components under the dual hazards of vessel collision and scour. In order to predict bridge response, a finite element model of the bridge was developed in OpenSees (Open system for earthquake engineering simulation) software [5], and IDA analysis
was performed to establish the response parameters. The selected bridge configuration was subjected to a direct barge inertia mass force with an initial velocity and a force-deformation stiffness spring to assess the effect of the above hazards on the responses of bridge components such as displacement, rotation, shear, and moment. The IDA plot typically illustrates the intensity measure (IM) on the vertical axis and the damage measure (DM) on the horizontal axis. This study focuses on the barge velocity as the intensity measure using increments of 0.25 m/sec varying from 0 to 2 m/sec with a constant barge mass of 1000 tons while the responses of the above bridge components express the damage measure. In all test cases, three different ship impact points (3, 4, and 5 m) and five different scour levels (0 m, 1 m, 2 m, 3 m, and 4 m) are used. The results show that at higher vessel velocities, the damage responses of the bridge increase as scour levels increase. It is also shown that using this type of an IDA parametric study, engineers would be able to make accurate predictions of bridge responses due to vessel collision under scour conditions and estimate the performance of structures under the above dual hazards. Hence, the IDA can be considered an additional tool of performance assessment under the above conditions. Furthermore, using the IDA results, the damageability of the bridge column under the intensity measure of barge velocity was evaluated through fragility analyses. The results show that the scour depth leads to an increase in the probability of exceedance of all damage levels thus contributing to large deformations in the bridge column under barge impact.
Chapter 1: Introduction and Literature Review

1.1 Background

The National Bridge Inventory (NBI) database indicates that around 500,000 out of 618,456 bridges in the United States (U.S.) are built over waterways [2]. There is also a higher frequency of navigable waterways, increase in ship sizes, use of narrow and congested waterways, and bridges obstructing navigation. This has led to increasing concerns of vessel collisions with bridges. As a result, increased ship collisions have been reported around the globe involving loss of lives and bridge damages and collapses. Some examples include Sunshine Skyway Bridge in Florida, U.S.A in 1980, Almo Bridge (Tjorn Bridge) in Almo sund Sweden in 1980, Tasman Bridge in Hobart, Australia in 1975, Benjamin Harrison Bridge in Virginia, U.S.A in 1977, and Northumberland Bridge in New York, U.S.A in 1973. Besides vessel-bridge collision, scour has been the predominant cause of bridge failure in the United States with an average of 22 bridges collapsing or being closed each year owing to severe deformation caused by scour [1]. Schoharie Creek Bridge in New York in 1987, Irish Bridge and Victoria Masonry Bridge in 2009, Loon Mountain Bridge and the Mississippi Highway 33 Bridge are such examples. When subjected to the dual hazards of ship collision under soil erosion, bridges come under the threat of severe deterioration, damage, and ultimate collapse. Vessels are of two types: ships and barges. The current study focuses on ship impact as time history dynamic load, and on barge impact as nodal mass with spring for IDA analyses and fragility curves. In this manuscript, scour depth has the same meaning as flood-induced scour or free pile length and soil-structure, soil-pile-structure, soil-abutment-structure interactions are used interchangeably.
1.2 Ship Collision Impact Force

Currently, there are two common methods used in evaluating the force of ship impact on bridge piers. First is the AASHTO equations adopted from Woisin’s test [6]. Woisin’s physical model was laid out for evaluating the ship impact force on a pier. This model test was developed from research conducted by Woisin in West Germany from 1967 to 1976, and this model was used to generate data for protecting nuclear power reactors of ships from collisions with other ships. The AASHTO specification [3] adopts Woisin’s model for determining the head-on ship collision impact force on a column and it is expressed in equation (1.1):

\[ P_s = 8.15 V (DWT)^{0.5} \quad (1.1) \]

where \( DWT \) is the dead weight tonnage (in tonnes) that includes everything except the weight of ship itself (weight of cargo, fuel, water and stores that submerges a vessel from empty draft to loaded draft), \( V \) is the vessel impact velocity (in feet per second), and \( P_s \) is the equivalent static impact force (in kips).

Second, the China highway specification [7] adopts equation (1.2):

\[ P = \frac{WV}{gT} \quad (1.2) \]

where \( T \) is impact time (sec), \( g \) is gravity (m/s²), \( V \) is vessel impact velocity (m/s), \( W \) is float gravity load (kN), and \( P \) is impact force (kN).

In the work described in this manuscript, in place of the equivalent static impact force, the time history of the dynamic ship impact force is utilized because it is, obviously, a more accurate and realistic way to depict the effect of collision.
1.3 Local Scour Depth

The second hazard, bridge scour, is the loss of soil due to erosion caused by water flowing around bridge supports. This study considers local scour which has the largest influence on bridge instability, with its scale being much greater than other types of scour.

To understand the process of determining the scour depth, one must first understand the underlying hydraulic principles of scour. Melville’s scour depth estimation is demonstrated in equation (1.3) [8]:

\[
d_s = K_{yb} K_t K_d K_s K_\theta K_c K_t
\]

where \(d_s\) is the scour depth, \(K_{yb}\) is the depth size of the pier, \(K_t\) is the flow intensity, \(K_d\) is the sediment size, \(K_s\) is the pier shape, \(K_\theta\) is the pier alignment, \(K_c\) is the channel geometry, and \(K_t\) is the time coefficient. Another more popular means of evaluating scour is the Hydraulic Engineering Circular (HEC 18) method, expressed in equation 1.4, (also known as the Colorado State University Equation [9]), which is based on upstream water depth, pier size, and Froude number.

\[
\frac{y_s}{y_1} = 2.0K_1 K_2 K_3 K_4 \left(\frac{a}{y_1}\right)^{0.65} Fr_1^{0.43}
\]

where \(y_s\) is the scour depth, \(y_1\) is the flow depth directly upstream of pier, \(K_1\) is the correction factor for pier nose shape, \(K_2\) is the correction factor for angle of attack of flow, \(K_3\) is the correction factor for bed condition, \(K_4\) is the correction factor for armoring by bed material, \(a\) is the pier width, and \(Fr_1\) is the Froude number directly upstream of pier. Figure 1.1 illustrates the ship impact and local scour scenario.
1.4 Concept of Incremental Dynamic Analysis

The Nonlinear Static Procedure (NSP) often called pushover analysis uses simplified nonlinear analysis to estimate structural deformation performances. Federal Emergency Management Agency FEMA 273/356 [10] defines the following performance levels: (1) Operational Performance (OP), (2) Immediate Occupancy (IO), (3) Life Safety (LS), and (4) Collapse Prevention (CP) for static nonlinear analysis. FEMA 350 and ASCE 41 guide [11] the estimation of the structural performance, while FEMA 273 has developed an analytical procedure for nonlinear static pushover analysis. These documents are defined by the acceptance criteria IO, LS, and CP, which provide the performance levels and damage description. However, Nonlinear Dynamic Procedure (NDP) is the most complete and accurate type of analysis. IDA is one method that can be used to perform NDP.
Recently, IDA has been used to evaluate the structural performance under seismic loading by Vamvatsikos and Cornell [4]. The underlying concept is to apply a series of earthquake ground motions under different intensities to the analyzed structure. IDA is considered the eyes of the assessment of demand and capacity, and hence it is an analysis that can be implemented to estimate the collapse capacity. IDA is applied to nonlinear material and structures, and IDA results are more realistic and accurate compared with nonlinear static pushover analysis. FEMA has adopted IDA to be utilized as a viable approach to determine the collapse capacity of an entire structure.

Usually, the IDA curves consist of Intensity Measure (IM) versus Damage Measure (DM) plots. As an example, in seismic loading, DM can represent a peak drift ratio or peak strain of concrete or steel, while IM can represent the spectral acceleration at the fundamental mode [4]. Selection of the relevant IMs and DMs is an important step in IDA. In this study, IDA analysis is involved in predicting the structural responses such as displacement, rotation, curvature, drift, shear, and moment of columns versus impact intensity level such as vessel mass or velocity (Figure 1.2). Interestingly, general IDA plots appear different than regular plots because conventionally, Y values depend on independent X values; however, in IDA curves the damage (X) depends on intensity measure (Y). In FEMA 350 [11], slopes of the IDA curves are used to represent levels of damage; when the consecutive points have a slope less than 20% of the elastic slope, those points are considered to be at collapse capacity. In this work, IM represents velocity which is an attribute of barge collision, and it is monotonically increased using positive increments of 0.25 m/sec in intensity associated with a positive scale factor.

An IDA curve with one vessel velocity cannot fully express the bridge behavior under vessel impact, and thus IDA utilizes the scaled velocity in order to capture nonlinear dynamic behavior of the bridge. Due to the dynamic and nonlinear nature of the vessel impact, the results
of this method in comparison to the other types of analyses are certainly closer to the reality of bridge behavior and vessel collision.

![Conceptual IDA curve](image)

Figure 1.2 Conceptual IDA curve.

IDA analysis could be performed in three ways: (1) fixed method, when the scale factor is fixed, (2) stepping method, when the scale factor increases until the collapse, and (3) Hunt-fill method, which starts with a coarse step up to the collapse condition, and then works backward to fill the intermediate steps. For earthquake studies, stepping would be implemented by incremental acceleration, velocity, or displacement until collapse is reached, whereas this work adopts incremental barge velocities with varying barge impact locations for different scour depth conditions.

### 1.5 Collapse Mechanism of IDA

Two collapse conditions are defined in IDA. The first is the simulated collapse. When the analysis fails to converge, the equations cannot be solved, or no numerical solution can be obtained to balance the forces on the structure. If the analysis is linear, then there will be no simulation
collapse compared to elastic materials and elements, nonlinear materials and elements or P-delta effect would lead to simulation collapse. The second condition is non-simulated collapse. This condition occurs when an artificial limit is assigned to prevent unrealistic structural response. It could be a displacement at a location of impact. Furthermore, the non-simulated collapse can be implemented during the analysis or the post processing stage. This study utilized the first method in order to define the collapse mechanism.

1.6 Literature Review of Scour and Ship Impact

Kameshwar and Padgett [12] evaluated the performance of bridge columns utilizing different geometric and design parameters subjected to barge impact with a variety of scour depths. Moreover, an analysis was performed to assess the post-collision stability of bridges under vehicular loads. They also conducted the impact analysis and finite element simulations to investigate the subsequent stability of bridges under vehicular loads. Their results show that the presence of free pile length reduces the shear demand and increases the moment demand on bridge columns. Kameshwar and Padgett [12] used a simplified approach to model the barge as a lumped point mass and a spring having suitable force-deformation relation. An equivalent pile is used in the analytical model to represent the entire pile group. The above paper was focused on a bridge column. In the present work, the entire bridge superstructure and substructure were modeled. In addition, particular attention was paid to the pile group and it was also modeled and analyzed. Furthermore, the soil-pile interaction, which is critical during the time history dynamic loading, was investigated. In this present study, all bridge element responses were studied under a variety of ship impact-scour hazards.

Consolazio and Cowan [13] computed the force-deformation curves for several hopper barge crush scenarios using the ADINA finite element simulation software. Consolazio and Cowan
[13] found that both the shape and size of the column impacts the relationship between force and deformation during crushing events. Furthermore, they found that if the column is flat-faced, then the width of the column influences the magnitude of force generated during crushing. On the other hand, the present study considers and investigates circular columns with different sizes and uses a time history based dynamic load.

1.7 Literature Review of IDA

Several studies have examined the application of IDA to structural problems. In a seminal publication by Vamvatsikos et al. [4], IDA is used to predict structural performance under various records of seismic loads. Furthermore, Vamvatsikos et al. [4] investigated the difference between a static pushover analysis and IDA by comparing the results of the two methods. The above work establishes definitions and provides solutions to issues with IDA intensity measure, damage measure, analysis approach, and collapse identification. Fan et al. [14] have published several research findings with respect to bridge-vessel collision. These works describe the collision duration and classifies ship-bridge collision into four interaction phases. In phase I, the non-zero speed of the ship and bridge are the same at the contact stage. Phase II occurs when the bridge and ship speed decelerate together until they reach zero velocity. In phase III, the bridge and ship accelerate in opposite directions. Finally, in phase IV, the bridge loses contact with the ship and goes into free vibration. The present study incorporates the above phases in the implementation of IDA with model assumptions.

Consolazio [15] determined two significant relationships with respect to crush-deformation: (1) barge bow force-deformation and (2) barge yield load. Consolazio [15] performed full-scale barge impact experiments in St. George Island, Florida on stand-alone pier impact, and intact bridge impact. Consolazio [15] found that the yield force of the barge depends on the shape
of the impacted bridge column and, if the column is circular in cross section, then the relationship will depend on the diameter of the column, as illustrated in equation (1.5).

\[ P_{BY} = 1400 + 30 w_P \]  \hspace{1cm} (1.5)

where \( P_{BY} \) is the barge yield load in kips, and \( w_P \) is bridge pier width in feet.

After reviewing the AASHTO crush deformation curve [3], Consolazio et al. [16] used finite element crush simulations of high-resolution barge bow models to develop new crush deformation curves, and they compared these curves with full-scale experimental data. These crush deformation curves are based on common types of barges (i.e. hoppers and tankers) in U.S. inland waterways. Consolazio et al. [16] recommended the utilization of new crush deformation curves in barge-bridge collision analysis. Furthermore, Consolazio et al. [16] found that bridge designs using round versus flat-faced piers or columns result in smaller impact loads and thus are more cost-effective.

Consolazio and Cowan [17] computed the force-deformation curves for several hopper barge crush scenarios, using the ADINA finite element simulation software. Consolazio and Cowan [17] found that both the shape and size of the column impact the relationship between force and deformation during crushing events. Furthermore, they found that if the column is flat-faced, then the width of the column influences the amount of force generated during crushing. The present study considers two aspects of Consolazio's work: force-deformation relationship and barge-yield load.

Getter et al. [18,19] put forth a simplified approach to estimate dynamic amplification effects, which generates conservative predictions of design forces for the columns and foundations. They also found that superstructure mass-related inertial forces significantly influence dynamic
bridge column behavior; however, static analysis approach common in bridge design does not consider such dynamic amplification effects. To bridge the gap, Getter et al. [18] developed the static bracketed impact analysis (SBIA) to statically mimic the most common form of collision-induced bridge dynamic amplification.

Kameshwar and Padgett [12] studied the effect of free pile length on force demand of bridge columns and the failure mode of bridges. Their work determined the shear and moment demand on bridge column and then developed metamodels for estimating these parameters without using any finite element programs. They [12] found that scoured bridges experience higher moment demands, but lower shear demand is imposed on the columns. Following Kameshwar and Padgett [12], the present study simulates the barge in OpenSees utilizing a mass node with an initial velocity and a bow spring with a suitable force-deformation relationship.

1.8 Literature Review of Fragility Analysis

Extreme events have low probabilities of occurrence but they may cause tremendous structural damage and significant socio-economic impact. Such extreme events can be earthquakes, hurricanes, and barge collisions on bridges with scoured foundations. The uncertainties associated with the response of reinforced concrete bridges to the above events have been extensively studied through fragility curves by various researchers (Akbari 2012, Kwon and Elnashi 2010) [20, 21]. Shinozuka et al. (2000, 2001) [22, 23] proposed a methodology to assess the seismic performance of RC bridges.

The fragility curve or function is a tool that will output the probability of different levels of damage based on the input which is the expected magnitude of an event. The probability of failure of a bridge can be defined as the probability of exceeding acceptable states of damage. Thus, a fragility function, which expresses the relation between the intensity of an event and the
quantitative measure of its probable damage to the structure, can be used to assess the vulnerability of a bridge structure.

The occurrence of barge impact with bridge columns in the presence of foundation scour can compromise the functionality of the bridge, and cause significant damage to the bridge or in the worst-case, possibly bridge failure. To the authors’ knowledge, only a few researchers have investigated this multi-hazard event using fragility curves. Kameshwar and Padgett [12] developed fragility functions for bridge columns to predict failure due to shear and flexural responses to barge-bridge collision in the presence of scour. The socio-economic impact and the structural damage are amplified by the co-occurrence of this multi-hazardous event and it deserves further study using alternative fragility functions.

