This dissertation presents a research program to continue the development of a vibration based damage detection technique to improve its practicality in identifying the existence and location of scour of coastal bridges, and to predict the effective height of supporting piles based on this selected damage feature of the superstructure. An analysis of recent bridge failures in the United States showed that the leading cause of bridge failures are due to hydraulic damage. In coastal regions, scour is a serious type of hydraulic damage resulting from a coastal storms. Scour occurs when strong water current removes compacted sediment around bridge pilings. Scour damage can be very difficult to detect since it is not always visually observable. Monitoring methods that involve underwater instrumentations are difficult for operators to perform and may result in damage or loss of equipment. Vibration based damage detection (VBDD) techniques have been developed to successfully detect scour damage in the laboratory and have also been implemented on an in-service bridge. The VBDD method based on the change in flexibility deflection of the superstructure has shown success in identifying the existence and location of scour damage, and has the potential to predict the effective pile height. But more research was desired to evaluate the practicality of VBDD techniques on more typical coastal bridge, such as bridges with multiple spans, or with several possible scour locations. The most crucial research of VBDD scour diagnosis that needed to be accomplished is the prediction of scour extent, or the effective pile height.

The first part of this dissertation focuses on the evaluation of the accuracy and practicality of the implementation of the selected VBDD technique on a chosen multi span bridge.
coastal structure. In order to apply this technique, the dynamic characteristics must be obtained from healthy and scoured bridge condition. Acceleration and impact loading during the vibration testing were collected in the field monitoring and signal processing was applied to determine the dynamic characteristics. The desired dynamic characteristics include the natural frequencies, the horizontal mode shapes, and the flexibility deflection of the superstructure. In this study, two similar portions of a coastal structure, except for the pile heights, are used to represent the healthy and scoured conditions. Thus, the problem becomes a comparison of the flexibility deflection from two similar structures with different pile heights. The results have shown that this VBDD technique can identify the existence of scour around the piles. Impact locations were also found to have an influence on the extracted horizontal mode shapes. The fundamental mode is also found to be the most important mode to be considered in the calculation of flexibility deflection.

The second part developed a mathematical relationship between the flexibility deflection and effective pile height. Stiffness ratio of the substructure to superstructure was found to be the primary parameter determining the dynamic behavior of a bridge superstructure. Numerous FE modal analyses were performed in order to establish the mathematical relationship. The connection is considered as part of the substructure that provides the horizontal resistance to the vibration of superstructure. Thus, the development of the mathematical relationship is first established for the case of a rigid superstructure-to-substructure connection. Then this relationship is adjusted to allow for different levels of connection stiffness. Lastly, a practical solution is proposed to be used in the field to predict the effective pile height. Through validation against finite element analysis, the proposed mathematical relationship shows reasonable prediction of effective pile height.
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Structural Vibration Diagnosis of Scour Damage in Coastal Bridges

by
Hao Hu

A dissertation submitted to the Graduate Faculty of
North Carolina State University
in partial fulfillment of the
requirements for the degree of
Doctor of Philosophy

Civil Engineering

Raleigh, North Carolina

2015

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To my father who taught me faith

To my mother who taught me love

To Yin my beautiful fiancee who brought me hope
BIOGRAPHY

Hao Hu was born on August 25th, 1988 in Wenzhou, China. After finishing his undergraduate studies in Zhejiang University within three years, he attended the NCSU-ZJU "3+1" Accelerated Master's program, and received his Master of Science degree in Civil Engineering in 2011 from North Carolina State University. The title of his master thesis was "Development of a new constitutive model for FRP-and-steel-confined concrete" under supervision of Dr. Rudolf Seracino. In September 2011, he was awarded the Southeastern Transportation Center (STC) Regional Outstanding Student Scholarship. Then he continued his Doctor of Philosophy degree in civil engineering under the supervision of Dr. Seracino. He conducted his doctoral research on vibration-based damage detection and health monitoring of coastal bridges.
ACKNOWLEDGMENTS

I would like to thank Dr. Rudolf Seracino for providing me with many great academic opportunities, his patient guidance throughout my research work, and his sincere care about my personal life. His academic excellence has always inspired me to improve my research and pursue success. I also would like to thank Dr. Sami Rizkalla for his guidance during my academic life at North Carolina State University. It was a meaningful experience to be your Teaching Assistant in CE327 Reinforced Concrete Design. Further, I would like to express my sincere appreciation to Dr. Mohammad Pour-Ghaz and Dr. Tom Birkland for their guidance and support that cannot be overlooked. I also would like to thank Dr. Jin-Guang Teng from the Hong Kong polytechnic University for his care and encouragement during my graduate program.

I would like to acknowledge the funding support of the Department of Homeland Security Coastal Hazards Center of Excellence as well as its staff and fellow researchers. I would like to thank Mr. Mike Muglia and Mr. Mike Remige for their help and collaboration during the organization of the field monitoring on Jennette's Pier. The experimental part of this research could not have been completed without the support and help of the staff at the Constructed Facilities Laboratory. I would like to thank Dr. Greg Lucier, and Mr. Jonathan McEntire. Particularly I would like to thank Mr. Jerry Atkinson, without your help and guidance, the field monitoring could not be easily carried out.

Special thanks to Mr. Zack Van Brunt, Mr. Bo Li, Mr. Pavan Chigullapally, Mr. Vivek Samu, Ms. Cuiyan Kong, Ms. Natasha Boger, and Mr. Bowen Shen for their generous offer to help during my experimental work.
The administrative assistance of Ms. Denise Thoesen and Mrs. Renee Howard throughout my academic life here is also appreciated.

I would like to thank my parents for their endless love and support during my life aboard. I would like to thank my beautiful fiancee Ms. Yin Sha for your love and endless encouragement.
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Chapter 1. Introduction

The U.S. Department of Transportation Bureau of Transportation Statistics identified that in 2011 11.5% of all highway bridges were structurally deficient and 12.8% were functionally obsolete. According to the Association of American State Highway and Transportation Officials (AASHTO), structurally deficient bridges are structures that cannot carry the expected load and are in need of repair, while functionally obsolete bridges are structures unable to fulfill their design and require immediate repair. Wardhana et al. (2003) investigated bridge failures from 1989 to 2000 in the U.S. and observed that hydraulic damage is responsible for 52.9% of all bridge failures studied. In coastal regions, hydraulic damage is commonly caused by coastal storms, during which storm surge may occur due to a rapid rise in the water level and can result in bridge deck uplift and excessive lateral loading restraint failure (Robertson et al. 2007). The strong currents also cause the removal of streambed resulting in scour of the pilings. But the existence and magnitude of scour damage can be very difficult to monitor in the field due to the cyclic nature of the scour process by removing sediment during a storm surge and partially refilling scour holes as the water recedes [FHWA, 2001]. The sediment refilled into the holes are not as compact as the rest of the soil around the pilings; thus, measuring streambed elevations may not be enough to capture the full extent of scour [De falco and Mele, 2002]. Also the presence of storm debris can influence the measurements of streambed elevation. Nondestructive testing methods such as ultrasound, magnetic field methods, and fiber-optic sensors usually require the portion of the bridge being inspected to be accessible. And many of them involve underwater instrumentation, that can be difficult for operators to operate and may result in damage or lost of equipment.
Therefore, vibration based damage detection (VBDD) techniques that avoid common monitoring issues have been developed as a means to diagnose scour damage of coastal bridges. This is based on the fact that damaged structures have a change in stiffness which will cause the vibration characteristic of the structure to change according to the damage extent. Elsaid (2011) successfully developed various VBDD techniques to diagnose scour damage by examining the change in vibration characteristics of the bridge superstructure between damaged and healthy conditions. Clark (2012) implemented those techniques on an in-service bridge. Both their works showed that the VBDD technique based on the change in flexibility deflection outperforms other techniques in identifying the existence and location of scour. In addition, this vibration damage feature has the potential to determine the extent of scour. But there is still a challenge in the practicality of the field implementation with this VBDD technique to identify scour of coastal bridges, such as bridges with multiple spans and several scour locations. A complete scour diagnosis framework should also include the prediction of scour extent, or the effective height of piles, once the existence of scour around the pilings has been identified.

1.1 Research Objective

This dissertation presents a research program to continue development of a vibration based damage detection technique to improve its practicality in identifying the existence and location of scour of coastal bridges, and to predict the effective height of piles based on the flexibility deflection of the superstructure. The output of this research aims to be used by first responders such as the Federal Emergency Management Agency (FEMA) to assess the condition and residual capacity of a bridge after a coastal storm. A quick and reliable
diagnosis of scour is crucial for them to make rapid and informal decisions regarding relief issues in cases of emergency.

1.2 Scope and Methodology

This research is divided into two primary subtasks. The first subtask focuses on the implementation of the VBDD technique based on the change in flexibility deflection on a chosen multi span coastal structure. In order to apply this technique, the dynamic characteristics must be obtained from healthy and scoured conditions. Acceleration and impact loading during the vibration testing were collected in the field monitoring and signal processing was applied to determine the dynamic characteristics. The desired dynamic characteristics include the natural frequencies, the horizontal mode shapes, and the flexibility deflection of the superstructure. In this study, as will be explained in Chapter 3, two similar portions of a coastal structure, except the pile heights, are used to represent the healthy and scoured conditions. Thus, the problem becomes a comparison of the flexibility deflection from two structures with different pile heights. The conclusion should be consistent with previous research that this VBDD can identify the existence of scour around piles.

The second subtask is to develop a mathematical relationship between the flexibility deflection and effective pile height. Theoretical study is conducted in order to understand the essential factors that determine the dynamic behavior of a bridge superstructure. Numerous FE modal analyse were performed in order to establish the mathematical relationship. The connection is considered as part of the substructure that provides the horizontal resistance to the vibration of the superstructure. Thus, this development of mathematical relationship is first established for the case of rigid superstructure-to-substructure connections. Then, the relationship is adjusted to allow for different levels of connection stiffness.
1.3 Organization of the Dissertation

This dissertation is organized in seven chapters.

Chapter 2 presents a review of bridge condition in the United States and leading causes of failure. Scour, as a serious type of bridge damage, is difficult to identify. The field application of different types of scour monitoring techniques are presented. The development history of VBDD technique on scour monitoring is reviewed, indicating several research gaps and providing the motivation for the research project.

Chapter 3 describes the experimental program implementing the VBDD technique based on change in flexibility deflection on Jennette's pier, a coastal structure located at Nags Head, North Carolina. The process of determining the test setup including the number and spacing of accelerometers is presented. The impact hammer and data acquisition system is also discussed. The details of the signal processing technique used to extract the natural frequencies and mode shapes are also described in this chapter.

Chapter 4 presents the data and results from the field monitoring on Jennette's Pier. This chapter presents the influence of impact location on the horizontal mode shapes. The calculation of flexibility deflection from the extracted natural frequencies and mode shapes is also described. Lastly, the comparison of flexibility deflection from two similar portions of the structure with different known pile height is made.

Chapter 5 describes the development of the proposed method focusing on the relationship of flexibility deflection to the effective pile height for the case of rigid connections between the superstructure and substructure of a bridge. The concepts of stiffness and mass ratio are introduced through a theoretical study. Numerous finite element models based on an idealized beam-column structure are used to complete the relationship
development. Then, the mathematical relationship is established based on a parametrical study and regression analysis.

Chapter 6 investigate the influence of connection stiffness on the vibration response of the bridge superstructure. An adjustment on the developed mathematical relationship presented in Chapter 5 is made based on various levels of connection stiffness. A practical solution is also proposed to relate the scour extent to flexibility deflection for bridges. Validation and application of this practical solution is performed based on finite element models of the idealized beam-column structure.