Therefore, in this paper, to assess the performance of RC bridges subjected to scouring effects and barge impact, fragility curves are developed using the methodology proposed by Shinozuka et al. [22]. Alipour et al. [24] used drift ratio as the engineering demand parameter. The drift ratio is the fundamental limit state because the study focuses on the bridge column where the collapse occurs and relates directly to barge impact. In this approach, the drift ratio which represents an engineering demand parameter (EDP) at the impact points along the column line is used to define the bridge performance levels.

1.9 Problem Statement and Research Objective

First and foremost, the AASHTO Load and Resistance Factor Design (LRFD) equations consider only one hazard (earthquake, wind, scour, or ship collision) at a time while, in reality, one can expect to have multiple hazards simultaneously. Second, AASHTO Specification [3] is based on an equivalent static ship impact force.
However, the actual ship impact load on bridges during ship collision is of dynamic nature. Hence, one objective of this research is to employ a dynamic load with a realistic time history in the above analysis.

The second and the main objective of this research is to identify the influence of the dual hazards, scour and ship impact, on the bridge structure and foundation. This will be evaluated by the shear forces, moments, and axial forces acting on the superstructure and substructure bridge elements and the displacements that those components undergo.

Most commonly, IDA curves are developed to estimate seismic demands and capacities. In reality, one can expect to have multiple hazards simultaneously, including wind, wave, and water current loads, among others. In this study, the IDA curves of the analyzed bridge system are generated considering dual hazards: scour and vessel collision. Hence, one objective of this research is to employ a nonlinear dynamic analysis with a realistic intensity measure such as the velocity or mass of vessels. Another objective is to generate IDA curves so that engineers will be able to use precise predictions of bridge behavior and responses.
Chapter 2: Assessment of Scoured Bridges Subjected to Ship Impact

2.1 Methodology

This study utilized 18 types of bridge configurations subjecting them to 2 different time histories of loading and 3 different soil configurations subjected to the 8 different scour depths and 5 different ship impact points. Of these, while the detailed results are presented for one bridge configuration, one selected time history of loading and one soil configuration, for the above combination of scour depths and ship impact points, comparison of important results for all the cases, is presented in the Section 2.5.5 (Generalization of Results).

Several software packages such as LS-DYNA and ABAQUS are available to model soil foundation-structure interaction in general and ship impact in particular. OpenSees [5] is an adaptable program which allows users to control each step during modeling, analysis, and post processing. To achieve the aforementioned objectives in this research, OpenSees was utilized to perform both static (gravity) and lateral dynamic load analyses because of its wide range of modeling capabilities and scripting for parametric analysis. Scour depth was incorporated in OpenSees using a soil structure interaction philosophy and modeling soil as nonlinear springs. The ship impact force was modeled as a nodal force and this node forms a plastic hinge because the magnitude of moment is generally excessive.

To develop the dynamic analysis, this study used Fan and Yuan's [14] impact force time history analysis. In the above research, the maximum ship bow crush depth was determined from the principle of energy conservation.
A dead weight tonnage (DWT) of 5000 was used for determining ship-impact loads associated with an initial impact speed of 2 m/s. Of the three time histories considered in the entire study, the one shown in Figure 2.1 (TH1) was selected for the presentation of results in this manuscript. The time history TH1 of ship-impact force consists of 76 points at 0.02 time intervals. To illustrate the response of the particular bridge studied in this research, the force was scaled down by 75% because a 20,000 kN peak impact force (Figure 2.1) would not allow the model to clearly capture the bridge behavior. Under the above conditions, a numerical model was developed in OpenSees to test the effects of the two simultaneous hazards mentioned above.

Figure 2.1 Time history of ship-impact force exerted by a 5000 DWT vessel at a 2 m/s velocity compared with the corresponding AASHTO specification.

The ship impact force determined by the AASHTO guide (Eqn. 1.1) specification is constant along the time impact duration. In addition, the value of empirical ship impact force specified by AASHTO is 17,660 kips compared to the time history dynamic load where the
maximum is 20,000 kips. The areas under Figure 2.1 clearly show the constant ship impact force of the AASHTO guideline is more conservative because the cumulative momentum change \[ \int_0^{1.6} p\,dt \] imparted by the time history dynamic load simulation is much lower. Furthermore, published literature such as Consolazio and Cowan (2003) also suggests that finite element simulations predict forces that are significantly less than those predicted by AASHTO guide specification.

In the OpenSees software, a 3D finite element bridge model was built, and nonlinearities were incorporated in the columns, piles, and soil in addition to the influence of eight scour depths from 0 to 7 m. The OpenSees results produced the effect of mass and damping that must be included in the soil-structural performance analysis. The dynamic analysis is performed with static pre-analysis based on the gravity load. The ship impact force time history TH1 for a speed of 2 m/s was applied at a number of selected elevations of bent column height in the transverse direction (Global Z). The response of different bridge scour cases is discussed in detail in the ensuing sections.

2.2 Geometry of the Bridge Used for the Presented Case Study

The bridge used for the case study presented in detail in this manuscript consists of the following elements: deck, cap beams, columns, abutments, pile caps, soil and piles. The concrete three-span box girder deck is of 1.9 m depth, with the deck having 30, 36, and 30 m spans and a width of 23 m. The deck is composed of prestressed box girders made of vertical webs of 0.2 m thickness. The deck ends are supported by seat-type abutments. The superstructure and substructure bridge cross section are depicted in Figure 2.2.

The deck is supported by cap beams with a 2 m x 2 m cross section. The bents contain two circular reinforced concrete columns which are founded on pile caps. The pile caps are built on
groups of 23 piles. The bridge columns are of 1.6 m diameter with the main longitudinal rebar (50) number 11 and a transverse rebar number 6 at a 10 cm pitch. The columns stand 10 m above the pile cap and the thickness of the cap is 1.524 m. The piles have a circular cross section of 0.4 m diameter with an 18 m length. This 3-D bridge was used by Alipour [25] in Opensees. The alternative ship impact points evaluated were 3, 4, 5, 6 and 7 m against scour depth selections of 0, 1, 2, 3, 4, 5, 6 and 7 m.

Figure 2.2 Geometry of the bridge used for the case study.
2.3 Development of the Model

A schematic diagram of a finite element model for a three-span bridge and the entire elevation view of the geometry model of the bridge system in Figure 2.3 depicts how the bridge components were modeled in *OpenSees*. The following subsections describe the structural details of each bridge element.

![Figure 2.3 Schematic diagram of 3D finite element model for the case study bridge.](image)

2.3.1 Deck

The deck is a 4-cell box girder of width 23 m and cross-sectional area 12 m². The deck was modeled as an elastic beam element because flexural yield is not anticipated in it during a ship impact event. According to Caltrans [26], the effective moment of inertia was selected to be equal
to 75% of the gross moment of inertia to account for possible cracking induced by the gravity load condition. The bridge has three spans of length 30 m, 36 m, and 30 m (Figure 2.4), which represent the end, middle, and end spans, respectively.

The mass of the superstructure is assigned to deck nodes based on the tributary areas. To determine accurate vibration modes, the rotational moment of inertia of each deck segment is computed based on a rod of uniform density. The superstructure response was simulated using six equal-length elastic beam-column elements in *OpenSees* as illustrated by Figure 2.3.

### 2.3.2 Cap Beam

The cap beam has a rectangular cross-sectional area of 4.0 m² and length 11 m, and it was also modeled as a linear elastic beam element because yielding of the beam is not expected during a ship impact. The cap beam is discretized into four equal elements. The element properties are based on the depth, width, area, moment of inertia, polar moment of inertia, and shear modulus of the cap beam. As described in the following sections, columns, piles, and abutments do experience nonelastic behavior.
2.3.3 Column

The two bridge bents each have two circular reinforced concrete columns that are of 10.0 m height and 1.6 m diameter with a longitudinal reinforcement ratio of 2.5%. The base of each column is supported on a pile cap that is founded on a group of 23 piles, as seen in Figure 2.4. The top of each column is rigidly connected to the cap beam. Column-deck connection is integrated without a bearing pad. Bridge design philosophy is established on the weak column-strong deck concept. Force-based elements (FBEs) were used in order to model the non-linear behavior of the column using moment-curvature analysis which displays the stages of cracking, yielding, forming plastic hinges, ultimate, and collapse. In this study, each column is modeled using one force beam-column element with five integration points. To capture the axial force-moment interaction, each column uses fiber sections with Concrete01, Concrete04 and Steel02 material models in OpenSees.
Concrete01 represents the unconfined cover concrete, while Concrete04 represents the confined concrete based on Mander et al. [27] and Steel02 represents the longitudinal reinforcing steel. Each fiber section has 20 rings and 18 wedges. The force-based element captures the axial-moment interaction due to overturning effects of ship impact. In the case of displacement-based element (DBE), a finer finite element mesh would be required to achieve similar results. Also, the solution accuracy of a DBE can only be improved by adding more elements and not by adding numbers of integration points or NIPs, which would increase the computational expense of the analysis.

2.3.4 Pile Cap

The bridge model includes four pile caps atop the piles that support the bridge columns. Each pile cap is 7.62 m x 7.62 m in plan and 1.524 m thick. Scour evaluation starts from the bottom of the pile cap, and the bottom of the pile cap is located at the mudline. The pile cap is modeled using elastic beam elements in OpenSees to ensure that failure does not occur therein and it remains rigid during loading. Also, assigning the pile cap as an elastic element enables one to focus on the potential failure in only the piles, columns, and abutments where failure occurs most likely. Each pile cap is a grillage of 40 elements that have a square cross section of 1.524 m x 1.524 m. One half of the 40 elements are parallel to the deck (in the longitudinal direction), and the other half of the elements are perpendicular to the deck (in the transverse direction).

2.3.5 Piles

As illustrated in Figures 2.2, 2.3 and 2.4, a group of 23 circular precast prestressed reinforced concrete piles with a diameter of 0.4 m support the pile cap. In OpenSees, the material properties for piles are assigned by section elastic, uniaxial material elastic and section aggregator commands. The section aggregator command allows users to group uniaxial materials in order to model displacement-based element (DBE) behavior. The above piles with a length of 18 m are
discretized into 36 displacement beam-column elements, each with two integration points. An element length of 0.5 m is selected to coincide with the soil discretization. Similar to the columns, the pile section is discretized into confined concrete, unconfined concrete, and steel fibers that represent the concrete within spiral ties, concrete cover and main longitudinal rebar, respectively. DBE is the best option to model the piles because it allows for discretization of the pile element into 36 elements, rather than the single element which a force-based elements (FBE) would allow. The accuracy of the displacement-based element, DBE, can be improved by increasing the number of elements, and not the number of integration points. On the other hand, for FBEs used in the column, the accuracy can be increased by either increasing the number of integration points (NIPs) or the number of elements.

2.3.6 Soil

At the bridge site, the soil profile consists of six layers of varying materials. In the Soil config 1 presented in this manuscript, soil around the foundation is divided into six layers with soft clay in the first layer, stiff clay in the second, and sand in layers three through six. Each layer has assigned soil parameters such as undrained shear strength (cohesion) for clay and soil friction angle, derived from SPT values, for sand. The first soil layer starts at the mudline and is located at the top of the pile cap. The topmost layer is soft clay (undrained shear strength 75 kPa) and the second layer from top is stiff clay (undrained shear strength 144 kPa). The other layers 3, 4, 5 and 6 consisting of cohesionless soils, have SPT values of 17, 15, 18 and 28, respectively.

As demonstrated in Figure 2.5, surrounding soil is modeled as a series of nonlinear p-y springs in the lateral directions (X and Y), t-z springs along the vertical direction (Z), and q-z springs in the vertical direction at the pile tip. The soil springs are modeled using zero-length elements with three translational degrees of freedom, and spring nodes are created along the
embedded pile length. The soil-pile interaction is modeled as a beam on a nonlinear Winkler foundation (BNWF).

Figure 2.5 (a) Schematic diagram of displacement-based element pile model and zero-length element spring soil (Note: Equal DOF indicates compatible displacements of soil and pole nodes); (b) $p-y$ curve for sand; (c) $p-y$ curve for soft clay.

The stiffnesses of the springs are calculated according to the recommendation of the American Petroleum Institute (API) [28] for the $p-y$ springs, Meyerhof’s criteria [29] for the $q-z$ springs and Mosher’s criteria [30] for the $t-z$ springs. Starting at the mudline, the soil springs are spaced 0.5 m apart and characterized by $p$ and $y$ values corresponding to 50% of the ultimate capacity (Figure 2.5(b) and 2.5(c)). To represent soil erosion and scour around a given pile, the soil springs within the pile length are removed from the model up to the desired scour depth.
2.3.7 Abutment

The seat-type bridge abutments consist of a stem wall, back wall, wing wall, shear key, and foundation (Figure 2.6). The backwall length is 23 m and the wing wall length is one-third of the back wall length (7.7 m). Zero-length spring elements represent the force-deformation behavior of the abutment components. The stem wall supports the bridge deck and it is modeled using a bilinear force-deformation model with material behavior Steel01 in OpenSees (Figure 2.7c).

The passive soil model for the abutment is based on Shamsabadi et al. [31 and 32]. The nonlinear behavior of the backfill material in the longitudinal direction uses the Hyperbolic Gap Material in OpenSees (Figure 2.8). The backfill was modeled as a compression-only, zero-length element because it is expected to be under compression during a ship impact event.

The bearing pads (BPs) were modeled based on recommendations of AASHTO LRFD Bridge Design Specifications. They have dimensions of 15” parallel to the deck, 24” in the transverse direction, and a thickness of 5.58”. Elastomeric BP are made of rubber and reinforced by horizontal shim steel plates under high vertical loads. They are combined with the back wall and backfill materials in series, and with the shear key in parallel when strains are the same and stresses and stiffnesses are additive (Figures 2.7e, 2.7f, 2.7g and 2.7h). The behavior of the bearing pad is represented by Steel01 in the longitudinal direction with compression-only behavior in the vertical direction. In addition, the longitudinal stiffness of the bearing pad is based on the shear modulus without shim steel plates, while the elastic stiffness in the vertical direction includes both rubber and shim steel plates. The bearing pad and the embankment are joined in series in the vertical direction with stresses and stiffnesses being the same while the strains are additive.
Figure 2.6 Bridge seat-type abutment components.

Zhang and Makris [33] provide the expression for determining the embankment stiffness. The stiffness was estimated using equation (2.1) based on a unit width of embankment:

\[
k_z = \frac{4(1+\nu_s)G_s}{S \ln \left(1 + \frac{2H}{SB_c}\right)}
\]  

(2.1)

where \(B_c\) is the width of the embankment at its crest, \(S\) is the slope of the embankment, \(H\) is the height of the embankment, \(G_s\) is the shear modulus of the soil, and \(k_z\) is the vertical static stiffness of the shear wedge.

In the current bridge model, the width of the embankment at the crest is 23 m, and the height is 4.5 m. Compression-only, zero-length springs model the embankment and they are placed in series with the bearing pad material to represent the vertical abutment behavior. The deck and abutments of bridge could collide due to the closing of a 5.0 cm gap that represents the expansion joint between the superstructure and the abutment. This gap closes due to the ship impact load and
causes forces called pounding forces, which can damage the bridge components. To monitor the impact behavior, Herzdamp’s model is used based on the bilinear impact model with gap proposed by Muthukumar [34]. The above model captures the impact effect and accounts for both energy dissipation and energy absorption. In OpenSees, the pounding force was modeled as a zero-length element with a uniaxial impact material.