Chapter 7 summarizes all the critical findings from this study. This includes validation of the ability of the selected VBDD technique to detect scour and the practicality of the implementation on a coastal multi span structure. The advantage and accuracy of the proposed mathematical relationship between damage feature and effective pile height are also discussed. In the end, recommendations about future work are made as well.
Chapter 2. Literature Review

2.1 Overview

Scour of bridge substructures is a primary cause of failure and damage in the United States, especially for those in coastal areas prone to hurricanes. However, this type of damage is not always visually observable which makes it difficult to detect. Many scour monitoring techniques have been developed based on all kinds of theories, including vibration based damage detection techniques (VBDD). Recent researcher shows that these VBDD techniques are successful in detecting scour damage and have the potential to determine the extent of scour. This chapter highlights scour as major concern for coastal bridges, and introduce various sophisticated scour monitoring techniques as well as recently developed VBDD techniques.

2.2 Review of Bridge Damages and Failures

As an important to the nation’s transportation infrastructure, the structural health condition of bridges are always a concern. According to the statistics report from the U.S. Department of Transportation Bureau of Transportation in 2010, there were 604,460 highway bridges in the United States. Of these bridges, 11.5% were considered structurally deficient and 12.8% as functionally obsolete. AASHTO 2009 Bottom Line Report is more concerned with the age of the majority of bridges in the U.S, showing nearly half of all bridges are more than 40 years old and account for 80% of the structurally deficient bridges. By the definition given by AASHTO, structurally deficient bridges are structures that cannot carry the expected load and in need of repair. Functionally obsolete bridges are structures unable to fulfill their design and require immediate repair. Based on the current condition of many
bridges in the United States, proper detection of damage is critical to the health and safety of the transportation method.

### 2.2.1 Types of Damages and Failures

Wardhana et al (2003) examined 503 cases of bridge failures that occurred from 1989 to 2000 in the United States. The bridges age ranged from under construction (one year) to 157 years, with a mean of 52 years. This comprehensive study defined failed bridges as structures or components of structures that can no longer perform the function as specified in the design and construction requirements. The dominant types of failed bridges are the steel beam/girder and steel truss bridges, which represent 50% of the total bridge failures. The number of bridge failures is given in Figure 2.1 according to the year the failure occurred, which shows a peak in 1993 with 112 bridge failures (22%).

![Figure 2.1 Number of failed bridges distributed by year [Wardhana et al., 2003]](image)

The peak in 1993 is believed to be caused by a major flood in the Midwest which coincide with most of these bridge failures. The flooding of the Mississippi and Missouri rivers and tributaries caused numerous bridge failures across Iowa, Minnesota, and Missouri.
It has been observed that damage caused by flooding highly increased since the early 1990's. Table 2.1 summarizes various types of failure causes given by Wardhana et al. (2003). It should be noted that the major cause of bridge failures are hydraulic (52.9%), collision (11.7%), and overload (8.8%). Hydraulic events cause more damage in the US than any other severe weather related event, particularly flooding and scour.

Table 2.1 Types of failure causes from 1989 to 2000 [Wardhana et al, 2003]

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<th>Failure causes and events</th>
<th>Number of occurrences</th>
<th>Percentage of total</th>
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Table 2.1 (continued)

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<td>Soil</td>
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<tr>
<td>Miscellaneous/other</td>
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<td>4.4</td>
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<tr>
<td>Total</td>
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<td>100</td>
</tr>
</tbody>
</table>

2.2.2 Storm Surge Bridge Failures

According to The National Oceanic and Atmospheric Administration (NOAA), coastal storms are often a reason for flooding. Coastal regions are subject to high risk of coastal storms, flooding and coastal erosion. In fact, six of the ten states that have the most bridge failures are coastal states. NOAA indicates that coastal counties are home to half of US's population although only comprising 17% of the nation's land. Thus it is crucial to recognize these hazards are major threats to coastal infrastructure which are key components of the evacuation process and emergency response during and after a coastal storm event.

Robertson et al. (2007) investigated the structural performance under Hurricane Katrina storm surge (a storm surge is defined as a rapid rise in the level of water that occurs during a storm). They observed that the major causes of damage were uplift, excessive lateral loading restraint failure, debris effects and scour. Failure of coastal bridges were mainly due to storm surge. Chen et al. (2009) described the failure mechanism as the combination of storm surge current and waves generated by strong winds, causing both an uplift force and horizontal impact force. The uplift force can exceed the inadequate restraint against uplift (normally self weight of the bridge deck) and the horizontal impact force can exceed the
lateral resistance of the connections of the deck to the substructure. This leads to the deck falling off the piling caps. A example of the washout failure of the bridge deck of the Collapse of US 90 bridge connecting Biloxi to Ocean Springs, MS during Hurricane Katrina is shown in Figure 2.2 in a photograph.

![Image](image_url)

*Figure 2.2 Collapse of US 90 bridge [Bartram, 2006]*

### 2.2.3 Scour Damaged Bridges

In the investigation report of Robertson et al. (2007) regarding structural performance under Hurricane Katrina storm surge, extensive levels of scour around bridge abutments and piers was responsible for collapse or partial collapse of many of the bridges in the Gulf Coast area. This type of scour resulting from storm surge is called liquefaction-induced scour, that is, the saturated sandy soil behaves like a viscous liquid and thus can be easily displaced with water flowing. Figure 2.3 shows the most severe scour found after Hurricane Katrina along the Mississippi coastline, which was at the west-end abutment of the US 90 bridge in the town of Bay St. Louis, with a maximum scour depth of 15 ft (4.5 m).
Coastal storms can cause massive damage and failure to engineered structures, especially coastal bridges. Both washout of bridge decks as well as scouring of substructures are severe damage threats for coastal infrastructure following a coastal storm. Hurricane Ivan and Hurricane Katrina cost over one billion dollars to rebuild coastal bridges according to Chen et al. (2009). However, since bridge failure caused by storm surge can be easily replaced or rebuilt after such storm hazards, it is more critical to identify the damage (such as scour) which is not always visually observable when assessing the post-performance of coastal infrastructure after an extreme storm event. Thus monitoring for scour was selected as a focus area for the Department of Homeland Security Coastal Hazards Center.

2.3 Introduction of Scour

The Federal Highway Administration (FHWA) defines scour as erosion or removal of streambed or bank material from bridge foundations due to flowing water, usually considered as long-term bed degradation, contraction and local scour. FHWA HEC18 (2001) indicates different materials scour at different rates; loose granular soils are rapidly eroded by flowing...
water, while cohesive or cemented soils are more scour-resistant. However, ultimate scour in cohesive or cemented soils can be as deep as scour in sand-bed streams. Determining the magnitude of scour is complicated because of the cyclic nature of the scour process. Scour can be deepest near the peak of a flood, but barely visible as floodwaters recede and scour holes refill with sediment. Shallow foundations suffering from scour are subject to collapse. Scour occurring at deeper foundations may collapse the structure or undergo uneven vertical settlement of the structure along with lateral displacement from lateral loads [FEMA, 2009].

2.3.1 Basic Concept of Scour

According to FHWA HEC18 (2001), total scour at a bridge considers three components: (a) long-term degradation of the river bed, (b) contraction scour at the bridge and (c) local scour at the piers or abutments. The three scour components are added to obtain the total scour at a pier or abutment. This assumes that each component occurs independent of the other. In addition, there are other types of scour that occur in specific situations as well as lateral migration of the stream that must be assessed when evaluating total scour at bridge piers and abutments.

Degradation of the river bed refers to long-term natural or man-made changes in the trend of the stream that causes sediment to be displaced overtime and the elevation of the streambed is changed. This type of scour can be easily identified by monitoring streambed elevations overtime.

Contraction scour is a lowering of the streambed across the stream or waterway bed at the bridge. This type of scour may result from contraction of the flow, which causes removal of material from the bed across all or most of the channel width, or from other contraction scour conditions such as flow around a bend where the scour may be concentrated near the
outside of the bend. Contraction scour is different from long-term degradation in that contraction scour may be cyclic and/or related to the passing of a flood.

This research project is more concerned with the scour at the bridge piers and abutments, therefore the be focus will be on local scour. Local scour involves removal of material from around piers, abutments, and embankments. It is caused by an acceleration of flow and resulting vortices induced by obstructions to the flow. Local scour can either be clear-water or live-bed scour. Clear-water scour occurs when there is no movement of the bed material in the flow upstream except locally around piers or abutment. The acceleration of the flow and vortices at the pier or abutment created by these obstructions cause the bed material around them to move. Live-bed scour occurs when there is transport of bed material and is often cyclic because the scour hole that develops during the rising stage of a flood refills during the falling stage.

Typical clear-water scour situations include (1) coarse-bed material streams, (2) flat gradient streams during low flow, (3) local deposits of larger bed materials that are larger than the biggest fraction being transported by the flow (rock riprap is a special case of this situation), (4) armored streambeds where the only locations that tractive forces are adequate to penetrate the armor layer are at piers and/or abutments, and (5) vegetated channels or overbank areas.

During a flood event, bridges over streams with coarse-bed material are often subjected to clear-water scour at low discharges, live-bed happens at the higher discharges and clear-water scour at the lower discharges on the falling stages. Clear-water scour reaches its maximum over a longer period of time than live-bed scour (Figure 2.4). Local clear-water
scour may increase steadily overtime and reach to a maximum about 10% greater than the equilibrium local live-bed pier scour.

Figure 2.4 Scour depth in a sand-bed stream over time [FHWA HEC18, 2001]

The hydraulic mechanism causing local scour around piers or abutments is the formation of vortices, known as the horseshoe vortex, as demonstrated in Figure 2.5. The horseshoe vortex results from the pileup of water on the upstream surface of the obstruction and subsequent acceleration of the flow around the nose of the pier or abutment. The flow of the vortex removes bed material around the base of the obstruction. The transport rate of sediment from around the base of the obstruction is greater than the transport rate in the surrounding flow causing a scour hole to develop. As the size of scour increases, the flow of the horseshoe vortex is reduced, which in turn reduce the transport rate from the base region. Eventually equilibrium is reached between bed material inflow and outflow and suspended sediment may settle into the scour hole.
Factors which govern the magnitude of local scour depth at piers and abutments are (1) velocity of the approach flow, (2) depth of flow, (3) width of the pier, (4) discharge intercepted by the abutment and returned to the main channel at the abutment, (5) length of the pier if skewed to flow, (6) size and gradation of bed material, (7) angle of attack of the approach flow to a pier or abutment, (8) shape of a pier or abutment, (9) bed configuration, and (10) ice formation or jams and debris. Based on the many factors affecting scour, identifying the extent of scour is difficult to determine or predict.
2.3.2 Methods for Monitoring Scour

The US Geological Survey (USGS), the Federal Highway Administration (FHWA) as well as various states Department of Transportation (DOT) organizations have put a significant effort into the study of bridge scour. A database of scoured bridges across the US has been created under collaboration between the USGS and the FHWA. They also proposed documents and training materials with recommendations to guide the practice of engineers.

In addition to government research, there are many academic research programs related to scour monitoring techniques. Developing methods to monitor local scour at submerged piers is not simple. The flow of the vortex removes sediment from the base creating scour. At the decreasing phase of the flood, the water velocity decreases and the suspended sediments precipitate and partially fill the pits. However, the suspended sediments that settle into the excavated pits do not provide good confinement for the pile since they are not as compacted as the rest of the soil. This phenomenon shows that simple measurements of sediment levels carried out immediately after the event of flood are of little significance [De falco and Mele, 2002].

Many scour monitoring techniques have been developed and are utilized in field. Some of them will be discussed in following sections categorized measurement techniques. It should be mentioned that most of the available methods used to detect scour are based on underwater instrumentation.

2.3.2.1 Underwater Investigation

Having a dive team performing underwater investigation may be the most straightforward solution. The drawbacks of this method are obvious. The judgment of the scour presence and severity depends on experience and knowledge of inspectors, therefore
important details could be missing due to human error. Visual inspections may be not capable to fully capture all structure damages such as scour holes refilled with loose sediment which is not visually observable. The biggest concern of divers inspection is highly dangerous life risk, which can be avoided by using a unmanned marine vehicles (UMV). Murphy et al. (2011) developed an UMV technique to perform post hazard bridge inspections. The UMV results were similar to the field inspection results done by a dive team. The UMV is operated by three pilots to gather images through all cameras and to observe all collected data output. Figure 2.8 shows an UMV used to inspect bridges.

![Figure 2.6 Sea-RAI USV UMV used to inspect Rollover Pass Bridge [Murphy et al., 2011]](image)

The researchers investigated the ability of the UMV to inspect damage on the Rollover Pass Bridge over Galveston Bay three months after Hurricane Ike. The UMV was able to capture the damage of the bridge substructure and topography of the debris area. But this method was unable to detect scour damage at the pilings. A major advantage compared to dive team investigation, as has already been mentioned, is the highly dangerous life risk to divers could be avoided by using a UMV. Human error could also be avoided since the UMV is less likely to miss important details. According to Murphy et al. (2011), there were certain
disadvantages of the UMV such as vulnerability to extreme working environment. That is, the equipment is subject to damage or loss risk due to changes in tide or current. Because all equipments are installed on the marine vehicle, nearby boat ramps are needed to get the UMV into working position. Also, large debris or changes in topography are challenges to navigate the UMV in water. Lastly, the ability to inspect connection damage or scour holes refilled with loose sediment was not improved by using an UMV for the same reason as dive team visual inspection.

2.3.2.2 Streambed Elevation Measuring Methods

This section describes three techniques which all measure elevation changes in the streambed level. These instruments include sounding weights, echo sounders, magnetic sliding collars and TDR.

Mueller et al. (1999) used sounding weights and echo sounders to measure the streambed elevation changes during and after a flood to investigate scour monitoring methods. The streambed elevation was obtained by measuring the distance between a datum point and the streambed. These two methods represent common portable streambed elevation measuring devices.

The sounding weights (typically 45 to 136 kilograms) are lowered and lifted by a bridge crane on the bridge deck to measure the streambed depth. This method can be used during a flood since the heavy weights can reach to the streambed under the circumstance of strong water currents flowing. In fact, this method has been used to measure changes in the streambed of the Red River flooded in May of 1990. There are several major disadvantages against this method. It could be dangerous to operate a crane on the bridge deck during a flood or shortly afterwards. Moreover, due to high current flow the weights may be swept off
the location while descending causing inaccurate measurements. It is critical for the weights to drop precisely in the vertical direction in order to make accurate measurements. Also, debris may snag the line as the weights are being lowered. In the worst scenario, if the line breaks the operators are at a safety risk and the weight may become lost. Thus, as a general rule, the sounding weights are suggested not to be used if the depths are greater than 10 m and velocities of flow are greater than 3 m/s. This method is only capable of capturing the depth of scour, not the size of the scour hole. The disadvantages of this system are significant, so Mueller et al. (1999) explored using more advanced echo sounders to measure streambed depths.

Echo sounders calculate the distance from a transducer above the water surface to the streambed by multiplying the speed of sound through the water and the time for the signal to reflect off the streambed and back to the transducer. Figure 2.7 shows an echo sound transducer encased by modified water skis being navigated around a bridge pier under operation.

![Echo sound transducer measuring streambed depth](image)

Figure 2.7 Echo sound transducer measuring streambed depth [Mueller et al., 1999]
This method performed well during three major floods since it created insignificant drag and was easy to operate in turbulent water. An obvious advantage of echo sounders compared to sounding weight devices is that measurements can be recorded at many different points so the scour level of the entire scour hole can be obtained. Echo sounders are capable of measuring the depth larger than 3 m deep to sediment and water velocities less than 4 m/s. In terms of disadvantages, echo sounders did not make good measurements during an intense flood because the signal was vulnerable to high levels of turbulence and air entrainment. Also, echo sounders may record incorrect data as there is a large amount of suspended sediment or debris in the water affecting the recorded signal. As mentioned before, sediment can refill scour holes and do not represent adequate confinement to the foundation structure.

The magnetic sliding collar (MSC) system is comprised of a steel rod vertically driven into the streambed and a collar that slides down the rod while the streambed elevation is lowered as the sediment erodes. Magnets attached to the collar determine the collar’s depth based on its location on the rod. This device can only be used to measure the maximum depth either manually or automatically. For manual magnetic sliding collar, installations are fairly inexpensive and can be easily installed during low flow events. One significant disadvantage is that manual MSC is very susceptible to debris in the water. Automatic magnetic sliding collar systems are more expensive and more robust than manual MSC. However, the automatic MSC reading can be interfered in biologically active environments, such as estuaries and tidal rivers, where barnacle growth and other biofouling can occur. The biofouling can prevent the collar from moving on the rod.

Time Domain Reflectometry (TDR) was originally used by electrical engineers to locate discontinuities in power and communication transmission lines. It can also measure
materials’ dielectric and electrical properties. Recently the applications were extended to civil engineering, such as measurements of soil water content and dry density, evaluation of concrete strength, and monitoring of bridge scour, according to Yu and Yu (2009). Figure 2.8 shows a typical configuration of a TDR system which generally is comprised of a TDR device (pulse generator and sampler), a connection cable, and a measurement probe. TDR systems measure the reflection that results from a fast rising step pulse. This reflections varies based on the change of system geometry or material dielectric permittivity. Then the sediment level, same as soil-water interface, can be determined by recording the round trip travel time of the pulse. But the major disadvantage of the TDR method is that the accuracy highly depends on environment temperature and humidity.

![Figure 2.8 Schematic of a typical TDR system [Yu and Yu, 2009]](image)

**2.3.2.3 Fiber-Optic Sensor Bridge Monitoring**

In recent years, techniques based on fiber optic sensors have been developed and used on real-time monitoring projects of bridge scour because of their advantages such as long-
term stability and reliability, resistance to environmental corrosion, and immunization on electrical and electromagnetic noise. Zarafshan et al. (2012) developed a vibration-based method to determine scour level around bridge piers and abutments by measuring fundamental natural frequency of a rod embedded into the streambed. Then the scour depth can be calculated based on an inverse relationship between the fundamental frequency and the free length of the sensor rod. The fiber-optic Bragg grating (FBG) dynamic sensor was chosen to determine the natural frequency of the rod because of many strengths mentioned above. This scour monitoring system is comprised of the FBG sensors connected to a rod and a fiber-optic cable routed to an optical unit that converts the signals of the sensor to vibration characteristic, a field computer to control the optical unit, a data acquisition system for the processing, plus a cellular wireless modem connected to the computer to wirelessly transfer the real-time data to the laboratory. Figure 2.9 shows the FBG sensors, rod, and fiber-optic cable embedded in the streambed.

![Figure 2.9 Fiber-optic sensor connected to rod embedded in streambed [Zarafshan et al., 2012]](image)

The researchers successfully investigated the scour condition of a bridge over Salt Creek River in Illinois through this method to observe the changes in sediment level. But there are certain disadvantages against this method. The recorded signal may be interfered by
debris, such as mud, rock or leaves around the sensors. Thus regular inspection and cleaning of any accumulated debris must be undertaken to insure the accuracy of the recording signal. These actions would increase labor and other costs of the monitoring. Lastly, as mentioned before, sediment could refill scour holes and do not provide adequate confinement to the foundation structure. The whole measurement of changes in sediment level might be of little significance.

2.4 Vibration-Based Damage Detection Techniques

Underwater investigations, streambed elevation measuring methods, measurements with fiber-optic sensors are all scour monitoring techniques that have been implemented in the field. Although many of these methods are able to measure the amount of scour, each of them have significant drawbacks in terms of monitoring process, as mentioned previously. As a crucial factor, underwater interactions are responsible for potential equipment damage or lost risk, and inaccuracy of measurements interfered by environmental conditions. In addition, underwater investigation techniques usually require specialized training, which may increase the difficulty of the monitoring process right after an emergency event. Vibration-based damage detection (VBDD) techniques is a developing scour monitoring system that completely avoids the concern of underwater interactions since these techniques are based on the dynamic response of only the bridge superstructure, and therefore should be beneficial to end-users for easy implementation. VBDD techniques have been developed and proved to be able to identify scour damage in theoretical, laboratory testing and field implementation. The following sections introduce important research work done by Mosavi (2010), Elsaid (2011), and Clark (2012), representing crucial efforts towards developing VBDD techniques to monitor scour damage of bridges.
2.4.1 Exploration of VBDD by Mosavi (2010)

Mosavi (2010) did not study the application of VBDD techniques on scour damage detection, but the research suggested VBDD techniques as a potential solution to identify the presence of scour based on his investigation of identifying the location of damage in steel girders. The researcher constructed an idealized two-span steel laboratory structure and damaged the beams by saw cutting different length cuts at two different locations. Then vibration response of the beam at various damage conditions was recorded through 15 sensors along the length of the beam. From the collected vibrations, damage diagnosis patterns were able to be derived based on comparison between healthy and damaged conditions using time series based statistical pattern recognition. The Fisher criterion was used to identify the damage location in which the Fisher criterion value should theoretically be highest. In this case, the highest Fisher criterion value corresponded to the sensor closest to the beam damage location. Thereby, Mosavi successfully identified the location of damage on steel laboratory beams. Based on his observation, the natural frequencies of the beam gradually decreased from the healthy beam condition to various damaged beam condition. Therefore, he suggested dynamic characteristics such as natural frequencies could be used to monitor scour.

2.4.2 Development of VBDD by Elsaid (2011)

Based on the observations and suggestions of Mosavi (2010), Elsaid (2011) developed several VBDD techniques to identify the presence and location of scour damage in bridges. He created an idealized two-span steel bridge laboratory structure with intermediate support of various heights to simulate different levels of scour. Figure 2.10 shows a photo of the idealized two span steel laboratory model.
The vibration response, namely accelerations, produced by an impact hammer were recorded through sensors along the span. The accelerations together with the applied impact load were used to derive the mass-normalized structure mode shapes based on calculation of the frequency response functions (FRF) and examination of FRF peaks, which revealed mode shapes and natural frequencies of the structure, according to Zhou (2006). It should be mentioned that Mosavi (2010) detected damage in steel beams with only vertical mode shapes and corresponding natural frequencies, while Elsaid (2011) obtained both horizontal and vertical mode shapes, as well as natural frequencies. Elsaid also observed that the scour models with longer intermediate support had a lower natural frequency compared to the shortest support length model which represented the reference healthy condition. His research indicated that the horizontal mode shapes were more sensitive to scour damage compared to vertical mode shapes. Therefore, Elsaid proposed and implemented three
techniques based on the horizontal mode shapes to identify the presence and location of scour damage, as introduced in following sections.

2.4.2.1 Method based on change in Mode Shape Curvature

Elsaid (2011) proposed the change in curvature of the horizontal mode shapes between the healthy structural condition and scoured condition as the indicator for the presence and location of scour damage. The curvature of the mode shapes were calculated using central difference approximation. To indicate the change in curvature, Elsaid applied the concept called the curvature damage factor (CDF) that was originally developed by Abdelwahab and De Roeck (1999). The CDF can be calculated based on the difference in curvature of the mode shape between the healthy and scoured structure and then divided by the total number of modes considered. The CDF value was calculated for all sensor locations along the bridge deck with local peaks expected at the damage location, through which the region of scour is revealed. However, the CDF value peaked at multiple sensor locations including the damage location and several false positives were observed in the theoretical models of different levels of scour. Elsaid normalized the CDF by the curvature of the healthy structure to emphasize the true peak corresponding to the damage location. This new normalized value was called the modified curvature damage factor (MCDF). Similar to the CDF, the MCDF value was calculated at all sensor locations along the bridge deck. Figure 2.11 gives the CDF and MCDF values calculated from finite element models for 24 in of symmetrical scour and the healthy condition considering the first five mode shapes. The true location of scour is at a distance of 9 feet, which is the location of the intermediate support.
Figure 2.11 CDF and MCDF values for 24 in scour from a finite element model of an idealized two span steel laboratory structure considering first five horizontal mode shapes [Elsaid, 2011]

The CDF, as mentioned before, was unable to effectively locate the scour damage because the value had significant local peaks at several locations. The MCDF successfully indicated the true peak corresponding to the damage location in this finite element study of the laboratory structure. Based on Elsaid's observation that the MCDF value increased with scour level, MCDF has the potential to also determine the amount of scour.
However, Figure 2.12 presents the CDF and MCDF values calculated from the laboratory vibration testing for the same 24 in symmetrical scoured condition and the healthy condition considering the first five mode shapes. The location of scour is still at distance of 9 feet, the location of the intermediate support.