Figure 2.7 Stress-strain material models (a) Concrete01 (b) Concrete04 (c) Steel01 bearing pad material in longitudinal direction; (d) ENT bearing pad material in vertical direction; (e and f) in-parallel material modeling, strains are equal and stresses are additive; (g and h) in-series material modeling, strains are additive and stresses are equal; (i) tension gap; (j) compression gap; (k) impact material (l) impact force.
2.4 Analyses

2.4.1 Pushover Analysis

This section demonstrates the use of OpenSees in order to perform the nonlinear static pushover analysis (NSPOA). The NSPOA was performed in the transverse direction of the bridge column at a 5 m ship impact height for different scour conditions (0, 1, 2, 3, 4, 5, 6 & 7 m), as illustrated in Figure 2.9. The abscissa represents the applied lateral displacement at the ship impact node while the ordinate represents the shear force that results from this applied displacement. The goal of the NSPOA is to evaluate the overall strength of the bridge column, typically measured through base shear, yield, and maximum displacement, as well as the ductility capacity of the bridge structure. To perform NSPOA, a gravity analysis is a necessary precursor which corresponds to the initial negative displacement in Figure 2.9.
This study clarified that pushover displacement control, which in this case is lateral displacement, is gradually increased until the control point (ship impact point) reaches target displacement (0.15 m). This strategy uses incremental displacement iteratively to determine the time required to impose the target displacement. Moreover, a reference node was defined and located at the ship impact node and displacement increments of $1.8 \times 10^{-4}$ m were used to reach the target displacement. This increment size was selected to be small enough to capture the progression of hinge formation and generate a smooth backbone curve. The pushover analysis displacement control technique is used to evaluate the capacity of the bridge column and highlight the flexibility of the bridge. According to Figure 2.9, the capacity for shear force at zero scour (9,200 kN) is 1.5 times the capacity at scour 7 m (6,000 kN) at the corresponding displacements where instability begins.

2.4.2 Modal Analysis

Another important step of time history dynamic (transient) analysis is to run modal analysis directly after gravity analysis. Determination of natural frequencies and mode shapes are a starting point for a transient analysis. The frequencies are dependent on the mass, stiffness, and damping properties of the bridge. Modal analysis can be performed either theoretically or experimentally, and in this study, the theoretical approach was used in order to model a discrete system of lumped mass points. When the structure is damaged, its natural frequency will be lowered because of the decrease in overall stiffness, and the modal shapes will be changed because of the stiffness redistribution due to the defects. Because of the development of cracks, damping is increased. An undamped free vibration system (with no external excitation forces present) is used to calculate the natural frequencies (eigenvalues) and mode shapes (eigenvectors) for multiple degree-of-freedom (DOFs) systems, as illustrated by the equations (2.2) to (2.5):
Figure 2.9 Pushover displacement control at the middle of the bridge column.

- General equation of motion:
  \[ [M] \times \{\ddot{x}\} + [c] \times \{\dot{x}\} + [k] \times \{x\} = \{f(t)\} \]  
  \[ (2.2) \]

- Free vibration undamped system:
  \[ [M] \times \{\ddot{x}\} + [k] \times \{x\} = \{0\} \]  
  \[ (2.3) \]

- Eigenvalue Problem:
  \[ [k - w_n^2 M] \times \Phi_n = \{0\} \]  
  \[ (2.4) \]

- Characteristic Equation:
  \[ \text{det}([k - w_n^2 M]) = 0 \]  
  \[ (2.5) \]

where \( \{x\} \) is the displacement vector, \( \{\dot{x}\} \) is the velocity vector, \( \{\ddot{x}\} \) is the acceleration vector, \( [M] \) is the diagonal mass matrix, \( [c] \) is the damping matrix, \( [k] \) is the symmetric and diagonally dominant stiffness matrix, and the \( \{f(t)\} \) represents all the forces acting on the system.
In this research, the goal of performing modal analysis is to investigate the effect of scour on the natural frequencies and periods of the bridge. In addition, the above analysis describes the configuration into which a bridge naturally displaces in longitudinal, transverse or torsion modes. Furthermore, modal analysis provides the knowledge of the frequencies that should prevent the resonance conditions associated with the occurrence of the scour and ship impact.

Figures 2.10 and 2.11 demonstrate the mode shapes 1, 2 and 3 for scour levels of 0 and 7 m, respectively. Mode 1 is dominated by longitudinal translation for both scour cases 0 and 7 m. However, transverse translation dominated Mode 2 for scour 0 while deck’s transverse translation and bent’s and abutment’s global torsion govern Mode 2 for scour 7 m. Mode 3 demonstrated a bending mode shape for scour 0 m while global torsion Mode 3 was prominent for scour 7 m. According to the modal analysis results, the influence of the scour changes the nature of the dynamic properties of the bridge such as frequency and mode shapes implying that the bridge becomes more flexible. Table 2.1 shows the period values of the bridge in the modal analysis for a variety of scour cases. The periods of the bridge increase with the scour depth. It is also seen from Table 2.1 that, for every 1 m increase in scour, there was an associated increase in the flexibility of the bridge in all mode shapes.

2.4.3 Moment-Curvature Analysis

Typically, the moment-curvature response shows the moment capacity of a reinforced concrete section in the uncracked, cracked, yield and ultimate regions. In the uncracked region of a reinforced concrete section, concrete and steel are both linear elastic and concrete remains uncracked. In the cracked region, concrete and steel are both elastic but concrete exists in tension. In the ultimate region, concrete and steel are both inelastic. In the yield region, the flexural stiffness (EI) is decreased. In the work presented here, this analysis was used to capture the axial-moment
interaction in the bridge columns composed of circular reinforced concrete sections (Figure 2.12). For modeling purposes in *OpenSees*, the force-based beam column element with the fiber discretization of the cross section is used. Two patches of concrete sections (confined and unconfined) and one layer of reinforcing steel are defined in *OpenSees*. Uniaxial material *Concrete01* is assigned for the column cover because the material assumes no tensile strength for concrete and does not consider tensile resistance. It is a Kent-Scott-Park concrete stress-strain relationship model [35]. Uniaxial material *Concrete04* is assigned for the column core because the material assumes compression and the exponential decay of tensile strength for concrete. It is a Karsan-Jirsa concrete model [36]. Columns have a diameter of 1.6 m with a longitudinal reinforcement ratio of 2.5%. Each column is reinforced with 50 main longitudinal rebar # 11, and spiral rebar #6 with a pitch spacing 0.1 m in the transverse direction. In the core region, columns are subdivided into 25 sections with equal angle spacing in the transverse direction, and further subdivided into 10 rings in the radial direction. As a result, there are 250 fibers in the core zone. In the cover region, columns are subdivided into 25 sections with equal angles in the transverse direction, and further subdivided into 2 rings in the radial direction, totaling 50 fibers in the cover zone. Therefore, total subdivisions of cover (unconfined) and core (confined) concrete amounts to 300 fibers. In other words, in *OpenSees*, the model type section’s force deformation test discretizes a section into small regions to evaluate the stress-strain response. The analysis of cross-section’s bending moment-curvature characteristics is the basic analytical tool for determining the ductility capacity of member curvature, which is necessary for predicting plastic deformation capacity of the bridge column. As illustrated in Figure 2.13, when an axial load of 5000 kN (gravity load) was applied on the section during the moment-curvature analysis, the resulting maximum moment was 19,650 kN.m.
Figure 2.10 Mode shape for scour 0.0 m.
Figure 2.11 Mode shape for scour 7.0 m.

Table 2.1 Modal analysis results of the bridge periods

<table>
<thead>
<tr>
<th>Mode</th>
<th>Period (seconds)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Scour 0</td>
</tr>
<tr>
<td>Mode1</td>
<td>0.60506</td>
</tr>
<tr>
<td>Mode2</td>
<td>0.35241</td>
</tr>
<tr>
<td>Mode3</td>
<td>0.28385</td>
</tr>
</tbody>
</table>
2.4.4 Moment-Curvature Demand Hysteretic Diagram

Hysteretic moment curvature characteristics are commonly used to determine the applied moment demand. Hysteretic models can capture the response of the material and can be used as a predictor of the ship impact performance that evaluates the adequacy of the column cross-section. The hysteretic loading protocol was used in OpenSees framework as follows: (1) The impact force was applied at different heights that represent the ship impact point along the column. The height was measured from the top of pile cap. (2) The ship impact load was applied sequentially at time step of 0.02 seconds, and the intensity of this force was increased at each time step until the peak time history impact force was reached at 0.8 seconds. (3) From this point, the load on the bridge column was decreased incrementally at the same time step until the structure was completely unloaded. (4) Following the unloading phase, the bridge was allowed to vibrate freely for 12 seconds. (5) During the loading and unloading phases, the moment and curvature at the point of impact were recorded in OpenSees at each time step, and post-processed to generate the moment-curvature graph (Figure 2.14).
Figure 2.14 illustrates the hysteretic behavior when a ship impact height of 3 m is associated with the two scour cases of 3 m (SH3Scr0) and 7 m (SH3Scr7). The observed unsymmetrical moment curvature hysteretic diagram is due to the lateral ship impact and it can predict how the concrete column reacts due to a ship impact. Based on the hysteretic diagram, a significant change of the column moment demand can be seen in comparison of the bridge stiffness. The decrease in stiffness increases the impact height due to present scour. This leads to increase in the column moment demand.

![Moment curvature of the bridge column](image)

Figure 2.13 Moment curvature of the bridge column.
The hysteretic loop shows the dissipated energy (area within the loading-unloading loop) and the absorbed energy (area under the unloading curve) of a system. Also, it is used to make sure that the bridge is ductile enough to resist the expected lateral ship impact force. The shape of the cyclic part in Figure 2.14 is referred to as pinched cycle. Moreover, Figure 2.14 shows that there will be more energy dissipation when scour depth is 7 m as opposed to 0 m.

### 2.5 Results and Discussion

Explicit dynamic nonlinear time history analyses were conducted to assess the response of bridge components such as deck, columns, piles and abutments under eight scour conditions for five ship impact cases. The maximum ship impact force occurred at 0.8 seconds during the 1.52 second total time interval. The impact locations represent mean high water levels (MHW) per AASHTO LRFD Bridge Specifications.
2.5.1 Effect on Deck Center Displacement

In order to assess the behavior of the bridge due to the existence of a scour during ship impact, the deck center displacement was also evaluated. Figure 2.15 illustrates the maximum displacement response due to the dynamic load time history at three ship impact locations (3, 5 and 7 m) for the scour conditions 0 to 7 m. It is seen from Figure 2.15 that the deck center displacement in the transverse direction (Z axis) increases when the soil is eroded more. However, the response of the deck was not significantly influenced when the scour condition was 7 m, even at a variety of impact point heights, whereas in the pristine condition, the displacement at the ship impact point of 7 m is increased 1.4 times compared to the ship impact point of 3 m. The above results lead to the conclusion that the change in the flexibility of the bridge is mostly due to the increase in the unsupported length of the pile, rather than the location of the ship impact point.

Figure 2.15 Deck center displacement due to various scour conditions (0 to 7 m) at different impact locations (3, 5 and 7 m).
2.5.2 Effect on Base of Column

2.5.2.1 Evaluation of Column Base Shear Force

The maximum absolute base shear demand of piers at the first integration point was calculated for all eight scour conditions under three ship impact points. As seen in Figure 2.16, the base shear did change under different scour cases for each specific impact point, changing significantly when the ship impact point was close to the column base. However, the base shear force of the column decreased significantly when the impact point shifted higher and away from the base of the column and decreased slightly due to the change in scour depth. When scour is present, the unsupported length of the column increases leading to a decrease in the stiffness of the column. As a result, the column base shear demand decreases, but the superstructure experiences greater shear demand.

![Figure 2.16 Base shear of column under various scour conditions (0 to 7 m) at different impact locations (3, 5 and 7 m).]
2.5.2.2 Evaluation of Column Base Moment

As illustrated in Figure 2.17, worsening scour conditions did increase the column base moment demand. However, the base moment of the column decreased significantly when the impact point shifted higher and away from the base of the column for all scour conditions.

![Figure 2.17 Base moment of column under various scour conditions (0 to 7 m) at different impact locations (3, 5, and 7 m).](image)

2.5.3 Effect on Piles

A plan view of the pile group is presented in Figure 2.18. The driven piles in the group are arranged in a 5 x 5 pattern with a spacing of 1.524 m in both directions. The piles are Class-140 precast prestressed reinforced concrete, and they are designed based on the California Department of Transportation (Caltrans) code [26]. The bridge column is subjected to ship impact toward the front row of piles (piles 1, 6, 11, 14, and 19). Because this case study uses a bridge having a group of piles, pile group p-multipliers are needed to be used in the analysis.
Furthermore, because the impact comes from a single direction, the rows of piles experience different levels of impact force. Therefore, pile group p-multipliers of 0.85 for the front row (closest to ship), 0.5 for the middle rows and 0.3 for the back-row piles were used in keeping with the results of Rollins’s experiment [37] and NCHRP Report 461 [38]. The following sections describe the results of the OpenSees analyses run on this pile arrangement.

Figure 2.18 Plan view of pile group under Column 1.
2.5.3.1 Evaluation of Pile Shear Force

The load carried by each pile in the group was determined in order to investigate the shear forces, moments, axial forces, and displacements. Group effects reduced the lateral resistance in all rows. Pile 19 in the bottom-right corner in group 1 was selected and analyzed for shear. Results in Figure 2.19 show that shear force is significantly affected by both the ship impact location and scour depth.

![Graph showing maximum absolute shear force of Pile 19 for various scour conditions (0 to 7 m) at different impact locations (3 to 7 m).](image)

Figure 2.19 Maximum absolute shear force of Pile 19 for various scour conditions (0 to 7 m) at different impact locations (3 to 7 m).

2.5.3.2 Evaluation of Pile Moment

The results also illustrated that presence of free pile length does lead to more significant flexural conditions associated with the ship impact force. According to OpenSees results, Pile 19 was exposed to the maximum applied moment under all simulated ship impact locations and scour conditions. Figure 2.20 shows the trend of flexural demand and it decreases when elevating the height of the impact location. Unsupported length of the pile increases with scour, and this changes the flexibility of the bridge.
The piles should be designed carefully to meet the demand of scour and ship impact force simultaneously, and this includes modifying the geometry, material and reinforcement of the piles.

Figure 2.20 Maximum absolute moment of Pile 19 for various scour conditions (0 to 7 m) at different impact locations (3 to 7 m).

2.5.3.3 Evaluation of Pile Displacement

The displacement along Pile 1 is computed at impact points varying from 3 m to 7 m combined with scour varying from pristine to 7 m conditions. As seen in Figure 2.21, the displacement curve shows an increasing trend with worsening scour at a specific ship impact point. Therefore, under the dual hazards of scour and vessel impact, the pile foundation could sustain severe damage possibly leading to the collapse of the entire bridge.
2.5.3.4 Evaluation of Pile Axial Force

The maximum absolute axial force occurs in Pile 11, which is in the middle front row (Figure 2.18), when the ship impact height is 3 m. The absolute axial force demand increases with scour depth (Figure 2.22). Therefore, under the dual hazards of scour and vessel impact, axial force demand may also lead to the collapse of the entire bridge.
2.5.4 Effect on Soil Response

In this study, the force exerted by the soil spring is also evaluated with respect to the depth of piles as seen in Figure 2.23. Piles 11, 12, and 13 were selected to illustrate the behavior of nonlinear soil springs at the front, middle, and back rows of piles, respectively. The corresponding soil responses were determined by evaluating the forces in the p-y springs and recorded at time step 40 (0.8 seconds). Figure 2.23 shows the spring force demand when the ship impact point was at a 3 m height and the scour depth was 0 m. It is seen that, as expected, the brunt of the impact is taken by the piles in the front row.

![Figure 2.23 Soil responses of piles 11, 12 and 13 with a scour of 0 m at a ship impact of 3 m.](image)

2.5.5 Generalization of Results and Parametric Studies

Finally, this study was expanded to consider a total of 18 types of bridge configurations subjecting them to 2 different time histories of loading, 3 different soil configurations subjected to the 8 different scour depths, and 5 different ship impact points considered in the presented case study.
Thus, 4,320 finite element simulations were performed in all. In each case, a time history dynamic analysis is performed to assess the shear force, moment, axial force, and displacement in the bridge components. Additionally, modal analysis, pushover analysis, and moment curvature in the column are also performed to evaluate the stiffness of bridges, shear, and the moment capacities of the column, respectively. The ensuing Figures 2.24-2.30 illustrate the differences in scoured bridge response due to the variations in the above parameters.

Displacement of ship impact point changes due to different soil configurations. Figure 2.24 illustrates that the bridge configuration 5 (end span 30 m and column height 10 m) experiences a slightly larger ship impact point displacement when the soil is soil configuration 3 (clay).