Figure 2.12 CDF and MCDF values for 24 in scour for the idealized two-span steel laboratory model [Elsaid, 2011]
When applied to the experimental data from the laboratory structure, apparently both CDF and MCDF were unable to identify the damage location. Elsaid attributed this to the unevenness of the calculated mass-normalized mode shapes. From a mathematic point of view, it is the higher order derivative making this curvature based method highly sensitive to the smoothness of the mode shape curves.

2.4.2.2 Method based on change in flexibility deflection

Elsaid proposed a second damage detection technique based on change in flexibility deflection. The flexibility can be calculated based on the horizontal mass-normalized mode shapes and the corresponding natural frequencies. Then the flexibility deflection is obtained by multiplying the flexibility by a unit load. Similarly to the previous curvature based method, the value of change in flexibility deflection between the scoured and healthy condition should peak at the damage location. Elsaid examined this method with both the finite element model and idealized laboratory structure. Figure 2.13 presents the change in flexibility deflection values calculated from finite element models between scoured and the healthy condition considering the first five mode shapes. Figure 2.14 is the change in flexibility deflection for the laboratory model between scoured and the healthy condition for the first five horizontal mode shapes and the natural frequencies. The scour depth in both cases was 16 in of symmetric scour at the location of the intermediate support.
Figure 2.13 Change in flexibility deflection for 16 in of symmetrical scour for two span finite element model [Elsaid, 2011]

Figure 2.14 Change in flexibility deflection for 16 in of symmetrical scour for idealized two span laboratory structure [Elsaid, 2011]
The method based on change in flexibility deflection successfully identify the damage location for the theoretical model as well as the laboratory structure. Elsaid also observed that the magnitude of the change in flexibility deflection at the location of scour increased with sour level increase for both cases, as shown in Figure 2.15 and Figure 2.16. These findings indicate that the method based on change in flexibility deflection is capable of identifying the presence and location of scour, and has the potential to reveal the extent of scour.

Figure 2.15 Relation between the change in flexibility-based deflection and scour for two-span finite element model [Elsaid, 2011].

Figure 2.16 Relation between the change in flexibility-based deflection and scour for idealized two-span laboratory model [Elsaid, 2011].


2.4.2.3 Method based on change in curvature of flexibility deflection

In addition to the previous methods, Elsaid (2011) investigated the performance of the change in curvature of the flexibility deflection between scoured and healthy structure conditions, as an indicator for the presence and location of scour. Figure 2.17 presents the change in curvature of flexibility deflection from the finite element model between the 16 in of symmetrical scour and the healthy structure. Similarly, Figure 2.18 shows the change in curvature of flexibility deflection from the idealized laboratory structure between the 16 in of symmetrical scour and the healthy structure.

Figure 2.17 Change in curvature of flexibility deflection between and 16 in of symmetrical scour and the healthy condition from the finite element model [Elsaid, 2011]
Theoretically, the method based on change in curvature of flexibility deflection can be used to reveal the presence and location of scour. However, these curvature based method was again unable to identify the presence and location of scour due to multiple false positives when experimental data was used. Again, this curvature based method is highly sensitive to the unevenness of the curves which is an unavoidable situation in laboratory and field testing.

### 2.4.3 Implementation of VBDD in field by Clark (2012)

With all the knowledge gained from Elsaid's (2011) development of three vibration based damage detection techniques, Clark (2012) successfully implemented these methods on an in-service bridge structure as well as a finite element model that was subject to scour at the location of the intermediate support. Figure 2.19 shows the field monitoring of the Hwy 17 Northeast Creek Bridge located in Jacksonville, North Carolina. The scour occurred at the submerged piles, as indicated in Figure 2.19. As an approximate simplification only the two spans adjacent to the scour location were investigated.
In this field monitoring, a Hydra-Platform truck was provided by NCDOT to gain access to the side of the superstructure in order to mount the sensors on the outside girder and impact the deck, as shown in Figure 2.20.
Two tests were undertaken, one before the retrofit, representing the scoured condition, and the other right after the retrofit to correct the scour problem, representing the healthy reference structure. The horizontal mode shapes and corresponding natural frequencies were derived by post processing of the gathered impact load data and acceleration response. It should be noted that due to limited channels in the data acquisition system, the vibration tests were performed for Span 1 and Span 2 separately. Then the mode shapes extracted from each test were combined to get the complete curves, as shown in Figure 2.21. However, the compromise of curve connection for two separate span tests due to limitation of the data acquisition system not only brought up difficulty in identifying mode shapes, but also resulted in different natural frequencies $\omega$ for the two spans, as shown in Figure 2.21, which is unrealistic.

![Mode Shapes](image.png)

Figure 2.21 The completed mode shape curve obtained from separate test in individual span
Clark examined the three techniques developed by Elsaid with finite element models and field bridge monitoring results. The performance of the modified curvature damage factor (MCDF) is shown in Figure 2.22 and Figure 2.23 for finite element model and field monitoring, respectively. As expected, the MCDF is unable to identify the presence and location of scour in a realistic structure due to its sensitivity to the unevenness of the mode shape curves.

![Figure 2.22 MCDF values considering first five horizontal mode shapes from finite element model [Clark 2012]](image1)

![Figure 2.23 MCDF values considering first five horizontal mode shapes from field monitoring [Clark 2012]](image2)
The flexibility-based deflection can be calculated based on the mode shapes and natural frequencies. Figure 2.24 and Figure 2.25 present the change in flexibility deflection from finite element model and field monitoring considering first five mode shapes, respectively. This method successfully identifies the location of scour at the intermediate support.

Figure 2.24 Change in flexibility deflection from finite element model considering first five horizontal mode shapes [Clark 2012]

Figure 2.25 Change in flexibility deflection from field monitoring considering first five horizontal mode shapes [Clark 2012]
Clark also examined the change in curvature of the flexibility deflection against the finite element model and field monitoring data, as shown in Figure 2.26 and Figure 2.27, respectively. As expected, this method performed well in identifying the scour location for finite element model, but not when using the field monitoring data.

Figure 2.26 Change in curvature of flexibility deflection from finite element model considering first five horizontal mode shapes [Clark 2012]

Figure 2.27 Change in curvature of flexibility deflection from field monitoring considering first five horizontal mode shapes [Clark 2012]
2.5 Research Gaps

From the above discussion comparing different VBDD indicators, one can conclude that the method based on the change in flexibility deflection outperforms the other methods, which was capable of identifying both the presence and location of scour. Moreover, it has the potential to quantify the amount of scour. A comparison of all indicators is summarized in Table 2.2.

Table 2.2 Performance comparison of proposed VBDD scour indicators

<table>
<thead>
<tr>
<th>Investigation type</th>
<th>Mode shape curvature (MCDF)</th>
<th>Flexibility based deflection</th>
<th>Flexibility based curvature</th>
</tr>
</thead>
<tbody>
<tr>
<td>FEM of idealized two-span structure</td>
<td>Presence+ location+ scour level</td>
<td>Presence+ location+ scour level</td>
<td>Presence+ location+ scour level</td>
</tr>
<tr>
<td>Laboratory idealized structure</td>
<td>none</td>
<td>Presence+ location+ scour level</td>
<td>Presence+ location</td>
</tr>
<tr>
<td>Northeast Creek Bridge field monitoring</td>
<td>Presence</td>
<td>Presence+ location+ scour level</td>
<td>none</td>
</tr>
<tr>
<td>FEM of Northeast Creek Bridge</td>
<td>Presence</td>
<td>Presence+ location+ scour level</td>
<td>Presence+ location+ scour level</td>
</tr>
</tbody>
</table>

VBDD can easily and quickly be implemented in the field. Compared with other scour monitoring techniques, VBDD techniques also reduce the risk of damage or lost equipment by avoiding underwater instrumentation. To develop and implement these techniques, all structures tested to-date only have two spans or were idealized as two span structures, the Hwy 17 Northeast Creek Bridge in Jacksonville, North Carolina, has 4 spans.
But the field test was undertaken only on two spans to monitor the scour at the location of one intermediate support, thus the bridge was considered as a two span structure.

However, the methodology of combining the mode shape curves from two separate tests to get the complete mode shapes for two spans resulted in different natural frequencies for the same mode, which is unrealistic and unreliable. Furthermore, the Hwy 17 Northeast Creek Bridge only had one scour location.

Therefore, there is a need to implement the VBDD technique on a coastal structure which has multiple scour locations and avoiding the need to combine the mode shape curves, As the first task in this study, the VBDD technique based on the change in flexibility deflection will be implemented on a coastal structure to validate its capability of identifying the existence and location of scour.

The most crucial work in scour diagnosis is the assessment of structural residual capacity. That relies on the prediction of the scour extent, or the current effective height of the pilings. Elsaid (2011) showed the a relationship between scour extent and the change in flexibility deflection existed through both experimental results and finite element analysis, by collecting 4 known conditions of flexibility deflection and the pile height, as shown in Figure 2.15 and Figure 2.16, respectively. However, Elsaid did not develop a relationship between the magnitude of the flexibility deflection determined from a field monitoring following a storm event and the corresponding height of the pile substructure. This represents the missing last step of a scour diagnostic methodology that will allow the residual capacity to be determined.
Chapter 3. Experimental Program and Data Processing

3.1 Introduction

For the reasons summarized in the research gaps of Chapter 2, it was necessary to show the practicality of using selected vibration based damage detection method to identify the existence and location of scour on a existing coastal bridge. Jennette's Pier was chosen as the field monitoring site, located at Nags Head, North Carolina. This structure has multiple spans and the effective height of supporting piles is changing all the time due to wave and the tidal effect. The substructure of the structure is composing of piles, which is representative of typical coastal bridges. The Pier is shown in Figure 3.1.

![Jennette's Pier, Nags Head, North Carolina](https://example.com/image.jpg)

Figure 3.1 Jennette's Pier, Nags Head, North Carolina [North Carolina Construction News, June 2011]

This chapter presents the experimental program and post data processing of the implementation of the current VBDD technique on Jennette's Pier. Chapter 4 will presents the results from this field implementation. It is expected that the flexibility based deflection increases as the pile height increases and the change in flexibility deflection at scoured
locations reach maximum, which would show its capability to identify sour. As summary, this field monitoring study aims to validate the chosen method through the implementation on a typical coastal structure.

3.2 Pier Geometry

This structure serves as a fishing pier and a tourist attraction. Thus, no heavy transportation is allowed on the pier. The Pier extends to the Atlantic Ocean with three rest platforms and 17 spans. As shown in Figure 3.2, spans A, B, and C defined by grid lines 27, 28, 29, and 30 were chosen as the bridge span to conduct the field testing. The following details of the Pier focus on the portion of structure starting from grid line of 27 to 30.

According to the plans provided for Jennette's Pier, the pile height at grid line 29 is 31.87 ft, as calculated from the top of pile to the seabed elevation, which is about 7 in deeper than the pile height at grid line 28 (31.3 ft). The increasing pile height due to the slope of the seabed was also verified by submerging a steel block to the seabed at the pile locations and measuring the length of rope. The approximate measurement of pile height shows the seabed
elevation at grid line 29 (40.9 ft) was about 12 in deeper than the seabed elevation at grid line 28 (39.8 ft) on the day of testing.

These three spans have the same length of 40 ft and identical cross-section. The superstructure is compromising of four precast concrete beams and a wood deck, that is supported by precast concrete bent caps (3 ft in depth) and four precast piles. The wood deck is made of 3/6 in planks, with an actual thickness of 2.5 in. An elevation of the pier is shown in Figure 3.3.

![Figure 3.3 Typical elevation of Jennette's Pier](image-url)
Bearing pads (or grout) between the precast beams and the bent cap, together with 150 ksi threaded rods grouted in corrugated sleeves are used to resist horizontal relative movement between the superstructure and support bent cap. The details are shown in Figure 3.4.

Figure 3.4 Pier structure connection details
The precast reinforced concrete piles have a square section, as shown in Figure 3.5. It should be noted that the outer two piles are not vertical.

3.3 General Plan of Field Test

The ideal field monitoring plan requires at least two site visits, including the collection of one "healthy" data set and one "damaged" data set immediately after an extreme storm event. However, the "healthy" data set of the pier is not strictly accurate since the Pier was reconstructed and reopened in 2011 after Hurricane Isabel destroyed the original pier in September 2003. Unfortunately, since the storm season coincide with the peak tourism season at Jennette's Pier no field monitoring was permitted at this time.

Thus, the monitoring plan was revised to be the comparison of two pile locations rather than monitoring the change of scour at one location. Because of the slope change of the seabed, the piles further from the shore have longer heights. Piles at grid line 28 and 29

Figure 3.5 Pile section detail
were investigated from the dynamic response collected from spans A and B, and spans B and C separately. As discussed previously, these two spans are identical including the connection between the superstructure and support piles except for their height, which makes the comparison of piles at gird lines 28 and 29 reasonable. Test AB was conducted on spans A and B, and Test BC was conducted on spans B and C to allow comparison of the flexibility based deflection from two spans of structures that have different pile height. In each test, the vibration signal is collected for two spans at same time. Therefore, there is no need to adopt Clark's (2012) methodology to merge mode shape of two spans from separate tests.

3.4 Determining the Instrumentation Requirements

3.4.1 Selection of Accelerometers

Bridge field monitoring requires collecting horizontal accelerations due to impact loading. Accelerations are recorded through accelerometers connected to a data acquisition system. For the purpose of determining the specifications of the accelerometers and their placement in field, a transient time history finite element analysis was performed through SAP 2000 to simulate the vibration response of the bridge structure under an impact loading. Accelerometers were chosen for testing based on the predicted range of accelerations the structure would reach during impact loading.

Due to the discontinuity of the beam girders from two adjacent span, as shown in Figure 3.4, the assumption is made that the impact will only cause significant vibration of the impacted span and the adjacent span. This is also verified by observations during previous bridge monitoring that the acceleration signals recorded two spans away from the impacted span is in the magnitude of the noise level. Thus only spans A, B and C were used to create the Finite Element (FE) model to simulate the vibration response of selected portion of the
structure under impact load at the middle of span B. Beam elements were used to model the structural components including the beams, wood deck, bent cap, and the supporting piles. The piles were fixed at the seabed, at a height of 30 ft, as an approximation of the actual height. The beams were fixed in the horizontal direction at abutments. The beams and the bent cap were assumed constrained at all three translational directions but free at all rotational degree of freedom. The mesh of the finite element model and the applied load location are shown in Figure 3.6.

![FE model of spans A, B and C of Jennette's Pier](image)

Figure 3.6 FE model of spans A, B and C of Jennette's Pier

An impact load was simulated in the model by applying a triangular shaped load function which reaches 5000 lb in 0.5 ms, and returns to zero at the same rate as shown in Figure 3.7. It should be mentioned that the impact load magnitude (5000 lb) was chosen based on previous field testing performed by Clark (2012).
Then, a transient time history analysis was performed to obtain the dynamic responses of the structure due to the applied loading. The acceleration signal from the FE analysis, as shown in Figure 3.8, was obtained at the location of impact loading. The maximum acceleration of 0.31 g was obtained at the transient part of the signal. And the steady state part of the signal has a maximum value of 0.01 g. Since no damping was considered in this FE simulation, the signal did not decay, as shown in Figure 3.8.
The measurement range of accelerometers should be greater than the maximum acceleration observed through the numerical model. According to Mosavi (2010) the experimentally observed accelerations often exceed those predicted by numerical simulation. Besides, the FE analysis underestimates predicted maximum acceleration responses by assuming full constraint in translational directions between beams and bent cap. The end abutments were assumed fixed in horizontal directions, which also underestimates the acceleration response.

The accelerometers have an inverse relationship between sensitivity and measurement range. In other words, the higher the sensitivity, the smaller the range of measurement. Therefore, the seismic ICP accelerometer (PCB 393B04) was used for the field monitoring. The accelerometer had a measurement range of $\pm 5 \text{ g}$ which exceeded the 0.31 g predicted in the finite element model. The accelerometer had a sensitivity of $1 \text{ V/g}$. With a light weight of 1.8 oz, this accelerometer can be easily mounted to the side of the beams. Figure 3.9 shows one of these accelerometers.

![Accelometer](image)
3.4.2 Placement of the Accelerometers

Designing the sensor placement for testing is an optimization problem which requires a balance between having enough sensors so no important information is lost while not having more sensors beyond economic limitations. Sensor optimization research done by Trendafilova et al. (2001) employed Shannon's idea of mutual information (Gallager, 1986) to find the optimum distance between sensors for the vibration measurement of a structural plate, with which no information is neither lost nor duplicated between two adjacent measurement points. In other words, this optimal distance makes information between adjacent sensors independent.

Elsaid (2011) successfully adapted the method developed by Trendafilova et al. (2001) to assess the optimal distance between horizontally mounted accelerometers on an idealized laboratory steel beam structure for damage detection. Clark (2012) also applied this method to estimate the spacing and number of accelerometers required for the field testing of the Hwy 17 Northeast Creek Bridge. Therefore, the same method was used in the current research to determine the optimal distance and number of accelerometers for the testing of Jennette's Pier.

Based on Shannon's idea of mutual information, given that measurement $a_i$ is from sensor A and measurement $b_j$ is from sensor B, the mutual information between these two measurements can be represented as Equation 3-1:

$$I(a_i, b_j) = \log_2 \left[ \frac{P_{ab}(a_i, b_j)}{P_a(a_i)P_b(b_j)} \right]$$  

Equation 3-1
where $I(a_i, b_j)$ is the mutual information between measurement location A and B, $P_{AB}(a_i, b_j)$ is the joint probability density for measurements taken at A and B, and $P_A(a_i)$ and $P_B(b_j)$ is the individual probability density for point A and B, respectively.

When the measurements between A and B are completely independent from each other, $I(a_i, b_j)$ will be equal to zero because the individual joint densities of the two points together will be equal to the joint probability density between the two points, as shown in Equation 3-2:

$$P(a, b) = P(a)P(b)$$

Equation 3-2

Then the average overall mutual information between measurement points equally spaced $\Delta x$ from adjacent points can be rewritten as Equation 3-3:

$$I(\Delta x) = \sum_{i=1}^{N} P(a_i, a_{i+\Delta x}) \log_2 \left[ \frac{P(a_i, a_{i+\Delta x})}{P(a_i)P(a_{i+\Delta x})} \right]$$

Equation 3-3

where $I(\Delta x)$ is the average mutual information, $P(a_i, a_{i+\Delta x})$ is the joint probability density between $a_i$ and $a_{i+\Delta x}$, and $P(a_i)$ and $P(a_{i+\Delta x})$ is the individual probability densities of $a_i$ and $a_{i+\Delta x}$, respectively.

The individual probability densities are calculated from marginal histograms of $a_i$. The joint probability densities can be calculated from the two-dimensional marginal histogram of $a_i$ and $a_{i+\Delta x}$. When $\Delta x$ increases, $I(\Delta x)$ should approach zero theoretically since the measurement from different sensors become completely independent. However, in practice, the mutual information will converge to a small value but not zero at a distance where information becomes most independent.

Based on the calculation of sensor spacing for the damage detection test on a structural plate, Trendafilova et al. (2001) recommended to choose the optimal distance of
sensor placement at which $I(\Delta x)$ reaches a minimum value and plateaus. The results of their calculation are shown in Figure 3.10.

![Graph showing $I(x)$ vs. $x$](image)

Figure 3.10 Distance between measurement points versus average mutual information of points [Trendafilova et al., 2001]

The researchers validated their recommendation by examining three different test setups as shown Figure 3.11. In the first setup, sensors were placed at a minimum distance. The second setup had the sensors placed at the calculated optimal distance, and the third set had spacing of sensors exceeding the optimal distance. They found that the third test setup could not detect any damage, while the first and second test setups successfully found the damage location in the plate.
The same approach developed by Trendafilova et al. (2001) was used in this study to determine the accelerometer spacing. The optimal distance was calculated based on the acceleration responses obtained from FE analysis of the three span structure of Jennette's Pier under a triangular impact load. The acceleration signals were recorded from the same beam that was impacted.

The minimum distance considered for calculating $I(\Delta x)$ was 1 ft. $I(\Delta x)$ was calculated for all twenty sensors that were 1, 2, 3, ... 20 ft from each other. In order to calculate $I(\Delta x)$, the density probability and joint density probability values had to be calculated based on marginal histograms and two-dimensional marginal histograms, respectively. Forty bins were used to construct the marginal histograms and two-dimensional marginal histograms of the calculated acceleration responses. The range of values for each bin depends on the maximum and minimum range of acceleration collected from the FE model at twenty sensors. Probability densities and joint probability densities were calculated by counting the number of acceleration data that falls in each bin with respect to the total number of acceleration data. The calculated values of $I(\Delta x)$ are plotted in Figure 3.12.

Figure 3.11 Three test setups of sensor placement [Trendafilova et al., 2001]
Figure 3.12 The average mutual information based on FE analysis of Jennette's Pier

Figure 3.12 indicates that the average mutual information reaches a minimum at approximately 13 ft spacing. Clark (2012) chose a spacing of 5 ft for the field testing implemented on the Northeast Creek Bridge, which was proven capable to detect damage. Considering the limited capacity (16 channels maximum) of the data acquisition system and the requirement of the general test plan that the acceleration response from the two spans should be collected at the same time, a spacing of 8 ft was chosen to ensure no important information will be neither and duplicated.

Since all three spans have the same length of 40 ft, a total of 12 sensors are required for two spans, as shown in Figure 3.13. Each span has one sensor close enough to the intermediate support pile location to ensure the continuity of the extracted mode shape curves. After finishing Test AB, accelerometers 7, 8, 9, 10, 11, 12 were kept on Span B while accelerometers 1, 2, 3, 4, 5, 6 were moved to Span C. Red arrows indicate the impact locations between every two sensors.
Figure 3.13 Sensor mounting locations for the two tests

The impact testing was performed on the north side of the pier, as shown previously in Figure 3.2. According to the research done by Elsaid (2011) on the comparison of behavior from beams on either side of a symmetrical laboratory structure, which showed no significant difference. It was assumed the dynamic characteristics would be the same if the beam on the south side was instrumented and impacted.