Column base moment changes due to variety of soil configurations. Figure 2.25 depicts that bridge configuration 5 (end span 30 m and column height 10 m) experiences the highest absolute maximum base moment demand under all the scour conditions when the soil is soil configuration 3 (clay).

The influence of soil configuration on the moment developed in Pile 19 due to a ship impact height of 3 m with and without scour for different type of soil configurations is shown in Figure 2.26. It is demonstrated that the soil configuration 3 experiences the highest maximum moment.
Figure 2.24 Impact point nodal displacement due to ship height 3 m and scour 7 m with different soil configuration (config 1 clay-sand, config 2 sand & config 3 clay).

Figure 2.25 Column base moment due to ship height 3 m with different soil configurations.
The influence of the different soil configurations on the displacement response of Pile 1 associated with the bridge configuration 5 with a scour depth of 5 m and ship impact height of 3 m is seen in Figure 2.27. It is observed that Pile 1 experiences the largest maximum displacement in the soil configuration 3 (clay).

Figure 2.27 Pile 1 displacement due to ship height 3 m at scour depth 5 m with different soil configurations.
A parametric study was conducted to observe how the ratio of the maximum central deck displacement due to ship impact with scour to that with no scour (vertical axis in Figure 2.28) varies with the ratio of column height to pile diameter (horizontal axis in Figure 2.28) for a pile foundation consisting of 23 piles and a soil configuration of 2 (all sand) for one ship impact (5m height) and scour (3m) case. It is seen from Figure 2.28 that if the pile diameter is increased in proportion to the column height, the additional risk due to scour can be minimized.

![Graph showing the comparison of central deck displacement due to ship impact with scour to that with no scour versus column height to pile diameter ratio.](image)

Figure 2.28 Comparison of central deck displacement due to ship impact with scour to that with no scour to column height/pile diameter ratio due to a ship impact at a height of 5 m with 3 m scour.

Moment versus curvature analysis was performed for the 10 m long and 1.6 m diameter column for two subdivision fibers selections (Figure 2.29). It is seen from Figure 2.29 that a fiber section with 300 subdivisions would provide results that are adequately accurate. This justifies the number of 250 subdivisions used in the detailed case study presented in the manuscript.
Figure 2.29 Moment curvature fiber comparison for 1.6 m with column height of 10 m.

Moment capacity versus curvature comparison was also performed for three columns of different sizes. Figure 2.30 illustrates the development of less curvature with high moment capacity for larger cross section columns.

Figure 2.30 Moment curvature comparison for 1.3, 1.6, 1.9 m diameter column with heights of 7.5, 10.0 and 12.5 m, respectively.
Chapter 3: Incremental Dynamic Analysis and Column Fragility Analysis of Scoured Bridges Subjected to Barge Impact

3.1 Barge Collision

3.1.1 Barge Impact Theory

The theoretical concepts covered in this section deal with the physics behind barge impact theory and nonlinear dynamic IDA analysis. The barge impact theory originates from experiments that were conducted on hopper barges striking bridge piers. Additionally, barge-bridge collisions occur more often than ship-bridge impact incidents because shallow draft barges are capable of navigating more waterways with bridge crossings. Standard hopper barges are commonly used in inland waterways, and hence this work focuses on barge impact. The AASHTO barge collision equations that are related to this research are presented below in the ensuing sections.

3.1.1.1 Barge Collision Energy

The barge kinetic energy is calculated using the classic kinetic energy equation, and it depends on two variables: (1) barge mass (2) barge velocity. The barge mass used for calculations in this work was acquired from the U.S. barge manufacturers.

\[ KE = CH \left( \frac{1}{2} MV^2 \right) \]  

(3.1)

where \( KE \) is the kinetic energy of barge in kip-ft U.S. customary unit, or Joules in metric unit, \( CH \) is a hydrodynamic mass of 1.05 if the under-keel clearance is more than half of the barge draft or 1.25 if the under-keel clearance is less than 0.1 times the draft, \( M \) is the mass of a barge in tonnes, and \( V \) is the impact velocity in ft/sec in U.S. customary unit or m/sec in metric unit. The under-
keel clearance is the spacing between the deepest point of the barge’s hull to the ground. The barge draft is the distance between the waterline and the barge hull bottom point.

The AASHTO barge kinetic energy equation [3] considers characteristics of the navigable waterway, barge traffic type, and bridge category factors in order to generate barge kinetic energy.

\[
KE = CHMV^2 \quad \text{metric unit} \\
KE = \frac{CHWV^2}{29.2} \quad \text{U.S. customary unit}
\]

where \( KE \) is the kinetic energy, \( CH \) is a hydrodynamic mass, \( M \) or \( W \) is the mass of a barge in tonnes, and \( V \) is the impact velocity.

### 3.1.1.2 Barge Bow Damage Length

After determining the barge kinetic energy, the second step is to establish the barge bow damage length. In this context, several terms are used to express barge bow deformation: bow damage length, the penetration depth of the bridge pier into the barge bow, and crush depth. The crush deformation of the barge is dependent on the dissipation of the aforementioned kinetic energy. AASHTO equations 3.4 and 3.5 reflects the relationship of the bridge pier penetration into the barge bow, crush deformation, with barge kinetic energy.

\[
a_B = 3100\left(\sqrt{1 + 1.3 \times 10^{-7}KE} - 1\right) \quad \text{metric unit} \tag{3.4}
\]

\[
a_B = 10.2 \left(\sqrt{1 + \frac{KE}{5,672}} - 1\right) \quad \text{U.S. customary unit} \tag{3.5}
\]

where \( a_B \) is the barge bow damage length in mm and \( KE \) is the barge collision energy in Joules. \( a_B \) shown in equation 3.5 is used for barge bow damage length, deformation depth, or penetration depth in feet, and \( KE \) is the kinetic energy of barge in kip-ft. Equations 3.4 and 3.5 are applicable for a standard hopper barge of width 35 ft or 10.2 m, and this penetration expression should be altered for other barge widths.
3.1.1.3 Barge Collision Force on Pier

The pier force-deformation relationship is based on an empirical bilinear function. The pier impact force for standard hopper barge is based on the crush depth limit as shown in equations 3.6-3.9 in two unit systems.

\[
\begin{align*}
\text{If } a_B < 100 \text{ mm } & \quad P_B = 6.0 \times 10^4 \, a_B \quad \text{metric unit} \tag{3.6} \\
\text{If } a_B \geq 100 \text{ mm } & \quad P_B = 6.0 \times 10^4 + 1600 \, a_B \quad \text{metric unit} \tag{3.7}
\end{align*}
\]

where \( a_B \) is the barge bow damage length in mm, and \( P_B \) is the barge impact force in Newtons.

The barge impact force should increase by the ratio of the nonstandard barge’s width to the width of standard hopper barge (35 ft or 10.2 m). The equations that predict the force-crush depth relation in the U.S. customary unit are shown below.

\[
\begin{align*}
\text{If } a_B < 0.34 \text{ ft } & \quad P_B = 4,112 \, a_B \quad \text{U.S. customary unit} \tag{3.8} \\
\text{If } a_B \geq 0.34 \text{ ft } & \quad P_B = 1,349 + 110 \, a_B \quad \text{U.S. customary unit} \tag{3.9}
\end{align*}
\]

where \( a_B \) is the barge bow damage length in feet, and \( P_B \) is the barge impact force in kips.

Equations 3.1 to 3.9 are utilized in order to model the material and element stiffness of the barge in the finite element solution for which the OpenSees software is used in this work.

3.2 Theoretical Concept of IDA

The nonlinear dynamic IDA is performed to establish the response parameters at a variety of different barge velocities from 0 to 2 m/sec and a specific barge mass (1,000 metric tons). In this research, the barge-bridge collision is represented by modeling the barge as a mass of point and a spring with force deformation relationship.

The nonlinear dynamic problem that needs to be solved is expressed in equation 3.10. The solution is the time dependent response of all bridge nodes that include inertial, damping, and stiffness forces. The inertial force is mass times acceleration, the damping force is damping
The coefficient times velocity, and the stiffness force is the stiffness times displacement. The two equations implemented in this work are the equation of motion during the collision and free vibration after the barge loses contact with the bridge.

- General equation of motion:
  \[ [M] \cdot \{\ddot{x}\} + [c] \cdot \{\dot{x}\} + [k] \cdot \{x\} = \{f(t)\} \]  
  \[ (3.10) \]

- Free vibration undamped system:
  \[ [M] \cdot \{\ddot{x}\} + [k] \cdot \{x\} = \{0\} \]  
  \[ (3.11) \]

where, for the bridge, \( \{x\} \) is the displacement vector, \( \{\dot{x}\} \) is the velocity vector, \( \{\ddot{x}\} \) is the acceleration vector, \( [M] \) is the diagonal mass matrix, \( [c] \) is the damping matrix, \( [k] \) is the symmetric and diagonally dominant stiffness matrix, and the \( \{f(t)\} \) represents all the forces acting on the system. There are characteristics of the \( [M] \) and \( [k] \) matrices. First, the mass matrix is built based on lumped mass points. The bridge model contains 10,276 nodes. Some nodes have only three degrees of freedom, whereas some have six degrees of freedom including translations and rotations. The second aspect is that a stiffness matrix is related to EI/L (modules of elasticity times moment of inertia divided by length of element), and the bridge model consists of 6,936 finite elements. The OpenSees program can handle these types of matrices and solve the dynamic equations in order to determine the responses such as displacements.

The direct integration of the dynamic differential equation is used in this study as a transient response analysis. This direct integration is adopted after approximation of velocity and acceleration components is done by a finite difference method. The direct integration method involves the system equations and requires many time steps to complete the solution in each step. The common methods of direct integration for dynamic analysis include the Newmark Method and Hilbert-Hughes-Taylor Method. The accuracy of these methods depends on the suitability of
a given time step. There are several components that should be defined in nonlinear analysis: constraint handler system equations, mapping between equation numbers, numbers of degree of freedom, system of equations, solution of the nonlinear equation algorithm, and convergence test of the stiffness matrix. After defining all nonlinear dynamic analysis components, then IDA analysis can be performed. In short, equations 11 and 12 are implanted to run the finite element nonlinear dynamic analysis simulation.

3.3 Bridge Configurations

This study was expanded to consider a total of 18 types of bridge configurations subjected to a simulated barge collision. Bridge configurations 1 to 9 are the same as the bridge configurations 10 to 18 with respect to all parameters except pile diameters. Table 3.1 shows all type configurations that this study investigated. The bridge configurations and soil profiles are utilized for generalization of bridge components responses. Bridge configuration 5 is the one that is most tested. The bridge has a concrete three-span box girder deck having 30, 36, and 30 m spans. Note, these profiles are utilized for the entirety of this research.

At the bridge foundation, the soil profile 1 consists of six layers of varying materials. The first layer is soft clay, the second is stiff clay, and third through sixth layers sand. Each layer has assigned soil parameters such as undrained shear strength (cohesion) for clay and soil friction angle, derived from SPT values, for sand. The first soil layer starts at the mudline and is located at the top of the pile cap. The topmost layer is soft clay (undrained shear strength 75 kPa) and the second layer from top is stiff clay (undrained shear strength 144 kPa). The other layers 3, 4, 5 and 6 consisting of cohesionless soils, have SPT values of 17, 15, 18 and 28, respectively (Table 3.2). Moreover, soil configuration 2 is only sand, and this configuration made of four layers having SPT values of 17, 15, 18 and 28 from top to bottom, respectively. Soil configuration 3 is only clay, and
it is composed of two layers of clay having soft clay (undrained shear strength 75 kPa) in the topmost layer and stiff clay (undrained shear strength 144 kPa) in the second layer. Table 3.2 illustrates the soil configurations that were tested in this research.

Table 3.1 Bridge configurations

<table>
<thead>
<tr>
<th>Bridge Config.</th>
<th>End Span (m)</th>
<th>Middle Span (m)</th>
<th>H_col (m)</th>
<th>D_col (m)</th>
<th>Nbars</th>
<th>lbar</th>
<th>P_dia (m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>15</td>
<td>18</td>
<td>7.5</td>
<td>1.3</td>
<td>40</td>
<td>11</td>
<td>0.4</td>
</tr>
<tr>
<td>2</td>
<td>15</td>
<td>18</td>
<td>10</td>
<td>1.6</td>
<td>50</td>
<td>11</td>
<td>0.4</td>
</tr>
<tr>
<td>3</td>
<td>15</td>
<td>18</td>
<td>12.5</td>
<td>1.9</td>
<td>60</td>
<td>11</td>
<td>0.4</td>
</tr>
<tr>
<td>4</td>
<td>30</td>
<td>36</td>
<td>7.5</td>
<td>1.3</td>
<td>40</td>
<td>11</td>
<td>0.4</td>
</tr>
<tr>
<td>5</td>
<td>30</td>
<td>36</td>
<td>10</td>
<td>1.6</td>
<td>50</td>
<td>11</td>
<td>0.4</td>
</tr>
<tr>
<td>6</td>
<td>30</td>
<td>36</td>
<td>12.5</td>
<td>1.9</td>
<td>60</td>
<td>11</td>
<td>0.4</td>
</tr>
<tr>
<td>7</td>
<td>45</td>
<td>54</td>
<td>7.5</td>
<td>1.3</td>
<td>40</td>
<td>11</td>
<td>0.4</td>
</tr>
<tr>
<td>8</td>
<td>45</td>
<td>54</td>
<td>10</td>
<td>1.6</td>
<td>50</td>
<td>11</td>
<td>0.4</td>
</tr>
<tr>
<td>9</td>
<td>45</td>
<td>54</td>
<td>12.5</td>
<td>1.9</td>
<td>60</td>
<td>11</td>
<td>0.4</td>
</tr>
<tr>
<td>10</td>
<td>15</td>
<td>18</td>
<td>7.5</td>
<td>1.3</td>
<td>40</td>
<td>11</td>
<td>0.6</td>
</tr>
<tr>
<td>11</td>
<td>15</td>
<td>18</td>
<td>10</td>
<td>1.6</td>
<td>50</td>
<td>11</td>
<td>0.6</td>
</tr>
<tr>
<td>12</td>
<td>15</td>
<td>18</td>
<td>12.5</td>
<td>1.9</td>
<td>60</td>
<td>11</td>
<td>0.6</td>
</tr>
<tr>
<td>13</td>
<td>30</td>
<td>36</td>
<td>7.5</td>
<td>1.3</td>
<td>40</td>
<td>11</td>
<td>0.6</td>
</tr>
<tr>
<td>14</td>
<td>30</td>
<td>36</td>
<td>10</td>
<td>1.6</td>
<td>50</td>
<td>11</td>
<td>0.6</td>
</tr>
<tr>
<td>15</td>
<td>30</td>
<td>36</td>
<td>12.5</td>
<td>1.9</td>
<td>60</td>
<td>11</td>
<td>0.6</td>
</tr>
<tr>
<td>16</td>
<td>45</td>
<td>54</td>
<td>7.5</td>
<td>1.3</td>
<td>40</td>
<td>11</td>
<td>0.6</td>
</tr>
<tr>
<td>17</td>
<td>45</td>
<td>54</td>
<td>10</td>
<td>1.6</td>
<td>50</td>
<td>11</td>
<td>0.6</td>
</tr>
<tr>
<td>18</td>
<td>45</td>
<td>54</td>
<td>12.5</td>
<td>1.9</td>
<td>60</td>
<td>11</td>
<td>0.6</td>
</tr>
</tbody>
</table>
Table 3.2 Soil configurations

<table>
<thead>
<tr>
<th>Soil Configuration</th>
<th>Layers</th>
<th>Type</th>
<th>Undrained Shear Strength (kPa)</th>
<th>SPT (blows)</th>
<th>Depth (m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>1</td>
<td>Soft clay</td>
<td>75</td>
<td>-</td>
<td>4.5</td>
</tr>
<tr>
<td></td>
<td>2</td>
<td>Stiff clay</td>
<td>144</td>
<td>-</td>
<td>3</td>
</tr>
<tr>
<td></td>
<td>3</td>
<td>Sand</td>
<td>-</td>
<td>17</td>
<td>3</td>
</tr>
<tr>
<td></td>
<td>4</td>
<td>Sand</td>
<td>-</td>
<td>15</td>
<td>4</td>
</tr>
<tr>
<td></td>
<td>5</td>
<td>Sand</td>
<td>-</td>
<td>18</td>
<td>3</td>
</tr>
<tr>
<td></td>
<td>6</td>
<td>Sand</td>
<td>-</td>
<td>28</td>
<td>6.5</td>
</tr>
<tr>
<td>2</td>
<td>1</td>
<td>Sand</td>
<td>-</td>
<td>17</td>
<td>4</td>
</tr>
<tr>
<td></td>
<td>2</td>
<td>Sand</td>
<td>-</td>
<td>15</td>
<td>6</td>
</tr>
<tr>
<td></td>
<td>3</td>
<td>Sand</td>
<td>-</td>
<td>18</td>
<td>4</td>
</tr>
<tr>
<td></td>
<td>4</td>
<td>Sand</td>
<td>-</td>
<td>28</td>
<td>10</td>
</tr>
<tr>
<td>3</td>
<td>1</td>
<td>Soft clay</td>
<td>75</td>
<td>-</td>
<td>15</td>
</tr>
<tr>
<td></td>
<td>2</td>
<td>Stiff clay</td>
<td>144</td>
<td>-</td>
<td>9</td>
</tr>
</tbody>
</table>

The deck was modeled as an elastic beam element because flexural yield is not anticipated in the deck during a barge impact event. The cap beam was also modeled as a linear elastic beam element because yielding of the beam is not expected during a barge impact. In the current study, the columns were modeled as force-based elements with fiber sections, and the column diameter was 1.6 m. Columns were simulated as one element with five integration points to capture the axial force-moment interaction. Each column used fiber sections with Concrete01, Concrete04 and Steel02 material models in *OpenSees*. The pile cap was modeled using elastic beam elements to ensure that failure did not occur therein and the pile cap remained rigid during loading. The piles with a length of 18 m were discretized into 36 displacement beam-column elements, each with two integration points. The soil springs were modeled using zero-length elements with three translational degrees of freedom, and spring nodes were created along the embedded pile length. The soil-pile interaction was modeled as a beam on a nonlinear Winkler foundation (BNWF). The
seat-type bridge abutments consisted of a stem wall, back wall, wing wall, shear key, bearing pads, and foundation, and these elements were modeled based on their behavior.

Table 3.3 Summary of materials and elements in *OpenSees* for bridge components

<table>
<thead>
<tr>
<th>Item</th>
<th>Bridge Components</th>
<th>Material in OpenSees</th>
<th>Element in OpenSees</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Deck</td>
<td>section properties needed</td>
<td>Elastic Beam Column</td>
</tr>
<tr>
<td>2</td>
<td>Cap beam</td>
<td>section properties needed</td>
<td>Elastic Beam Column</td>
</tr>
<tr>
<td>3</td>
<td>Column</td>
<td>Uniaxial Material</td>
<td>Force Beam Column</td>
</tr>
<tr>
<td>4</td>
<td>Pile cap</td>
<td>section properties needed</td>
<td>Elastic Beam Column</td>
</tr>
<tr>
<td>5</td>
<td>Pile</td>
<td>Uniaxial Material</td>
<td>Disp Beam Column</td>
</tr>
<tr>
<td>6</td>
<td>Soil</td>
<td>Uniaxial Material</td>
<td>Zero Length</td>
</tr>
<tr>
<td>7</td>
<td>Abutment</td>
<td>section properties needed</td>
<td>Elastic Beam Column</td>
</tr>
</tbody>
</table>

3.4 **Methodology of IDA**

Consolazio and Cook (2002) performed full scale barge impact testing of the St. George Island Causeway Bridge at velocities up to 6 knots (3 m/s). However, such velocities are just for testing and considered extreme in practice. Consolazio and Cook (2002) provided many numerical examples that show the minimum boat velocity for towing or pushing is 3.376 ft/sec (1 m/sec). On the other hand, 2 m/s is a common traveling velocity for barges in U.S. waterways. Furthermore, exceeding the velocity intensity measure of 2 m/s in this IDA required a large number of time steps to achieve model convergence in the finite element simulation. Therefore, the maximum impact barge velocity that was used to perform the present IDA work was 2 m/s.

Generally, ship geometry varies, but barge shape is more consistent and uniform. Therefore, this research used a barge in order to implement the Incremental Dynamic Analysis. The barge was modeled using a lumped point mass at the same location as the barge impact point which was considered to be the location of the inertia force. The node had a mass of 1,000 metric
tons and a series of initial velocities from 0 to 2 m/sec with increments of 0.25 m/sec. The force
deformation relationship of the spring model was based on the appropriate AASHTO
recommendation. The spring material type was uniaxial material Steel01 (Figure 3.1), and it was
used to construct a uniaxial bilinear strain hardening material, with zero length element as defined
in *OpenSees*. Steel01 is symmetric in terms of tension and compression stress, but in this study,
Steel01 material was implemented only when dealing with the compression. Steel01 is defined as
a function of yield stress ($F_y$), stiffness ($E_0$), and secondary stiffness ($b \times E_0$) (hardening
coefficient). To satisfy model assumptions, the tension side of the barge spring was reached by
removing the Steel01 material and spring element from the model.

Furthermore, the barge bow yield force used was based on Kameshwar and Padgett [12].
The spring represented a bow deformation which is considered displacement-dependent (stiffness)
and connects barge mass node to the column impact point node. This study adopts the bow force
deformation relationships for spring stiffness proposed by AASHTO, using AASHTO equations
3.14.11-1 and 3.14.11-2 (equations 3.6 and 3.7 in Section 3.1.1.3 of this manuscript).

![Steel01 model in OpenSees for barge force displacement relationship.](image)

Figure 3.1 Steel01 model in *OpenSees* for barge force displacement relationship.
In addition to force-deformation relationships that were adopted in this study from AASHTO, the mass was adopted based on AASHTO definitions for two types of weight: dead weight tonnage (DWT) which includes the weight of cargo, fuel, and water where its starts to submerge a vessel from empty draft to loaded draft, and the displacement tonnage which includes both the weight of the vessel (light-weight tonnage) and the dead weight tonnage. To satisfy model assumptions, a value for the point mass was assigned based on the St. George Island Causeway Bridge report written by Consolazio and Cook [39]. As seen in Figure 3.2, the barge is modeled as a mass node and a spring with suitable stiffness. Barge yield is also specified by Consolazio et al. 2009 [16]. The above authors have demonstrated that pier shape and size affect barge-bow force-deformation characteristics. Based on that, round bridge column geometry is recommended in the UF/FDOT load-prediction model [15,18].

Figure 3.2 Sketch of finite element model for the barge-bridge column collision.
During the current modeling exercise in *OpenSees*, the barge loses contact with the bridge column when the barge spring stress is changed from compression to tension, and the barge mass and its spring are removed after five seconds. The model assumes that the barge is no longer in contact with the bridge column after the collision, and the bridge is under free vibration. The *remove node* and *remove element* commands in *OpenSees* are used to end the contact between the barge and bridge column. The above commands are used to remove components from the model, and if the stress in the spring changes from compression to tension, then the barge mass and spring effectively disappear from the model. The steps involved in the execution of IDA and the computational modeling in *OpenSees* are summarized below:

1. Add barge node at location of impact vessel with initial velocities.
2. Add compression-only zero-length element (with Steel01) representing bow stiffness.
3. Remove barge node, element and recorder for force-deformation spring after contact is lost between the bridge column and barge.
4. Free vibration phase of bridge is initiated to continue the dynamic behavior of the bridge, and this free vibration state was carried out by imposing a loading duration longer than the contact time between the barge and bridge column.
5. Duration of collision was set to 5 seconds covering four collision phases: Phase I when velocities of barge and bridge are the same at the collision moment, Phase II when the barge and the bridge decelerate up to almost zero velocity, Phase III when acceleration changes the direction for both the barge and bridge, and Phase IV when the bridge separates from the barge and the bridge goes into free vibration.
6. To complete this computational resource-intensive IDA analysis, parallel application was used for running software simulation because the availability of multi-core processors on a personal computer optimized the analysis time.

In short, the barge impact was modeled directly by barge mass, spring stiffness and bridge column associated with free vibration. This modeling procedure treats the barge mass and the spring as a dynamic load with an inertia force associated with initial velocities. The finite element model for a column and pile foundation of the geometry model of the bridge system in Figure 3.2 depicts how the bridge components were modeled in OpenSees.

### 3.5 Results of IDA

#### 3.5.1 IDA Curves for Central Deck Displacement

In Figure 3.3(a), the IDA curve shows the variation of the central deck displacement in terms of damage measure barge velocity, for five scour depths from 0 to 4 m with a barge impact height of 3 m. Under the pristine scour condition (blue dashed curve), the slope of the curve is much steeper than at a scour depth of 4 m (pink squares curve), indicating that shallower slopes correspond to increased bridge flexibility. Similarly, Figures 3.3(b) and 3.3(c) illustrate the change in IDA characteristic when the impact point is moved up to 4 m and 5 m, respectively.

#### 3.5.2 IDA Curves for Column Impact Point Displacement

In Figure 3.4, the vertical axis represents the intensity measure (IM), which is the barge velocities ranging from 0 to 2 m/sec in intervals of 0.25 m/sec. The horizontal axis represents the damage measure (DM), which is column displacement at the impact point for five incremental scour depths from 0 to 4 m with barge impact heights of 3 m, 4 m, and 5 m from the base of the column. In the absence of scouring (blue dashed curve), the slope of the curve is much steeper than at a scour depth of 4 m (pink squares curve); shallower slopes indicate increases in the unsupported
length of the column, resulting in higher column displacement due to the same changes in barge velocity. Note that the increase in barge impact height from 3 m in Figure 3.4(a) to 4 m in Figure 3.4(b) or 5 m in Figure 3.4(c) results in less column displacement at all five scour depths.

3.5.3 IDA Curves for Column Impact Point Moment

In Figures 3.5(a), (b), and (c), IDA curves show the impact of IM, barge velocity from 0 to 2 m/sec in intervals of 0.25 m/sec, on DM, column impact moment, for five incremental scour depths from 0 to 4 m for barge impact heights of 3 m, 4 m, and 5 m. Figure 3.5(a) shows that at low barge velocities less than 0.75 m/sec, scour depth does not have a significant effect on column impact moment. At higher barge velocity, column impact moment increases with scour depth.

Figure 3.5(b) shows that at barge velocities less than 1 m/sec, scour depth does not have a significant effect on column impact moment. Figure 3.5(c) shows that regardless of barge velocity, scour depth does not have a significant effect on column impact moment (i.e. column impact moment is the same at all five scour depths for any given barge velocity between 0-2 m/sec).
Figure 3.3 IDA curve for central deck displacement for five scour cases with incremental barge velocities under barge impact points of (a) 3 m, (b) 4 m, and (c) 5 m above the pile cap.
Figure 3.4 IDA curve for column impact point displacement for five scour cases with incremental barge velocities under barge impact points of (a) 3 m, (b) 4 m, and (c) 5 m above the pile cap.
Figure 3.5 IDA curve for column impact moment for five scour cases with incremental barge velocities under barge impact points of (a) 3 m, (b) 4 m, and (c) 5 m above the pile cap.
3.5.4 IDA Curves for Column Impact Point Shear

In Figures 3.6(a), (b), and (c), IDA curves show the impact of intensity measure, barge velocity from 0 to 2 m/sec at intervals of 0.25 m/sec, on damage measure, column impact shear, for five scour depths from 0 to 4 m for barge impact heights of 3 m, 4 m, and 5 m. In Figures 3.6(a) and 3.6(b), at low barge velocities less than 0.25 m/sec, scour depth does not have a significant effect on column impact shear. In Figure 3.6(c), at barge velocities less than 0.75 m/sec, scour depth does not have a significant effect on column impact shear. At barge velocities higher than 0.75 m/sec, column impact shear increases with scour depth. Unlike column impact moment, column impact shear increases with barge impact height.

3.5.5 IDA Curves for Column Impact Point Rotation

In Figures 3.7(a), (b), and (c), the IM, which is the barge velocities ranging from 0 to 2 m/sec at intervals of 0.25 m/sec, is plotted against the DM which is column rotation at the impact point for five incremental scour depths from 0 to 4 m for a barge impact height of 3 m, 4 m, and 5 m from the base of the column. Shallower slopes with increasing scour indicate that increases in the unsupported length of the column result in more column rotation due to the same changes in barge velocity. In Figure 3.7, the curves also indicate large flexibility of the columns to rotation with increasing scour. Furthermore, higher impact heights seem to decrease the extent of column rotation.
Figure 3.6 IDA curve for column impact shear for five scour cases with incremental barge velocities under barge impact points of (a) 3 m, (b) 4 m, and (c) 5 m above the pile cap.
Figure 3.7 Traditional IDA curve for column impact point rotation for five scour cases with incremental barge velocities under barge impact points of (a) 3 m, (b) 4 m, and (c) 5 m above the pile cap.
3.5.6 IDA Curves for Displacement of Pile 1

Pile 1 is in the bottom-left corner in the group of 23 piles under column 1 (Figure 2.18). This pile was selected and analyzed for displacement because of its location. Pile 1 experiences large displacements due to ship impact, and it is subjected to p-multipliers of 0.85 for the front row (closest to ship).

In Figure 3.8(a), IDA curve shows the impact of IM, barge velocity from 0 to 2 m/sec at intervals of 0.25 m/sec, on DM, Pile 1 displacement, for five incremental scour depths from 0 to 4 m for a barge impact height of 3 m. Under the pristine scour condition (blue, dashed curve), the slope of the curve is much steeper than at a scour depth of 4 m (pink squares curve) resulting in more pile displacement due to the same changes in barge velocity. Figure 3.8(b) is identical to Figure 3.8(a) with respect to the intensity and damage measures. Increasing barge impact height from 3 m in Figure 3.8(a) to 4 m in Figure 3.8(b) results in less pile displacement at all five scour depths. In Figure 3.8(c), the increase in barge impact height to 5 m results in less Pile 1 displacement at all five scour depths compared to 3 m and 4 m barge impact points.
Figure 3.8 (a), (b), and (c) IDA curve for maximum absolute displacement of Pile 1 for five scour cases with incremental barge velocities under barge impact points of (a) 3 m, (b) 4 m, and (c) 5 m above the pile cap.
3.5.7 IDA Curves for Axial Force on Pile 1

In Figure 3.9, the vertical axis represents the IM, which is the barge velocity ranging from 0 to 2 m/sec at intervals of 0.25 m/sec. The horizontal axis represents the DM which is the maximum absolute axial force applied to Pile 1 for five incremental scour depths from 0 to 4 m for a barge impact height of 3 m from the base of the column. In the absence of scouring (blue, dashed curve), the slope of the curve is much steeper than at a scour depth of 4 m (pink squares curve); shallower slopes indicate increases in the unsupported length of the pile, resulting in greater axial force due to the same changes in barge velocity. Furthermore, the IDA figure did not display any significant effect of scour when the barge velocity was equal to or less than 0.25 m/sec. Figure 3.9(b) is identical to Figure 3.9(a) with respect to the intensity and damage measures. Increasing barge impact height from 3 m in Figure 3.9(a) to 4 m in Figure 3.9(b) results in reduced Pile 1 axial force at all five scour depths. In Figure 3.9(c), the increase in barge impact height to 5 m results in less axial force in Pile 1 at all five scour depths compared to 3 m and 4 m barge impact points. The axial force in Pile 1 was greater when scour depth was higher.
Figure 3.9 (a), (b), and (c) IDA curve for maximum absolute axial force in Pile 1 for five scour cases with incremental barge velocities under barge impact points of (a) 3 m, (b) 4 m, and (c) 5 m above the pile cap.
3.5.8 IDA Curves for Moment in Pile 1

In Figure 3.10(a), the IDA curve shows the impact of IM, barge velocity, on DM, maximum absolute moment of Pile 1, for five incremental scour depths from 0 to 4 m for a barge impact height of 3 m. At low barge velocities less than 1 m/sec, scour depth does not have a significant effect on maximum absolute moment of Pile 1. At higher barge velocities, moment of Pile 1 increases with scour depth. Figure 3.10(b) is identical to Figure 3.10(a) with respect to the intensity and damage measures. Increasing the barge impact height from 3 m in Figure 3.10(a) to 4 m in Figure 3.10(b) results in reduced maximum absolute moment of Pile 1 at all five scour depths. At barge velocities less than 1 m/sec, scour depth does not have a significant effect on moment of Pile 1. At barge velocities higher than 1 m/sec, the moment of Pile 1 increases with scour depth.