Figure 3.14 shows the mounting of accelerometers on the side of the beam using hot glue. The coaxial cable was taped to the side of the beam to reduce extra frequency content from movement of the cable as well as to prevent the sensor from falling into the sea.
3.4.3 Data Acquisition System

The field testing required a data acquisition system to collect the signal from the accelerometers and load cell of the impact hammer. A National Instruments high channel industrial platform module was used to measure the simultaneous dynamic response from 12 accelerometers due to impact load. The chosen system was comprised of a NI PXI-4496 data acquisition module encased by a rugged chassis from National Instruments, model NI PXI-1033. The module and the chassis are shown in Figure 3.15 (a) and (b), respectively.
The NI PXI-4496 module has the capacity of sampling analog data from 16 input channels simultaneously at a sampling rate up to 204.8 kHz. The module was configured with a 24-bit Analog to Digital Convertor (ADC) which allows for high resolution measurements. The data acquisition system was also configurable with software to measure the input from ICP accelerometers by allocating a 4 mA excitation current. The PXI-4496 data acquisition module has variable anti-aliasing filters to filter unnecessary frequency content of the measured vibration responses due to digital sampling of the analog signal.

The NI PXI-1033 5 slots chassis is capable of simultaneously running 5 different peripheral modules. Only one slot was used to run the one module in this study. The chassis also includes an integrated MXI-express controller card that can be plugged into a laptop, which allows the computer control of the data acquisition system through software, as well as receive and store the sampled analog data from testing. Figure 3.16 shows the laptop and the data acquisition system during testing.
NI LabVIEW software v8.5 was used to control the measurement and transfer the data to the laptop. Also, the excitation current input calibrations for each accelerometer were configured through the LabVIEW interface. In this study, the LabVIEW interface is designed to process the measured acceleration in the time domain. The measured dynamic responses of the structure were stored in an ASCII text format on the laptop. The sampling frequency used was 20 kHz.

3.4.4 Impact Hammer

The VBDD technique requires collecting horizontal accelerations and corresponding impact loading. The large-sledge impulse hammer used to excite the superstructure is shown in
Impact hammer PCB 086D50 was chosen in this research since it was the same impact hammer used by Mosavi (2010) and Clark (2012). The 12 lb hammer has a load cell tip that can produce a force of 5000 lbf and was capable to excite the structure to a frequency range of 5 kHz. The hammer has a sensitivity range of approximately 1 mv/lbf. The hammer was connected to the NI PXI data acquisition module through a low noise coaxial cable to transfer and store the measured analog signal data on the computer along with the correlated accelerations during the time of recording.

3.5 Field Testing

The superstructure must be impacted in the horizontal direction to collect the dynamic characteristics to apply the VBDD technique. The field testing was conducted on April 4th
2014 and the wind at Nags Head was 8 mph. As designed at the beginning of the experimental program, Test AB was conducted on Spans A and B first in the morning from 10:30 am to 11:30 am, and Test BC was conducted on Spans B and C later in the afternoon from 2:30 pm to 3:30 pm. The temperature of the Pier at impact locations increased from 70°F in the morning testing period to 75°F in the afternoon testing period. It should be noted that the accelerometers, impact hammer, and data acquisition system all have specifications that are valid at 70°F. The testing temperatures in the afternoon slightly exceeded the optimal temperatures for the equipment to perform at calibrated specifications.

As shown in Figure 3.13, either in Test AB or Test BC, there are 12 accelerometers mounted on the side of outer beam on the north side. Because of symmetry of these three spans, the pier was impacted only along Span B, in the middle of every two sensors, between sensors 7 and 8, 8 and 9 ... 11 and 12. At each of the 5 impact locations at least 6 tests were performed to ensure sufficient data was collected. In each test, five impacts were made at the impact location during a 10 second period. Duron et al’s (2005) suggested to perform multiple impacts when collecting data from a structure to determine dynamic characteristics since a longer signal can ensure a full range of frequency response.

Figure 3.18 shows that it was possible to perform the impact loading on the side of superstructure by reaching over the railing.
Figure 3.18 Impact illustration on the side of a beam at Jennettes' Pier

Figure 3.19 shows a sample signal from Test AB for impact load between sensors 7 and 8 and acceleration response from sensor 7. Care should be taken to impact the side of the beam perpendicular to the face, or else it will cause the acceleration response not proportional to the magnitude of impact load, as shown in Figure 3.19. The maximum acceleration response is less than the range of the selected accelerometer (±5 g), which shows that the accelerometer selection was appropriate.
Figure 3.19 Sample (a) Impact load between sensors 7 and 8, and (b) acceleration response at sensor 7

For comparison, Figure 3.20 shows the acceleration response recorded from the furthest sensor 1 from Test AB, which has a maximum value of 0.15 g, which is about five hundred times larger than the noise level, as shown in Figure 3.21. Thus noise in this study will not significantly influence the test results.
Figure 3.20 Acceleration response recorded from sensor 1 from the impact test taken between sensors 7 and 8 of Test AB

Figure 3.21 Typical noise recorded during the test

3.6 Signal Processing

The data produced by the accelerometers and the impact hammer due to the horizontal impact is actually a voltage signal in time domain. This data collected through the
data acquisition system was then processed through LabVIEW which converted the voltage to acceleration or force based on the calibration values for every sensor and the impact hammer. The acceleration and corresponding impact load in the time domain were used to extract the dynamic characteristics of the pier by the following three steps.

### 3.6.1 Step 1: Frequency Response Function (FRF)

Frequency response functions (FRF) are frequently used in signal processing, particularly in this study if is used to determine the mode shapes and corresponding natural frequencies. Given a single input and output system as shown in Figure 3.22, the frequency response function $H(\omega)$ is defined as the ratio of the Fourier Transform of the system output $v(t)$ or response to the system input $u(t)$ or excitation as shown in Equation 3-4.

$$H(\omega) = \frac{V(\omega)}{U(\omega)}$$

where $V(\omega)$ is the Fourier Transform of the system output $v(t)$, and $U(\omega)$ is the Fourier Transform of the system input $u(t)$.

![Figure 3.22 Single input/output system](image)

FRF can also be used for nonlinear or time-variant systems, which will change the unique FRF to a function of the nonlinear system or a function of time in time-variant systems. For these systems, a more accurate Equation 3-5 can be used to calculate FRF as
the ratio of the cross-spectrum between the input and output signals to the power spectrum of the input signal.

\[ H(\omega) = \frac{G_{uv}(\omega)}{G_u(\omega)} \]  

Equation 3-5

where \( G_{uv}(\omega) \) is \( U^*(\omega)V(\omega) \), that is cross-spectrum between \( u(t) \) and \( v(t) \) while \( G_u(\omega) \) is \( U^*(\omega)U(\omega) \), that is the power spectrum of \( u(t) \). \( U^*(\omega) \) is the complex conjugate of \( U(\omega) \).

The advantage of Equation 3-5 can be seen in the practical situation where a single input/single output system encounters noise at both input and output signals, as illustrated in Figure 3.23.

![Figure 3.23 Single input/single output system with noise](image)

In this practical situation of single input/single output system with noise, the FRF can be calculated as Equation 3-6.

\[ H'(\omega) = \frac{Y(\omega)}{X(\omega)} = \frac{V(\omega) + N(\omega)}{U(\omega) + M(\omega)} \]  

Equation 3-6

where the upper case letters are the Fourier Transform of the corresponding time domain data.

In that case, the measured FRF could be a good approximation of the true FRF based on the assumption that the noise input and noise output are relatively small compared to the input and output signals. Equation 3-6 can be rewritten as Equation 3-7 by multiplying the
numerator and denominator by the complex conjugate of $X(\omega)$ given that $m(t)$ and $n(t)$ are non-coherent with each other and with the input signal $u(t)$.

\[
H'(\omega) = \frac{G_{uv}(\omega)}{G_u(\omega) + G_m} = \frac{H(\omega)}{1 + \left(\frac{G_m(\omega)}{G_u(\omega)}\right)}
\]

Equation 3-7

where $H(\omega)$ is the true frequency response function, and $\frac{G_m(\omega)}{G_u(\omega)}$ is the noise-to-input signal ratio. When this ratio approach zero, the measured frequency response function, $H'(\omega)$, will be approximately the true frequency response function $H(\omega)$.

For those systems with multiple input/outputs, such as the field monitoring in this study, the measured frequency response for excitation at location $p$ and response measured at location $q$ can be expressed as  Equation 3-8

\[
H'(\omega) = \frac{V_q(\omega)}{U_p(\omega)}
\]

Equation 3-8

where $V_q(\omega)$ is the Fourier Transform of the acceleration measured at location $q$ and $U_p(\omega)$ is the Fourier Transform of the impulse loading at location $p$ (Halvorsen and Brown, 1977).

The calculation of the FRF of the recorded signals were processed through MATLAB. The MATLAB program has a sampling rate of 20kHz, the same as the sampling rate used in LabVIEW. The frequency response function is a complex number comprising of magnitude
and phase angle. The MATLAB function `tfestimate` was used to find the transfer function estimate for the input and output signal as a linear, time-invariant function, which is the ratio of the cross power spectral density and the power spectral density. The magnitude and the phase angle of the `tfestimate` function can be used to derive the mode shapes and corresponding natural frequency. An example of a calculated FRF magnitude plotted versus frequency up to 100Hz is presented in Figure 3.24, the case shown is from Test AB with the impact at location between sensors 7 and 8.

![Figure 3.24 Magnitude of FRF plotted versus frequency up to 100Hz from all 12 sensors with impact location between sensor, 7 and 8 from Test AB](image)

3.6.2 Step 2: Natural Frequencies and Mode Shapes

In civil engineering, the peak-picking method is the most widely used method to extract the natural frequencies and mode shapes of a structure excited by an impact load
(Zhou, 2006), mainly due to its simplicity and accuracy. This method identifies natural frequencies as the peak values of a Fourier response spectrum plot. The theoretical detailing is introduced in the following.

The Fourier Transform of the displacement for a single degree freedom system undergoing forced vibration can be expressed as Equation 3-9 (Clough and Penzien, 1975).

\[
X(\omega) = A(\omega) \cdot P(\omega)
\]

Equation 3-9

where \(\omega\) is the frequency of the excitation force, \(P(\omega)\) is the Fourier Transform of the excitation force and \(A(\omega)\) is the complex dynamic amplification factor of the single degree freedom system. \(A(\omega)\) can be calculated as Equation 3-10

\[
A(\omega) = \frac{1}{1 - \left(\frac{\omega}{\omega_o}\right)^2 + 2\xi \left(\frac{\omega}{\omega_o}\right) \cdot i}
\]

Equation 3-10

where \(\omega_o\) is the natural frequency of the system and \(\xi\) is the damping ratio of the system. \(\frac{\omega}{\omega_o}\) is a frequency ratio of excitation force and system

The magnitude of \(A(\omega)\) is shown in Equation 3-11.

\[
|A(\omega)| = \frac{1}{\sqrt{1 - \left(\frac{\omega}{\omega_o}\right)^2}^2 + \left[2\xi \left(\frac{\omega}{\omega_o}\right)^2\right]^2}
\]

Equation 3-11

The magnitude \(A(\omega)\) at different frequency ratio is illustrated in Figure 3.25. The peak in the dynamic amplification factor curves occur when frequency ratio \(\frac{\omega}{\omega_o} = \sqrt{1 - 2\xi^2}\), which is given by Equation 3-12.
\[ |A(\omega)|_{\text{max}} = \frac{1}{2\xi \sqrt{1-\xi^2}} \approx \frac{1}{2\xi} \]

Equation 3-12

Figure 3.25 Dynamic amplification factor magnitude at different frequency ratio and damping ratios (Zhou 2006)

The range of damping ratios for steel and concrete structures in civil engineering is usually between 0.5% and 2%, that gives frequency ratio at the peak ranging from 0.999975 to 0.9996. This shows that the location of the peak in Figure 3.25 is very close to the natural frequency of the system. However, in this study, the dynamic amplification factor remains unknown; instead, the frequency response function was derived from the field test. The same peak-picking method still works as indicated by Equation 3-10, that the Fourier Transform of the displacement of the system (i.e. the response of the system) is equal to the product of the Fourier Transform of the excitation and the dynamic amplification factor. Based on this method, those peaks in the frequency response functions indicate natural frequencies of the structure. Then the same frequency value can be used to plot the un-normalized mode shape based on the frequency response functions from all sensors.
As mentioned previously, the FRF is a complex number comprised of a magnitude and phase angle. The magnitude part of the FRF from one sensor is same as the magnitude of the mode shape at the sensor location. The phase angles sign indicates the sign of the mode shape at the sensor location. Figure 3.26 shows the magnitude and the phase angle of the FRF for sensors 2, 4, and 6 when subject to an impact load between sensors 7 and 8 during Test AB. The magnitude of the peaks of the FRFs for different sensor locations are same as the un-normalized mode shape of the structure at the corresponding frequency as shown in Figure 3.27. In this case, the mode shape at sensors 2, 4, and 6 is positive and above the x axis.
Figure 3.26 FRF magnitude and phase angle versus frequency for sensors 2, 4, and 6 when subject to impact load between sensors 7 and 8 from Test AB.
The application of the peak-picking method shown in Figure 3.26 and Figure 3.27 is straightforward. However, the frequency response function contains numerous local peaks, as shown in Figure 3.24, which include FRFs of 12 sensors from one impact test. As mentioned previously, at least six impact tests were performed at each impact location, resulting in 30 tests for both Test AB and Test BC. Manually determining peak value frequencies is extremely time consuming due to large amount of data and many "false" peaks.
Thus, the histogram method from statistics was applied here to reduce the effort of the peak-picking method by counting the accumulated occurrences of local peaks of FRFs from all sensors at each impact location. Figure 3.28 gives a sample histogram to identify possible "true" FRF peaks that correspond to horizontal mode shapes.