In Figure 3.10(c), the increase in barge impact height to 5 m results in less moment of Pile 1 at all five scour depths compared to 3 m and 4 m barge impact points. The moment in Pile 1 was greater when scour depth was higher. At barge impact height of 5m, the moment of Pile 1 is lower compared to that of barge impact heights of 3 m and 4 m. Regardless of barge velocity, scour depth does not have a significant effect on moment of Pile 1 for given barge velocity of 1 m/sec or less.
Figure 3.10 (a), (b), and (c) IDA curve for maximum absolute moment (DM) of Pile 1 for five scour cases with incremental barge velocities under barge impact points of (a) 3 m, (b) 4 m, and (c) 5 m above the pile cap.
3.5.9 IDA Curves for Shear Force of Pile 1

In Figure 3.11(a), IDA curves show the impact of IM, barge velocity from 0 to 2 m/sec at intervals of 0.25 m/sec, on DM, shear of Pile 1, for five incremental scour depths from 0 to 4 m for a barge impact height of 3 m. At barge velocities less than 0.25 m/sec, scour depth does not have a significant effect on shear of Pile 1. At higher barge velocity, shear of Pile 1 increases with scour depth. In Figure 3.11(b), IDA curve showing the impact of intensity measure, barge velocity from 0 to 2 m/sec at intervals of 0.25 m/sec, on damage measure, shear of Pile 1, for five incremental scour depths from 0 to 4 m for a barge impact height of 4 m. At barge velocities less than 0.25 m/sec, scour depth does not have a significant effect on shear of Pile 1. At barge velocities higher than 1 m/sec, shear of Pile 1 increases with scour depth. In Figure 3.11(c), IDA curve showing the impact of intensity measure, barge velocity from 0 to 2 m/sec in intervals of 0.25 m/sec, on damage measure, shear of Pile 1, for five incremental scour depths from 0 to 4 m for a barge impact height of 5 m. At barge velocities less than 0.75 m/sec, scour depth does not have a significant effect on shear of Pile 1. At barge velocity higher than 0.75 m/sec, column impact shear increases with scour depth. Unlike column impact moment, shear of Pile 1 increases with barge impact height.
Figure 3.11 (a), (b), and (c) IDA curve for maximum absolute shear of Pile 1 for five scour cases with incremental barge velocities under barge impact points of (a) 3 m, (b) 4 m, and (c) 5 m above the pile cap.
3.6 Discussion of IDA

In this work, the impacting barge is modeled with a compression spring, and inertia force is applied during impact and its free vibration for a duration of five seconds. This paper shows four steps in order to implement IDA curves:

1. Determine the intensity and damage measures for scoured bridge under vessel collision.
2. Decide on the analysis approach. This study used a stepping approach by increasing the barge velocity, and implemented the approach with OpenSees to run the IDAs.
3. Decide on the collapse conditions. This work used a non-simulated collapse, which is an artificial limit placed on the model to prevent unrealistic structure responses.
4. Set up post-processing to plot the IDAs curves.

The interaction between the barge and bridge is modeled by introducing a node mass for the barge and the spring element between the barge and bridge column. With small values of velocities less than 0.75 m/sec, present scour conditions could not significantly influence column impact responses. The results show the IDA curves for dual hazards, scour and vessel collision, do not include an ultimate flat line, as usually shown in the IDA curves for seismic loads, because of the nature of inertia forces and model assumptions such as bilinear strain hardening spring. The limitation of this study is its novelty; it is difficult to compare the results because only IDA curves for seismic loads are currently available and not for other extreme loads such as vessel collision loads.

3.7 Theoretical Fragility Function

Equation 3.12 is used to define the fragility function and a maximum likelihood approach was adopted to estimate the parameters.
where \( F_j \) is the fragility function for the \( j^{th} \) damage state as a function of the velocity or mass of barge intensity measure, \( IM \), and the scour depth, \( S_c \); \( c_j \) and \( \zeta_j \) are the median and log-standard deviation of the fragility function; \( \Phi \) is the cumulative normal distribution function.

Two methods could be used to formulate the likelihood functions, as shown in equations 3.13(a) and 3.13(b). The difference between the two methods lies in the estimation of the log-standard deviation for the damage states. In Method 1, the log-standard deviation for each damage state is estimated independently, whereas Method 2 assumes the same log-standard deviation for all damage states. For the two methods, the parameters are obtained by setting the Jacobian of the natural log of the likelihood function, \( L \), to zero (equation 3.14).

Method 1: \[ L = \prod_i^k \left[ F(IM) \right]^{x_i} \left[ 1 - F(IM) \right]^{1-x_i} \] \hspace{1cm} (3.13a)

Method 2: \[ L = \prod_{j=1}^k \prod_{i=1}^n \left[ P_j(IM_i; E_k) \right]^{x_i} \] \hspace{1cm} (3.13b)

\[ \frac{\partial \ln L}{\partial v_j} = 0 \] \hspace{1cm} (3.14)

where \( E_k \) is the damage state; \( P_j(IM_i; E_k) \) is the probability that a bridge will suffer a damage \( j \) under the intensity measure \( i \); \( v_j \) represents the variables \( c_j \) and \( \zeta_j \) and \( x_i = 1 \) if bridge sustains damage state \( j \) under a \( IM_i \) and \( x_i = 0 \) otherwise. Method 2 is used in this paper to develop the likelihood function.

The analytical fragility function parameters in equation 3.12 were estimated from IDA results to determine the IM value at which each velocity or mass reaches the given damage states. The probability of exceeding damage states was estimated under any given barge mass/velocity and scour.
The intensity measures considered in this paper are the barge velocity (m/sec) and mass (tons). For the bridge, three levels of damage states (DS) or limit states (LS) were considered: minor (minor spalling, IO), moderate (column structurally sound, LS), and major (column structural unsafe, CP). The minor, moderate, and major damage levels correspond to 1%, 2%, and 4% drift ratios at column impact point, respectively. The fragility curves are developed for predicting the vulnerability of the critical components such as columns to finally assess the damageability of a bridge or another structure. Moreover, the fragility curves can be utilized to estimate the losses, select the suitable retrofitting techniques, and provide help for decision making with respect to design.

3.8 Methodology of Fragility Function

The University of Toledo has developed a tool for creating the fragility functions under the supervision of Dr. Gunner, http://www.utoledo.edu/engineering/faculty/serhan [21]. The purpose of this tool is to transfer IDA results into the probability of exceeding limit states. User must fill the intensity measures, engineering demand parameters (EDP), and the performance limits tables before proceeding with the fragility generator. The above tool could be generalized for any type of extreme events such as earthquakes, wind, tsunamis, vessel collisions, etc. The outputs of the tool are standard deviation and log standard deviation of drift ratio. Thus, these steps lead to the production the fragility curves of all damage state cases.

3.9 Discussion of Fragility Analysis Results

Figure 3.12 shows the fragility curves of the bridge column using the barge mass as the intensity measure (IM). As it can be seen from Figure 3.12, the probability of exceeding a specific damage state increases when a larger scour depth is recorded. Figure 3.12(a) displays the probability of exceedance of various damage states for the bridge column in its pristine condition.
Furthermore, it is clear that the column experiences only minor and moderate damages with no scour. With the development of the flood-induced scour at the pier foundation, the probability of exceeding the minor and moderate damages increases (Figure 3.12(b), (c), and (d)). The added flexibility in the system brought about by the erosion of soil sediments around the bridge foundation results in the bridge column experiencing up to 30% probability of reaching the major damage limit state. For a barge mass of 2000 metric tons, the probability of exceedance of major damage states of the column are 3%, 13%, and 30% for scour depths of 1.0 m, 2.0 m, and 3.0 m, respectively.

In Figure 3.13, fragility curves are defined as a relationship between barge velocity and the probability of exceeding minor, moderate, and major performance levels. Once again, as the scour depth increases the probability of exceeding the damage levels also increase. Furthermore, for all scour cases considered, the bridge column experiences minor, moderate, and major damages. For barge velocity of 2.0 m/s, the probability of exceedance of bridge column major damage states are 10%, 20%, 35%, and 65% for scour depth of 0.0 m, 1.0 m, 2.0 m, and 3.0 m, respectively. The constructed fragility curves show that the presence of scour at the bridge foundation negatively influences the failure probability of the column.
Figure 3.12 Fragility curves of drift ratio of bridge columns as functions of mass under (a) 0 m (b) 1 m (c) 2 m (d) and 3 m scour depths.
Figure 3.13 Fragility curves of drift ratio of bridge columns as functions of barge velocity under (a) 0 m (b) 1 m (c) 2 m (d) and 3 m scour depths.
3.10 Generalization of IDA Results

IDA results were generalized based on a parametric study involving two parameters: 1) soil configuration and 2) bridge column configuration. Thus, this study was expanded to consider a total of 3 types of bridge column configurations subjected to a simulated barge collision with a lumped point mass associated with velocity ranging from 0 m/sec to 2 m/sec with increment of 0.25 m/sec, 3 different soil configurations, 5 different scour depths, and 3 different barge impact points of 3 m, 4 m, and 5 m. Hence, 1080 finite element simulations were performed in all. In each case, IDA is performed to assess the shear force, moment, axial force, and displacement in the bridge column for three soil configurations and for three bridge configurations 4, 5, 6. Figures below illustrate the differences in scoured bridge response due to variations in the above parameters with respect to bridge column configurations.

3.10.1 Generalized IDA Curves under Different Soil Configurations

At the bridge foundation, the soil profile 1 consists of six layers of varying materials. The first layer is soft clay, the second is stiff clay, and third through sixth layers are sand. Each layer has assigned soil parameters such as undrained shear strength (cohesion) for clay and soil friction angle, derived from SPT values, for sand. The first soil layer starts at the mudline and is located at the top of the pile cap. The topmost layer is soft clay (undrained shear strength of 75 kPa) and the second layer from top is stiff clay (undrained shear strength of 144 kPa). The other layers 3, 4, 5 and 6 consisting of cohesionless soils, have SPT values of 17, 15, 18 and 28, respectively. Moreover, soil configuration 2 is only sand, and this configuration made of four layers having SPT values of 17, 15, 18 and 28 from top to bottom, respectively. Soil configuration 3 is only clay, and it is composed of two layers of clay having soft clay (undrained shear strength of 75 kPa) in the topmost layer and stiff clay (undrained shear strength of 144 kPa) in the second layer.
Case 1: Displacement of central deck point was evaluated for different soil configurations. Figure 3.14 illustrates that the soil configuration 3 type (clay) produces a slightly larger central deck displacement for all scour depth cases with the bridge configuration 5 (end span 30 m and column height 10 m) under a barge impact at a height of 5 m.

Case 2: Displacement of column-barge impact point was investigated for three different soil configurations. Figure 3.15 illustrates that the soil configuration 3 (clay) produces a slightly larger barge impact point displacement with the bridge configuration 5 under a barge impact at a height of 5 m. The column impact point displacement was largest under the largest scour depth of 4 m.

Case 3: Column impact moment was examined at three soil configurations. Figure 3.16 depicts that the soil configuration 3 (clay) produces the highest moment demand under all the scour conditions with bridge configuration 5 under a barge impact at a height of 5 m.

Case 4: Column impact shear was tested at three soil configurations. Figure 3.17 depicts that the soil configuration 3 (clay) produces the highest shear demand under all the scour conditions with bridge configuration 5 under a barge impact at a height of 5 m.

Case 5: Column impact rotation was studied for three soil configurations. Figure 3.18 depicts that the soil configuration 3 (clay) produces the highest absolute maximum rotation demand under all the scour conditions with bridge configuration 5 under a barge impact at a height of 5 m.

Case 6: The influence of three soil configurations on the displacement response of Pile 1 associated with the bridge configuration 5 was investigated for scour depths of 0 m, 2 m, and 4 m and a barge impact at a height of 5 m as seen in Figure 3.19. It is observed that Pile 1 experiences the largest maximum displacement with the soil configuration 3 (clay).
• Case 7: The variation of axial force developed in Pile 1 with barge velocity due to a barge impact at a height of 5 m with and without scour for different types of soil configurations is shown in Figure 3.20. It is illustrated that Pile 1 experiences the highest axial force at scour depths of 2 m and 4 m for the soil configuration 3 (clay). When the scour depth is 0 m, Pile 1 in the soil profile 2 experiences the highest axial force.

• Case 8: The variation of moment developed in Pile 1 with barge velocity due to a barge impact at a height of 3 m with and without scour for different types of soil configurations is shown in Figure 3.21. It is demonstrated that Pile 1 experiences the highest moment at scour depths of 0 m and 2 m for the soil configuration 3 and the highest moment at the scour depth of 4 m for the soil configuration 1.

• Case 9: The variation of shear force developed in Pile 1 with barge velocity due to a barge impact at a height of 5 m with and without scour for different types of soil configurations is shown in Figure 3.22. It is illustrated that Pile 1 experiences the highest shear at scour depths of 0 m and 2 m for the soil configuration 2 (sand) and the highest shear at the scour depth of 4 m for the soil configuration 1.
Figure 3.14 IDA curves for displacement of deck with three soil configurations and bridge configuration 5 at an impact point height of 5 m, and scour depths of (a) 0 m, (b) 2 m, and (c) 4 m.
Figure 3.15 IDA curves for displacement of bridge column with three soil configurations and bridge configuration 5 at an impact point height of 5 m, and scour depths of (a) 0 m, (b) 2 m, and (c) 4 m.
Figure 3.16 IDA curves for impact point moment of column with three soil configurations and bridge configuration 5 at an impact point height of 5 m, and scour depths of (a) 0 m, (b) 2 m, and (c) 4 m.
Figure 3.17 IDA curves for impact point shear of column with three soil configurations and bridge configuration 5 at an impact point height of 5 m, and scour depths of (a) 0 m, (b) 2 m, and (c) 4 m.
Figure 3.18 IDA curves for impact point rotation of column with three soil configurations and bridge configuration 5 at an impact point height of 5 m, and scour depths of (a) 0 m, (b) 2 m, and (c) 4 m.
Figure 3.19 IDA curves for displacement of Pile 1 with three soil configurations and bridge configuration 5 at an impact point height of 5 m, and scour depths of (a) 0 m, (b) 2 m, and (c) 4 m.
Figure 3.20 IDA curves for axial force of Pile 1 with three soil configurations and bridge configuration 5 at an impact point height of 5 m, and scour depths of (a) 0 m, (b) 2 m, and (c) 4 m.
Figure 3.21 IDA curves for moment of Pile 1 with three soil configurations and bridge configuration 5 at an impact point height of 5 m, and scour depths of (a) 0 m, (b) 2 m, and (c) 4 m.
Figure 3.22 IDA curves for shear of Pile 1 with three soil configurations and bridge configuration 5 at an impact point height of 5 m, and scour depths of (a) 0 m, (b) 2 m, and (c) 4 m.
3.10.2 Generalized IDA Curves under Different Bridge Column Configurations

In this phase of IDA generalization, three bridge column configurations (4, 5, and 6) were considered as seen in Table 3.4. When producing IDA curves, all parameters were kept constant except the heights of columns. All of them have the same variables: end span (30 m) and middle span (36 m), column diameter (1.6 m), main longitudinal rebar (50) #11, and size of piles (0.4 m). However, these configurations differ in column height (7.5 m, 10 m, and 12.5 m, respectively). The objective was to evaluate the influence of the column length to diameter ratio the column shear, moment, and displacement produced during barge impact. When examining IDA generalization for these three bridge column configurations, the barge impact point height was kept at 3 m, the barge velocity was varied in 0.25 m/sec increments from 0 m/s to 1.0 m/s, and only soil profile 1 was used (Figure 3.23).

- Case 10: Displacement of central deck point was evaluated for three bridge column configurations. Figure 3.24 illustrates that the bridge column configuration 4 experiences larger central deck displacement for all scour depth cases under a barge impact at height of 3 m. There is an inverse relationship between deck displacement and the column height.

- Case 11: Displacement of column at impact point was evaluated for three bridge column configurations. Figure 3.25 illustrates that the bridge column configuration 6 experiences larger column impact point displacement for all scour depth cases under a barge impact height of 3 m. There is a direct relationship between column displacement and the column height.

- Case 12: Moment of column at impact point was evaluated for three bridge column configurations. Figure 3.26 illustrates that the bridge column configuration 6 experiences larger column impact point moment for all scour depth cases under a barge impact at a height of 3 m. There is a direct relationship between column moment and the column height.
• Case 13: Shear of column at impact point was evaluated for three bridge column configurations. Figure 3.27 illustrates that the bridge column configuration 4 experiences slightly larger column impact point shear for all scour depth cases under a barge impact at a height of 3 m. There is an inverse relationship between column shear and the column height.

• Case 14: Rotation of column at impact point was evaluated for three bridge column configurations. Figure 3.28 illustrates that the bridge column configuration 4 experiences smaller column impact point rotation for all scour depths under a barge impact at a height of 3 m. There is direct relationship between column rotation and the column height.

• Case 15: Maximum displacement of Pile 1 was evaluated for three bridge column configurations. Figure 3.29 illustrates that Pile 1 of the bridge column configuration 4 experiences lower displacement for all scour depth cases under a barge impact at a height of 3 m. There is direct relationship between displacement of Pile 1 and the column height.

• Case 16: Axial force of Pile 1 was evaluated for three bridge column configurations. Figure 3.30 illustrates that the bridge column configuration 4 experiences smaller axial force for all scour depth cases under a barge impact at a height of 3 m. There is direct relationship between axial force of Pile 1 and the column height.

• Case 17: Moment of Pile 1 was evaluated for three bridge column configurations. Figure 3.31 illustrates that the bridge column configuration 6 experiences larger moment of Pile 1 for all scour depth cases under a barge impact at a height of 3 m. There is direct relationship between moment of Pile 1 and the column height.
Case 18: Shear of Pile 1 was evaluated for three bridge column configurations. Figure 3.32 illustrates that the bridge column configuration 6 experiences larger shear of Pile 1 for all scour depth cases under a barge impact at a height of 3 m. There is direct relationship between shear of Pile 1 and the column height.

Table 3.4 Bridge column configurations of generalized IDA

<table>
<thead>
<tr>
<th>Bridge Config.</th>
<th>End Span (m)</th>
<th>Middle Span (m)</th>
<th>H_col (m)</th>
<th>D_col (m)</th>
<th>Nbars</th>
<th>lbar</th>
<th>P_dia (m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>4</td>
<td>30</td>
<td>36</td>
<td>7.5</td>
<td>1.6</td>
<td>50</td>
<td>11</td>
<td>0.4</td>
</tr>
<tr>
<td>5</td>
<td>30</td>
<td>36</td>
<td>10</td>
<td>1.6</td>
<td>50</td>
<td>11</td>
<td>0.4</td>
</tr>
<tr>
<td>6</td>
<td>30</td>
<td>36</td>
<td>12.5</td>
<td>1.6</td>
<td>50</td>
<td>11</td>
<td>0.4</td>
</tr>
</tbody>
</table>
Figure 3.23 Soil profile 1.
Figure 3.24 IDA curves for displacement of deck with three bridge column configurations and soil configuration 1 at an impact point height of 3 m, and scour depths of (a) 0 m, (b) 2 m, and (c) 4 m.
Figure 3.25 IDA curves for displacement of bridge column with three bridge column configurations and soil configuration 1 at an impact point height of 3 m, and scour depths of (a) 0 m, (b) 2 m, and (c) 4 m.
Figure 3.26 IDA curves for moment of bridge column with three bridge column configurations and soil configuration 1 at an impact point height of 3 m, and scour depths of (a) 0 m, (b) 2 m, and (c) 4 m.
Figure 3.27 IDA curves for shear of bridge column with three bridge column configurations and soil configuration 1 at an impact point height of 3 m, and scour depths of (a) 0 m, (b) 2 m, and (c) 4 m.
Figure 3.28 IDA curves for rotation of bridge column with three bridge column configurations and soil configuration 1 at an impact point height of 3 m, and scour depths of (a) 0 m, (b) 2 m, and (c) 4 m.
Figure 3.29 IDA curves for displacement of Pile 1 with three bridge column configurations and soil configuration 1 at an impact point height of 3 m, and scour depths of (a) 0 m, (b) 2 m, and (c) 4 m.
Figure 3.30 IDA curves for axial force of Pile 1 with three bridge column configurations and soil configuration 1 at an impact point height of 3 m, and scour depths of (a) 0 m, (b) 2 m, and (c) 4 m.
Figure 3.31 IDA curves for moment of Pile 1 with three bridge column configurations and soil configuration 1 at an impact point height of 3 m, and scour depths of (a) 0 m, (b) 2 m, and (c) 4 m.
Figure 3.32 IDA curves for shear of Pile 1 with three bridge column configurations and soil configuration 1 at an impact point height of 3 m, and scour depths of (a) 0 m, (b) 2 m, and (c) 4 m.
Chapter 4: Conclusion

In the first phase of this study, ship impact on a bridge with a scoured foundation was investigated using a dynamic load with a designated time history instead of an equivalent static load that is employed in the current practice. In order to achieve the above objective, a 3D finite element program (OpenSees) was used to model the superstructures and the substructures of a number of model bridge configurations supported by a pile foundation in different soil types. The model was utilized to perform gravity, pushover, modal, moment curvature, and dynamic analyses for the reinforced concrete box girder of the bridges while considering the soil-structure interaction at the pile foundations. Eight bridge scour cases under five ship impact heights along the bridge columns were investigated in order to evaluate the vulnerability of the bridges.

Inclusion of the pristine case in the analysis enabled the comparison of ship impact under scour to corresponding conditions with that of no scour. It was seen that the bridge response changes dramatically because of the co-occurrence of ship impact and scour. The following specific conclusions can also be drawn from the results of the above initial study.

1. According to modal analysis results, the natural period of mode 1 of the scoured bridge modeled in this study increased almost 200% at the maximum scour (7 m) compared to the pristine scour case of mode 1.

2. The elevation of the ship impact point was seen to increase the transverse displacement of the mid-span of the deck. This observation verifies increased bridge flexibility due to scour. For example, the displacement at the ship impact point of 7 m is increased 1.4 times compared to that at a ship impact point of 3 m.
3. It was seen that the shear demand of the column base decreases due to increased scour, especially at lower impact locations.

4. The results show that presence of scour increases the moment demand on the bridge column. This means that the size of the column and current provisions of rebar may not be sufficient to sustain the two hazards at the same time. Parametric studies show that this can be addressed by designing larger columns sizes.

5. An increase in the scour depth was observed to increase the base moment demand on the bridge column, particularly for lower impact locations such as 3 m. For the bridge model used in this study, at a ship impact point of 3 m, the moment is 4,100 kN when scour is 0 m while it was 14,540 kN when the scour is 7 m, representing a 255% increase.

6. The results and further parametric studies show larger displacement in piles under increased scour, especially for clayey foundations.

7. There is a significant increase in the pile axial force demand under increased scour. For instance, the maximum absolute axial force at ship impact of 3 m of Pile 11 was 300 kN for 0 m scour depth, while it was 675 kN for scour depth of 7 m.

8. Evaluation of the nonlinear soil spring force showed that the spring force demand increases with scour with the potential for inducing soil yielding.

9. Parametric studies showed that the risk of excessive deck displacement due to expected foundation scour can be reduced by strengthening the pile foundation in proportion to the column height.

The column failure is commonly used as the controlling component when assessing bridge collapse because columns are directly subjected to barge collision. In addition, the design philosophy of bridges against barge impact is for failure to occur in columns. Therefore, the failure
probability presented in the form of fragility curves was evaluated at the bridge columns. Three pre-determined damage states, minor, moderate, and major, were utilized to correlate the damageability of the bridge column in response to the intensity measures, barge mass and velocity. The fragility analysis based on the above intensity measures indicates that in its intact condition, the exceedance probability of failure of the bridge column at the impact location increases for all damage states with the increase in intensity measure. Also, the probability of exceeding a specific damage state increases when a larger scour depth is recorded. It was clear that the column experiences only minor and moderate damages with no scour. For a barge mass of 2000 metric tons, the probabilities of exceedance of major damage states of the column at the impact point location of the studied bridge configuration are 3%, 13%, and 30% for scour depths of 1.0 m, 2.0 m, and 3.0 m, respectively. On the other hand, for a barge velocity of 2.0 m/s, the probabilities of exceedance of major damage states of the same bridge column are 10%, 20%, 35%, and 65% for scour depth of 0.0 m, 1.0 m, 2.0 m, and 3.0 m, respectively.

In the second phase of this study, Incremental Dynamic Analysis (IDA) was applied to study barge impact on a bridge with scoured foundation. The intensity measure component of the IDA curves was the barge velocity, and the damage measure was the bridge component responses. The above study fills the gap of knowledge in the application of nonlinear dynamic analysis such as IDA under extreme events other than seismic loads. The outcome of this IDA application demonstrates that the method can be a valuable tool to predict bridge component behavior under the above hazards. Furthermore, the IDA tool was also able to respond to the research need to estimate the performance of important individual bridge components under extreme events such as foundation scour and vessel collision. Although, this research has focused on the column
responses of bridge only, it can also be used to cover the individual IDA of a variety of bridge components.

On generalization of IDA results under different soil configurations, it was concluded that for all scour cases under a barge impact at a height of 5 m, clay foundation produces larger central deck displacement. With respect to the column at the impact point clay foundations produced the largest displacement, moment, shear, and rotation demands. In addition, for clay foundations, the displacement and axial force of Pile 1 are the highest. For a scour of 4 m, soil configuration 1 (sand and clay) produces larger shear force and moment demands for Pile 1. This shows that the clay foundations provide better confinement to bridges foundation during dual hazard scour and barge impact.

In the IDA parametric study involving different bridge column configurations, it was concluded that for all scour cases under a barge impact at a height of 3 m, bridge configuration 4 (column length 7.5 m) produces a larger central deck displacement. The reason is that the barge impact location is 4 m from the deck while this distance in the bridge column configurations 5 and 6 are 7 m and 9.5 m, respectively. In addition, bridge configuration 4 produces higher shear of column at impact point. Furthermore, bridge configuration 6 produces larger displacement, moment, and rotation of bridge column at the impact point. These responses are known to be influenced by the column lengths which justify the increase of the observed demands of larger displacement, moment, and rotation. With respect to Pile 1, bridge column configuration 6 produces larger displacement and moment due to unsupported length of column.

Based on the above conclusions, engineers should be able to use the results of this study to predict the column, deck, piles, and soil responses such as displacement, rotation, shear, moment, and axial forces under the above load scenarios.
Finally, the vulnerability of the bridge to multiple hazardous events was evaluated through analytical fragility functions. With respect to the analysis based on the fragility function, the scour depth leads to an increase in the probability of exceedance of all damage levels that contribute to large deformations in the bridge column.

One limitation is the oversimplified bridge components in the OpenSees model. In addition, the study can be further generalized using extended parametric studies, including different pile lengths, number of columns per bent, soil layer arrangements, etc.
References


Appendix A: OpenSees Analysis Main File

# Bridge analysis in OpenSees
#---------------------------------------------
# General Analysis File (Must be called from Dynamic.tcl, Pushover.tcl, or ModalAnalysis.tcl)
#-----------------------------------------------
# Source general files
source debug.tcl
# Redefine analysis command (for display)
wipe
if {[info commands real_analyze] eq ""} {
rename analyze real_analyze
proc analyze {nSteps args} {
    global ship_in_contact ship_recorders
    for {set i 0} {$i < $nSteps} {incr i} {
        update
        set res [real_analyze 1 {*}$args]
        if {$res != 0} {
            break
        } else {
            if {$ship_in_contact} {
                if {[eleResponse 1235 material 1 stress] > 0} {
                    remove node 1235
                    remove element 1235
                    foreach recorderTag $ship_recorders {
                        remove recorder $recorderTag
                    }
                    set ship_in_contact false
                    puts "REMOVED SHIP/BARGE at T = [getTime]"
                }
            }
        }
    }
    return $res
}
}
set ship_in_contact false
# Procedure to set default values
proc default {name value} {upvar $name var; if {[info exists var]} {set var $value}}
# General variables
default AnalysisType Dynamic; # Options: Pushover, Dynamic, ModalAnalysis, MomentCurvature
set DoMomentCurvature true; # set to false to turn off
set pi [expr acos(-1.0)]
# Define default dynamic settings
default ShipVel 1.00
# Define default ship height settings
default ShipHeight 3; # Height of impact node
# Define full parameter lists
set BridgeConfigs {5}
set SoilConfigs {1 2 3}
set ScourDepths {0 1 2 3 4 5 6 7}
# Override of parameter lists: (uncomment the lines below)
set BridgeConfigs {5}
set SoilConfigs {1}
set ScourDepths {0 1 2 3 4}
# Read bridge parameters from CSV file
set fid [open BridgeConfigurations.csv r]
set lines [lmap line [read -nonewline $fid] {
    set line [split $line ,];
    if {[lindex $line 0] eq {}} {
        continue
    } else {
        set line
    }
}]
set BridgeVars [lindex $lines 0]
# This loop loops through all the bridges, unless specified
foreach BridgeValues [lrange $lines 1 end] {
    foreach var $BridgeVars value $BridgeValues {set $var $value}
    if {$BridgeConfig ni $BridgeConfigs} {
        continue
    }
    # Print bridge info
    pvar BridgeConfig
    pvar End_Span
    pvar H_col
    puts ------
    set dataDir1 [file join OpenSeesOutput BridgeConfig=$BridgeConfig]
    foreach SoilConfig $SoilConfigs {
        pvar SoilConfig
        set dataDir2 [file join $dataDir1 SoilConfig=$SoilConfig]
        foreach scr $ScourDepths {
            pvar scr
            set dataDir3 [file join $dataDir2 Scr=$scr]
            # Define final data directory and subfolders
switch $AnalysisType {  
  Dynamic {  
    set dataDir [file join $dataDir3 Dynamic ShipHeight=$ShipHeight ShipVel=$ShipVel]  
    file mkdir $dataDir/01DeckDispN19  
    file mkdir $dataDir/02ColumnDispN1  
    file mkdir $dataDir/03Element400Deformation  
    file mkdir $dataDir/04PileDisp  
    file mkdir $dataDir/05PileForce  
    file mkdir $dataDir/06BaseShearColumn  
    file mkdir $dataDir/07MomCurvature  
    file mkdir $dataDir/08ColumnDrift  
    file mkdir $dataDir/09ColumnEnvelope  
    file mkdir $dataDir/10BaseShearBase1  
    file mkdir $dataDir/13ShipDispN1234  
    file mkdir $dataDir/14SoilDisp  
    file mkdir $dataDir/15SoilForce  
    file mkdir $dataDir/17Abutment  
    file mkdir $dataDir/18Barge  
    logFile $dataDir/log.txt  
  }  
  Pushover {  
    set dataDir [file join $dataDir3 PushOverDispCont ShipHeight=$ShipHeight]  
    file mkdir $dataDir/11PushOverDispShipNode  
  }  
  ModalAnalysis {  
    set dataDir [file join $dataDir3 ModalAnalysis]  
    file mkdir $dataDir/16ModeShape  
  }  
}; # switch for analysis type  
# Build model  
puts "Building Model"  
switch $End_Span {  
  15 {source ShortSpan.tcl}  
  30 {source MediumSplitPMSoil.tcl}  
  45 {source LongSpan.tcl}  
}  
pvar ShipHeight  
puts [format "Node 1234 Coordinates: X = %.3f, Y = %.3f, Z = %.3f" {*}[nodeCoord 1234]]  
# Set gravity loads as constant  
loadConst -time 0.0  
wipeAnalysis; # clear analysis objects  
puts "Gravity Loads Applied"  
# Do specified analysis  
switch $AnalysisType {
Dynamic {
    # Define damping
    set omega [lmap [lindex $omega 0] \{sqrt($lam)\}]; # Get eigen values and convert to frequency
    set DampRatio 0.05; # Unreinforced masonry 3% Reinforced masonry 7% code specify 5% Source Newmark recommended damping value
    set Omegal [lindex $omega 0]; # perform eigen first; eigen is performed already in medium file
    set Omega2 [lindex $omega 5]; # lindex: Retrieve an element from a list
    set eta [expr 2*$DampRatio/($Omegal+$Omega2)]; #betaKcomm; Committed stiffness damping
    set alpha [expr 2*$DampRatio*$Omegal*$Omega2/($Omegal+$Omega2)]
    set gamma 0.5;
    set beta 0.25;
    rayleigh $alpha 0.0 0.0 $eta; # rayleigh $alphaM $betaK $betaKinit $betaKcomm
    set ShipMass 1000; # ship mass default 1,000 George bridge (in Mg or metric tonnes)
    # node $nodeTag (ndm $coords) -mass (ndf $massValues) -vel (ndf $velValues)
    node 1235 {[*]nodeCoord $ImpactNode -mass 0.0 0.0 $ShipMass 0.0 0.0 0.0 -vel 0.0 0.0 $ShipVel 0.0 0.0 0.0 -disp {[*]}nodeDisp $ImpactNode}
    fix 1235 1 1 0 1 1 1
    set E 60000.0; # from AASHTO 3.14.11-1(Padgett & Consolazio)
    set Fy 5000; # average from padgett Table 1(kN), Response and fragility assessment of bridge columns
    set b 0.01; # post yield stiffness of ship (barge)
    uniaxialMaterial Steel01 1235 $Fy $E $b
    element zeroLength 1235 1235 1234 -mat 1235 -dir 3
    set ship_in_contact true; # Triggers check in analyze
    lappend ship_recorders ""
    lappend ship_recorders [recorder Element -file $dataDir/18Barge/Force.out -ele 1235 localForce]
    lappend ship_recorders [recorder Element -file $dataDir/18Barge/Deformation.out -ele 1235 deformation]
    # Define Recorders
    source RecordsDynamic.tcl
    record; # record to recorder @ t=0 before analyze
    # Dynamic settings
    test EnergyIncr 1.0e-6 10 0
    integrator HHT 0.7
    algorithm KrylovNewton
    numberer RCM; # Reverse Cuthill-McKee method will output a warning when the structure is disconnected
**constraints** Transformation;  