![Histogram plot of local FRF peaks from all 12 sensors of one test at impact location between sensors 7 and 8 during Test AB](image)

Figure 3.28 Histogram plot of local FRF peaks from all 12 sensors of one test at impact location between sensors 7 and 8 during Test AB

The labeled value in Figure 3.28 are close to the true frequencies of first five mode shapes for Span AB. Other histogram bars are likely due the local peaks coming from noise or mode shapes involving partial horizontal movement. This histogram application significantly reduced the time effort of the peak-picking method by predicting all potential value ranges of natural frequencies statistically. Then the natural frequencies and mode shapes can be obtained for each test at every location. The average value from six repetitive tests at each impact location was calculated to reduce the uncertainties in dynamic data. However, the final result of natural frequencies and mode shapes is not simply the average of the 30 test
results per test setup. More discussion of the influence from impact location and the results of natural frequencies and mode shapes are presented in Chapter 4.

3.6.3 Step 3: Mass-Normalizing the Mode Shapes

As known in dynamics, the amplitude of the mode shape is always indeterminate. However, the shape of the mode, as the ratio between different measurement locations, is determinate. In order to compare relative change in mode shapes from Test AB and Test BC, normalization must be performed for the mode shapes. One common method is to normalize the mode shapes so that the largest value is unity (Chopra, 2007). However, this normalization method is not reasonable if the largest value of the mode shape occurs at the damage location, which would result in no change of mode shape at the damage location.

Mass normalization is widely adopted in theoretical calculation, which use the mass matrix $[m]$ to normalize mode shapes, as shown in Equation 3-13:

$$\phi_i^T [m] \phi_i = 1$$  \hspace{1cm} \text{Equation 3-13}

where, $\phi_i$ is the $i$th mass-normalized mode shape. Then the mass-normalized mode shapes can be calculated as Equation 3-14:

$$\bar{\phi}_i = \frac{1}{\sqrt{\phi_i^T [m] \phi_i}} \phi_i$$ \hspace{1cm} \text{Equation 3-14}

where, $\bar{\phi}_i$ is the $i$th un-normalized mode shape. In this study, the mass matrix was simplified to be a lumped mass matrix. After extracting the natural frequencies and the mass-normalized mode shapes, the flexibility-based deflection was calculated. The details of the experimental results and discussion are presented in Chapter 4.
Chapter 4. Analysis of Experimental Results and Discussion

4.1 Introduction

As described in Chapter 3, the field testing was performed on Span AB and Span BC of Jennette's Pier, as indicated in Figure 3.13. The intermediate pile group of Span BC was believed to have a longer height than the pile group of Span AB, by approximately 1 ft. Considering Span AB and Span BC have identical superstructure and connection design, the dynamic characteristics extracted from the field data from Test AB and Test BC is expected to be different due only to the difference in pile height. For this reason, Span AB and its intermediate pile may be assumed as the unscoured reference condition while Span BC and its intermediate pile is assumed as the scoured condition.

This chapter presents the extracted modal properties including natural frequencies and horizontal mode shapes for both Span AB and Span BC. As the only damage feature in this study, the flexibility-based deflection is calculated based on these extracted modal properties, which has been validated to be able to identify the existence, location and possibly extent of scour from Elsaid (2011) and Clark (2012). In this study, the primary expectation of this damage feature is reasonable indication of the different intermediate pile heights of Span AB and Span BC.

4.2 Extracted Modal Properties

The peak picking method is used to determine natural frequencies based on Frequency Response Functions (FRFs) calculated from the excitation loads and acceleration response for each impact test. By finding peaks of the FRFs from all accelerometers locations for each impact test, the vibration mode shapes of all sensor nodes can be obtained. In this
study, only the first five horizontal mode shapes are calculated. A sample of determining the second mode shape was presented in Figure 3.26 and Figure 3.27.

As mentioned in Chapter 3, the natural frequencies and mode shapes from six repetitive tests at each impact location were averaged to reduce the uncertainties in the dynamic data, such as environmental condition and sensor attachment to the bridge. But environmental condition such as wind and wave should not affect the data greatly since the noise signal level is much smaller compared to the test signal, as was shown in Figure 3.20 and Figure 3.21. The issue that exists in the attachment of the accelerometers to the concrete beam is the non-uniformity among the different accelerometers, which comes from the hot glue used to attach the accelerometer, and the unevenness of the concrete surface. Thus the average process is required for the above consideration.

Elsaid (2011) took the procedure to average all results of natural frequencies and mode shapes obtained from each test at different impact locations, which is reasonable since the test subject is a small scale two-span laboratory steel structure with both abutments fixed in the horizontal direction, and his test results does not show significant difference in mode shapes from different impact locations. Clark (2012) also applied this average method on the data from the test results from Northeast Creek Bridge. The tests were performed for one span at a time, and the mode shape curves was connected from the results of the two spans. Therefore, her primary objective during mode shape extraction was the connection of the curves and identification of modes. Without awareness of influence from impact location on mode shape, 'inappropriate' mode shape results were discarded.

The fundamental difference from monitoring on Jennette's Pier to the two span laboratory steel structure constructed by Elsaid (2011) is that the current tests were
conducted on only three spans of a continuous bridge structure. During the modal properties extraction process, the impact locations were found to have large influence on the mode shapes but little on natural frequencies. The results of natural frequencies and mode shapes, along with the discussion on impact location are presented in the remainder of the chapter.

4.2.1 Natural Frequencies

Table 4.1 shows the natural frequencies for first five mode shapes extracted from tests on Span AB and Span BC. The natural frequencies calculated from the FE model of Jennette's Pier are also included for comparison. The FE model is the same one created in Chapter 3.

<table>
<thead>
<tr>
<th>Test Span</th>
<th>1&lt;sup&gt;st&lt;/sup&gt; mode</th>
<th>2&lt;sup&gt;nd&lt;/sup&gt; mode</th>
<th>3&lt;sup&gt;rd&lt;/sup&gt; mode</th>
<th>4&lt;sup&gt;th&lt;/sup&gt; mode</th>
<th>5&lt;sup&gt;th&lt;/sup&gt; mode</th>
</tr>
</thead>
<tbody>
<tr>
<td>Span AB</td>
<td>1.984</td>
<td>11.510</td>
<td>17.955</td>
<td>41.504</td>
<td>70.038</td>
</tr>
<tr>
<td>Span BC</td>
<td>1.831</td>
<td>11.342</td>
<td>18.158</td>
<td>41.199</td>
<td>70.648</td>
</tr>
<tr>
<td>FE Model</td>
<td>1.578</td>
<td>7.888</td>
<td>15.855</td>
<td>29.360</td>
<td>35.900</td>
</tr>
</tbody>
</table>

The natural frequencies calculated from the FE model in SAP 2000 are much lower than the experimental results for every mode, which indicates that the FE model is overestimating the stiffness of the structure by assuming stiffer abutments and constraints than reality. Also, the pile height may not be the same as the value assumed in the analysis. According to the rough estimation by dropping the steel block to the seabed on the test date, the pile height increased by 10 ft since the inspection performed in 2012 as part of the Maintenance Lift Project for the Coastal Studies Institute at Jennette's Pier. But it should be...
noted the FE model is used here to help understand the horizontal mode shapes, thus an accurate match of FE and experimental results are not critical. The trends of mode shapes are presented in the next section.

As expected, the natural frequencies of each mode from Span AB and Span BC only have very small difference (less than 1%), which is reasonable since these three adjacent spans are a portion of a continuous structure. This also shows that the slightly variation in temperature from morning to afternoon does not alter the dynamic behavior of the structure. However, similar natural frequencies does not mean the mode shapes of the two spans are identical since mode shapes are the relative dynamic movement of different nodes, which is related to the stiffness and mass allocation in the structure.

4.2.2 Mode Shapes

The mode shapes are one of the fundamental dynamic characteristics of a structure. The mode shapes in this study refer to the horizontal direction. As mentioned, the FE model of the three span structure from Jennette’s Pier was created based on the observation during testing that the impact performed in Span B will only cause significant vibration of the adjacent spans. For same reason, the mode shapes obtained experimentally from each two spans can be connected to get a complete mode shape for the three span model.

Figures 4.2 - 4.6 show schematics of the first five horizontal mode shapes, along with the mass-normalized mode shapes from Span AB and Span BC, respectively, in each figure. To be noted, the mode shapes presented in these figures are obtained from the impact test at the location most closest to the intermediate support piles, as indicated by arrows in Figure 4.1. Also, the mode shapes obtained from experimental results of each two spans are not
connected here since it is not the methodology taken in this study, and it is just shown for illustration of the mode shapes.

Figure 4.1 Impact location close to intermediate support corresponding to the mode shapes shown for Test AB and Test BC (Figures 4.2 - 4.6)
Figure 4.2 First horizontally displaced mode shape: (a) FE schematic mode shape (b) Calculated mode shape from Span AB and Span BC
Figure 4.3 Second horizontally displaced mode shape: (a) FEM schematic mode shape (b) Calculated mode shape from Span AB and Span BC
Figure 4.4 Third horizontally displaced mode shape: (a) FEM schematic mode shape (b) Calculated mode shape from Span AB and Span BC
Figure 4.5 Fourth horizontally displaced mode shape: (a) FEM schematic mode shape (b) Calculated mode shape from Span AB and Span BC
Figure 4.6 Fifth horizontally displaced mode shape: (a) FEM schematic mode shape (b) Calculated mode shape from Span AB and Span BC
4.2.3 Influence of Impact Location

In this study, the impact is performed on Span B, which is the middle of three spans of a continuous structure while the accelerometers were mounted over each two span for two tests. Therefore only the mode shapes from two spans are calculated from the recorded signal. It should be highlighted that the impact location does not change modes, which can be seen from the constant value of natural frequencies from Table 4.1. However, the impact location will cause the accelerometers to capture different parts of the mode shape information. In this study, the mode shapes extracted from tests at the furthest impact location from the intermediate support are shown in Figures 4.8 - 4.10. The impact locations are shown in Figure 4.7.