# Transformation method is recommended for a transient analysis

**system** ProfileSPD;  

# ProfileSPD system is how to store & solve the system of equations in the analysis (provide solution of Ku=P)

**analysis** Transient

**puts** "Starting Ship Impact Analysis"
**set** startT [clock seconds]
**set** duration 5
**set** THdt 0.1
**set** THnpt [expr {int($duration/$THdt)}]
**set** tFinal [expr $THnpt*$THdt]
**set** res [analyze $THnpt $THdt]
**source** TransientConverg.tcl
**set** endT [clock seconds]

**puts** "Execution time: [expr $endT-$startT] seconds."


{Pushover}

**set** Dincr [expr 0.00001*$P_Length];  

# Displacement increment

**set** maxD 0.15;  

# Maximum displacement Choose 0.3 in order to see the difference in capacity for each case

**timeSeries** Linear 2

**pattern** Plain 2 2 {

**load** 1234 0.0 0.0 1 0 0 0
}

**source** RecordersPushover.tcl

**record**

# Start modification for pushover analysis

**set** Nsteps [expr int($maxD/$Dincr)]

**set** IDctrlNode 1234;  

# node # @ top of column

**set** IDctrlDOF 3;  

# dof in z-direction

# Create integration scheme

**system** BandGeneral

**numberer** RCM

**constraints** Transformation

**integrator** DisplacementControl $IDctrlNode $IDctrlDOF $Dincr 1

$Dincr $Dincr

**test** NormDispIncr 1.0e-3 100

**algorithm** Newton;  

# Newton -initial -initialThenCurrent;

**analysis** Static

**puts** "Starting Pushover Analysis"

**set** ok [analyze $Nsteps]

**set** LunitTXT meter

**set** fmt1 "%s Pushover analysis: CtrlNode %3i, dof %i, Disp= %3f %s"

**set** Tol 1.0e-3
ModalAnalysis {
    set numModes 7
    puts "Eigen analysis"
    set eigenvalues [eigen $numModes]
    modalProperties -file $dataDir/ModalProperties.txt
    set T {}
    foreach lam $eigenvalues {
        lappend T [expr 2.0*$pi/sqrt($lam)]; # T=1/f= 2pi/wn
    }
    set Periods [open "$dataDir/Periods.txt" "w"]
    foreach t $T {
        puts $Periods "$t"
    }
    close $Periods
    set fid [open $dataDir/16ModeShape/NodeCoords.csv w]
    puts $fid "Node,X,Y,Z"
    foreach nodeTag [getNodeTags] {
        puts -nonewline $fid "$nodeTag,"
        puts $fid [join [nodeCoord $nodeTag] ,]
    }
    close $fid
    set fid [open $dataDir/16ModeShape/ElementConnectivity.csv w]
    puts $fid "Element,iNode,jNode"
    foreach eleTag [getEleTags] {
        puts -nonewline $fid "$eleTag,"
        puts $fid [join [eleNodes $eleTag] ,]
    }
    close $fid
    for {set mode 1} {$mode <= $numModes} {incr mode} {
        set fid [open $dataDir/16ModeShape/Mode$modeShape.csv w]
        puts $fid "Node,X,Y,Z"
        foreach nodeTag [getNodeTags] {
            puts -nonewline $fid "$nodeTag,"
            set locations [lmap dof {1 2 3} {nodeEigenvector $nodeTag $mode $dof}]
            puts $fid [join $locations ,]
        }
        close $fid
    }
}
default {
    puts "Wrong analysis"
}); # end switch analysis
}); # end foreach scour depth
}); # end foreach soil config
}); # end foreach bridge

Wipe
Appendix B: Fragility Function Generator File (Publicly Available)

Figure B.1 Screenshot of Fragility function Generator from The University of Toledo, OH, USA. Available at: [http://www.utoledo.edu/engineering/faculty/serhan-guner/data1/3S_FragilityFuncGenerator_V1.0.zip](http://www.utoledo.edu/engineering/faculty/serhan-guner/data1/3S_FragilityFuncGenerator_V1.0.zip)
Appendix C: Fragility Function Generator Numerical Example

This Appendix illustrates a numerical example for generating fragility curves. This example shows the intensity measure, velocity, that is varied from 0 to 2 m/sec under a scour condition of 3 m. The vertical axis of the fragility curves represents the probability of exceedance of the limit states. In this example, there are three damage states or performance levels: minor, moderate, and major.

1. Total Number of Load Intensities Analyzed
2. Intensity Label
3. Intensity Unit: m/sec

[4] Create Data Table

<table>
<thead>
<tr>
<th>Intensity No.</th>
<th>1</th>
<th>2</th>
<th>3</th>
<th>4</th>
<th>5</th>
<th>6</th>
<th>7</th>
<th>8</th>
</tr>
</thead>
<tbody>
<tr>
<td>Velocity [m/sec]</td>
<td>0.25</td>
<td>0.5</td>
<td>0.75</td>
<td>1</td>
<td>1.25</td>
<td>1.5</td>
<td>1.75</td>
<td>2</td>
</tr>
</tbody>
</table>

5. Number of Analyses Performed in Each Intensity
6. Engineering Demand Parameter (EDP) Considered
7. EDP Unit
8. Total Number of Performance Levels Considered

[8] Create Performance Level Table

<table>
<thead>
<tr>
<th>Performance Level</th>
<th>1</th>
<th>2</th>
<th>3</th>
</tr>
</thead>
<tbody>
<tr>
<td>Drift Limit (ratio)</td>
<td>0.01</td>
<td>0.02</td>
<td>0.03</td>
</tr>
</tbody>
</table>

10. Minimum Intensity in the Fragility Function
11. Maximum Intensity in the Fragility Function
### 13. INPUT TABLE

<table>
<thead>
<tr>
<th>Velocity (m/sec)</th>
<th>Drift (ratio)</th>
<th>Ln(Drift)</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.25</td>
<td>0.0004052013</td>
<td>-5.50854</td>
</tr>
<tr>
<td>0.5</td>
<td>0.0034010</td>
<td>-4.50664</td>
</tr>
<tr>
<td>0.75</td>
<td>0.016382467</td>
<td>-4.71767</td>
</tr>
<tr>
<td>1</td>
<td>0.024150577</td>
<td>-3.72452</td>
</tr>
<tr>
<td>1.25</td>
<td>0.0227711</td>
<td>-3.38648</td>
</tr>
<tr>
<td>1.5</td>
<td>0.033600733</td>
<td>-3.5104</td>
</tr>
<tr>
<td>1.75</td>
<td>0.038655533</td>
<td>-3.45442</td>
</tr>
<tr>
<td>2</td>
<td>0.0392111</td>
<td>-3.44449</td>
</tr>
</tbody>
</table>

Standard Deviation: 0.010023653
Ln of Std. Dev: 0.63172373

---

### 15. Fragility Function Data

<table>
<thead>
<tr>
<th>Velocity</th>
<th>Fragility for Limit States of Drift at 0.01 (ratio)</th>
<th>0.02 (ratio)</th>
<th>0.03 (ratio)</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.0000</td>
<td>0.0000</td>
<td>0.0000</td>
<td>0.0000</td>
</tr>
<tr>
<td>0.0667</td>
<td>0.0667</td>
<td>0.0667</td>
<td>0.0667</td>
</tr>
<tr>
<td>0.1333</td>
<td>0.1333</td>
<td>0.1333</td>
<td>0.1333</td>
</tr>
<tr>
<td>0.2000</td>
<td>0.2000</td>
<td>0.2000</td>
<td>0.2000</td>
</tr>
<tr>
<td>0.2667</td>
<td>0.2667</td>
<td>0.2667</td>
<td>0.2667</td>
</tr>
<tr>
<td>0.3333</td>
<td>0.3333</td>
<td>0.3333</td>
<td>0.3333</td>
</tr>
<tr>
<td>0.4000</td>
<td>0.4000</td>
<td>0.4000</td>
<td>0.4000</td>
</tr>
<tr>
<td>0.4667</td>
<td>0.4667</td>
<td>0.4667</td>
<td>0.4667</td>
</tr>
<tr>
<td>0.5333</td>
<td>0.5333</td>
<td>0.5333</td>
<td>0.5333</td>
</tr>
<tr>
<td>0.6000</td>
<td>0.6000</td>
<td>0.6000</td>
<td>0.6000</td>
</tr>
<tr>
<td>0.6667</td>
<td>0.6667</td>
<td>0.6667</td>
<td>0.6667</td>
</tr>
<tr>
<td>0.7333</td>
<td>0.7333</td>
<td>0.7333</td>
<td>0.7333</td>
</tr>
<tr>
<td>0.8000</td>
<td>0.8000</td>
<td>0.8000</td>
<td>0.8000</td>
</tr>
<tr>
<td>0.8667</td>
<td>0.8667</td>
<td>0.8667</td>
<td>0.8667</td>
</tr>
<tr>
<td>0.9333</td>
<td>0.9333</td>
<td>0.9333</td>
<td>0.9333</td>
</tr>
<tr>
<td>1.0000</td>
<td>1.0000</td>
<td>1.0000</td>
<td>1.0000</td>
</tr>
<tr>
<td>1.0667</td>
<td>1.0667</td>
<td>1.0667</td>
<td>1.0667</td>
</tr>
<tr>
<td>1.1333</td>
<td>1.1333</td>
<td>1.1333</td>
<td>1.1333</td>
</tr>
<tr>
<td>1.2667</td>
<td>1.2667</td>
<td>1.2667</td>
<td>1.2667</td>
</tr>
<tr>
<td>1.3333</td>
<td>1.3333</td>
<td>1.3333</td>
<td>1.3333</td>
</tr>
<tr>
<td>1.4000</td>
<td>1.4000</td>
<td>1.4000</td>
<td>1.4000</td>
</tr>
<tr>
<td>1.4667</td>
<td>1.4667</td>
<td>1.4667</td>
<td>1.4667</td>
</tr>
<tr>
<td>1.5333</td>
<td>1.5333</td>
<td>1.5333</td>
<td>1.5333</td>
</tr>
<tr>
<td>1.6000</td>
<td>1.6000</td>
<td>1.6000</td>
<td>1.6000</td>
</tr>
<tr>
<td>1.6667</td>
<td>1.6667</td>
<td>1.6667</td>
<td>1.6667</td>
</tr>
<tr>
<td>1.7333</td>
<td>1.7333</td>
<td>1.7333</td>
<td>1.7333</td>
</tr>
<tr>
<td>1.8000</td>
<td>1.8000</td>
<td>1.8000</td>
<td>1.8000</td>
</tr>
<tr>
<td>1.8667</td>
<td>1.8667</td>
<td>1.8667</td>
<td>1.8667</td>
</tr>
<tr>
<td>1.9333</td>
<td>1.9333</td>
<td>1.9333</td>
<td>1.9333</td>
</tr>
<tr>
<td>2.0000</td>
<td>2.0000</td>
<td>2.0000</td>
<td>2.0000</td>
</tr>
</tbody>
</table>

---

![Graph of Fragility Function Data](image-url)
Appendix D: Copyright Permissions

The permission in the email below is used for generating the fragility curves in Chapter 3. Please note this tool is publicly available and published under supervision of Professor Gunner from The University of Toledo, OH, USA. The email below shows the given permission for utilizing the tool in the present study. In this dissertation, this tool is used in scoured bridge and barge collision.
Re: Copyright

Amir Irhayyim
Sat 2/26/2022 2:11 PM
To: Guner, Serhan <Serhan.Guner@UToledo.edu>; Rafael Salgado <rafael.desalgado@gmail.com>
Cc: Manjriker Gunaratne; Leetta Schmidt; Amir Irhayyim <amiri2004@yahoo.com>

Dear Prof. Guner:
I really appreciate your effort for generate the tool!
I applied your tool with barge impact extreme event, and surely, I will send you my completed
dissertation when it will be ready!
Thanks for all you have done for the academy and researchers.
Sincerely,
Amir

From: Guner, Serhan <Serhan.Guner@UToledo.edu>
Sent: Saturday, February 26, 2022 12:40 PM
To: Amir Irhayyim <airhayyim@usf.edu>; Rafael Salgado <rafael.desalgado@gmail.com>
Cc: Manjriker Gunaratne <gunaratn@usf.edu>; Leetta Schmidt <lmschmidt@usf.edu>; Amir Irhayyim
<amiri2004@yahoo.com>; Amir Irhayyim <airhayyim@usf.edu>
Subject: RE: Copyright

Dear Amir,

Thanks for reaching out to me. Of course, you can use the tool in your studies. Please cite the following
reference (feel free to adjust the format to match the format of your reference list). Also better if you
don't include the tool as a zip file but refer readers to the web link to reach the latest version along with
documentation and videos on how to use it.

I would also appreciate receiving your completed thesis given my personal interest into this topic.

Spreadsheet, Department of Civil and Environmental Engineering, The University of Toledo, Ohio, USA,
https://www.utoledo.edu/engineering/faculty/serhan-guner/spreadsheets.html <accessed Feb 26,
2022>
You may also include link to the video as the user’s guide: https://youtu.be/Rah5detuekU

All the best with your studies and regards to your advisors.

Serhan Guner, Ph.D., P.Eng.
Associate Professor
Department of Civil and Environmental Engineering
The University of Toledo
2801 W. Bancroft St., MS 307, NI 3021
Toledo, Ohio 43606, USA
Phone: +1 (419) 530 8133

Website: www.utoledo.edu/engineering/faculty/serhan-guner