![Figure 4.7 Impact location furthest from the intermediate support corresponding to the mode shapes shown for Test AB and Test BC (Figures 4.8 - 4.10)]
The first mode is also the called fundamental mode, which could be dominate mode in the dynamic behavior for many structures. The mode shape is close to the FE schematic when the impact tests is performed at locations indicated in Figure 4.7. Again, the first mode shape shown in Figure 4.8 is the same as the result presented in Figure 4.2. But the difference in shape reflects that the capture of vibration response by sensors is different due to impact at different locations. In this study, the symmetrical shape in Figure 4.2 is desired to calculate the flexibility deflection as the pile height of each two spans is under investigation.

![Graph](image)

Figure 4.8 First mode shape for Span AB and Span BC when impacted furthest from the intermediate support

Figure 4.9 and Figure 4.10 presents the second and third mode shapes extracted from impacts at locations shown in Figure 4.7, respectively. Since the pile is at the stationary node, the second mode shape extracted from tests at different impact locations does not show much variation compared to the one presented in Figure 4.3. The third mode shape is also found to be insensitive to impact location changes by comparing Figure 4.10 with Figure 4.4.
Figure 4.9 Second mode shape for Span AB and Span BC when impacted furthest from the intermediate support

Figure 4.10 Third mode shape for Span AB and Span BC when impacted furthest from the intermediate support
The fourth and fifth mode shapes can only be determined based on the tests at the impact location close to the intermediate support. This is likely because the impact at the furthest location is not enough to cause significant vibration of adjacent spans for higher modes. As observed in Figure 4.5 and Figure 4.6, the portion of the mode shape at Span A and C are relatively less than that of Span B. This was never experienced in Elsaid's (2011) small scale test and Clark (2012) also did not encounter this problem with her methodology of connecting mode shape curves from two span test.

Based on the above discussion, the impact location only has a major influence on the first mode shapes. Thus only those mode shapes presented above from impact tests at intermediate support locations are considered as desired mode shapes in this study and used to calculate the flexibility-based deflection.

The trends of mode shapes from tests performed at locations between the closest and furthest end are similar. Considering the purpose of this study is to indicate which intermediate pile has greater height by comparing the flexibility-based deflection from Span AB and Span BC. It is reasonable to just use the mode shape extracted from the impact test performed closest to the intermediate support. Also, a recommendation for future testing in a bridge structure like this, the impact location should concentrate on the intermediate supports.
4.3 Damage Feature

Among several damage features developed by Elsaid (2011) to detect scour damage, the change in flexibility-based deflection is capable of reveal the existence, location, and possibly the extent of scour according to Elsaid (2011) and Clark (2012). Thus this damage feature is selected in this study. As mentioned in Chapter 3, this experimental program is designed to examine the ability of flexibility-based deflection to differentiate which substructure has greater stiffness, or shorter pile height.

4.3.1 Flexibility-Based Deflection

The flexibility matrix is defined as the inverse of the stiffness matrix. The measured flexibility matrix \([G]\) could be calculated from the mass-normalized mode shapes \(\phi\) and natural frequencies (Doebling et al., 1996) as Equation 4-1

\[
G \approx [\phi] \cdot [\Lambda]^{-1} \cdot [\phi]^T
\]

Equation 4-1

where, \([\Lambda]^{-1}\) is a diagonal matrix containing the reciprocal of the square of the natural frequencies in ascending order. The approximate estimation of Equation 4-1 for \([G]\) is because of the fact that only the first few modes are measured. Equation 4-1 can be rewritten in the form shown in Equation 4-2.

\[
G \approx \sum_{i=1}^{n} \frac{1}{\omega_i^2} \{\phi_i\} \{\phi_i\}^T
\]

Equation 4-2

where \(\{\phi_i\}\) is the \(i\)th mass-normalized mode shape, and \(\omega_i\) is the \(i\)th modal frequency. The first \(n\) modes are considered in the calculation. Then the flexibility-based deflection is determined by considering the deflected shape due to a uniform static load which is equal to the flexibility matrix multiplied by a unit load vector. In this study, the flexibility deflection is calculated for Span AB and Span BC.
4.3.2 Contribution of Modes

Although the calculation of the flexibility matrix is an approximation from the first few modes, it was shown that the flexibility matrix converges rapidly with increasing values of frequency (Pandey and Biswas, 1994). They attributed this to the inverse relation between the flexibility matrix and the square of the natural frequencies. Therefore the approximation from a few of the lower frequency modes provides an accurate estimation of \([G]\).

Elsaid (2011) considered the first few modes equally important and suggest use of at least the first five modes to determine the flexibility-based deflection. That is due to the small-scale laboratory two span steel structure having relatively close values of natural frequencies for the first few modes. The natural frequency of the fifth horizontal mode is less than three times greater than the first mode.

However, in this study, the natural frequencies from the second mode to the fifth modes are at least six times greater than the fundamental frequency. A contribution factor \(C\) is defined to represent the contribution from higher modes other than fundamental mode. Since the mode shapes were mass normalized in the same magnitude, the contribution factor \(C_i\) of the ith mode could be simply estimated as the square ratio of the ith mode frequency to the fundamental frequency, as shown in Equation 4-3.

\[
C_i = \left(\frac{\omega_i}{\omega_1}\right)^2 \times 100
\]  

Equation 4-3

In this study, the contribution factors \(C_i\) for modes other than then fundamental mode are calculated based on the natural frequencies of Span AB shown in Table 4.1. The results of \(C_i\) are presented in Table 4.2 as a percent.
Table 4.2 Contribution factor for second to fifth modes

<table>
<thead>
<tr>
<th></th>
<th>2\textsuperscript{nd} mode</th>
<th>3\textsuperscript{rd} mode</th>
<th>4\textsuperscript{th} mode</th>
<th>5\textsuperscript{th} mode</th>
</tr>
</thead>
<tbody>
<tr>
<td>$C_t$</td>
<td>2.5%</td>
<td>1.0%</td>
<td>0.19%</td>
<td>0.07%</td>
</tr>
</tbody>
</table>

From Table 4.2, the higher modes in this study only represent 3.76\% contribution to the flexibility matrix. It also should be noted that the intermediate support pile for both Span AB and Span BC is on the stationary nodes of second and fourth mode shape, which means that these mode shapes are insensitive to change in pile height. The final results of the flexibility-based deflection for Span AB and Span BC are shown in following.

4.3.3 Comparison of Flexibility-Based Deflection

As mentioned previously, the mode shapes obtained from impact tests at locations close to the intermediate support are used to calculate the flexibility-based deflection for Span AB and Span BC. Figure 4.11 show the results considering different modes. For comparison of Span AB and Span BC, the flexibility deflection is plotted versus the same distance since these spans have same length and same accelerometer setup.
Figure 4.11 Flexibility deflection (a) considering first five modes (b) considering only the fundamental mode

No significant difference (less than 3%) is observed between the results by considering the first five modes and only considering the fundamental mode. Therefore it is acceptable to just use the first horizontal mode shape to calculate the flexibility deflection when the contribution factor for higher modes are relatively low, such as in this application.

The change in flexibility-based deflection was used by Elsaid (2011) and Clark (2012) to determine the location of scour damage because the magnitude reaches a maximum at the location of scour. In this study, the difference between flexibility-based deflection from Span AB and Span BC is used to indicate which support piles have a greater height. As mentioned in Chapter 3, the piles of Span BC have greater height due to the seabed slope, which is verified by rough estimation of dropping a steel block along side of the pier. Thus the flexibility deflection of Span BC is expected to be greater than that of Span AB due to the more flexible pile support, which can be seen easily in Figure 4.12.
Figure 4.12 The difference in flexibility-based deflection from Span AB and Span BC

4.4 Summary

From the comparison of flexibility-based deflection of Span AB and Span BC, it could be seen that this damage feature is able to differentiate varying pile height of intermediate supports. In other words, this implementation on Jennette' Pier once again validated the VBDD technique based on the change in flexibility-based deflection. This conclusion is made based on the assumption that the pile height is the only difference for Span AB and Span BC, which is reasonable due to the identical design of the pier.

Impact location is also found to have influence on fundamental mode shapes. As a strategy to examine the scour condition of similar multiple span bridge structures, the impact test should be performed at a location close to the intermediate support to obtain the desired mode shape as shown in Figures 4.2 - 4.6. The contribution from modes in the calculation of flexibility-based deflection depends on the square of the ratio between the frequency of that mode to the frequency of the fundamental mode. The first horizontal mode shape is considered the most critical mode shape in cases such as Jennette's Pier.
Chapter 5. Development of Relationship between Flexibility-Based Deflection and Scour Extent for Rigid Beam-Column Connections

5.1 Introduction

Vibration-based damage detection techniques focus on the dynamic response of the bridge superstructure influenced by the effect of scour at substructure. As introduced in Chapter 2, these techniques can be easily implemented in the field to rapidly assess the bridge condition at a lower cost and without any underwater instrumentation. Elsaid (2011) developed various vibration based damage detection techniques through experimental and analytical investigations. These techniques are based on either change in curvature such as the Curvature Damage Factor, or based on change in flexibility based deflection. Clark (2012) implemented these techniques in the field, by conducting a scour monitoring on Hwy 17 Northeast Creek Bridge in Jacksonville, North Carolina, based on the change of vibration characteristics. She concluded that the method based on the change in flexibility deflection shows better ability than the other methods to identify the damage location and is promising in terms of predicting the extent of scour because the magnitude of deflection increases with scour. Also, as presented in Chapters 3 and 4, the implementation of this VBDD technique on Jennette's Pier has shown the ability to differentiate the extent of intermediate pile height from the superstructure responses of two parts of structure, which is assumed to only have different pile heights. The field test in this study not only succeeded in implementing the current VBDD technique on a coastal structure, but also demonstrated a feasible way to examine possible scour locations in a multiple span bridge structure by conducting the field monitoring on every two span of the bridge.
Predicting the extent, or severity, of damage is the next goal in the structural health monitoring diagnostic process. However, as a matter of fact, there is currently no theory to determine the extent of scour directly from the response of superstructure, which leads to the current research. The primary research objective is to develop a relationship between the amount of scour and the selected damage indicator, namely the change in flexibility-based deflection. This relationship is considered as the last critical piece of a scour diagnosis framework. In this research, the general procedure to develop this relationship is divided into two steps: Chapter 5 which presents the development of a relationship between scour extent and flexibility-based deflection in the idealized case where the connection between superstructure and substructure can be assumed as rigid; and Chapter 6 which considers the influence of connection stiffness on the relationship between damage feature and extent of scour. The connection stiffness has an impact on the dynamic behavior of the superstructure since it will affect the total stiffness of the substructure. Chapters 5 and 6 together present the complete development of a relationship between flexibility-based deflection and extent of scour.

5.2 Theoretical Study

The calculation of flexibility-based deflection depends on the flexibility matrix, which is comprised of natural frequencies and mode shapes of the first few modes, as shown in Equation 4-2. Therefore, to develop a relationship between the amount of scour and the damage indicator, a theoretical study regarding the two basic dynamic characteristic, namely natural frequency and mode shape, was undertaken to gain a better understanding of the structural dynamic response influenced by scour.
5.2.1 Eigenvalue Problem

As is well known, the eigenvalue problem can be solved to obtain eigenvalues and eigenvectors. In structural dynamics, the eigenvalue is a natural frequency, and the eigenvector is the mode shape. The eigenvector is a magnitude free quantity and can be normalized by different methods. In the current research, mass normalization is adopted. That is, the mode shapes depend on the proportion of allocation of stiffness and mass in the structural system, rather than the absolute values of stiffness and mass. In other words, the same magnifications in stiffness and mass for all components in the structural system would not change the natural frequencies and the mode shapes. Unlike mode shape, natural frequency is not a magnitude free quantity and is determined by the absolute value of stiffness and mass in addition to the proportion of stiffness and mass in the structural system. This is illustrated in following section.

5.2.2 Lambda, Stiffness ratio, Mass ratio

The eigenvalue problem usually can only be solved through numerical methods in the case of multi-degrees-of-freedom. But it does have an analytical solution for simple structures, such as the Bernoulli-Euler uniform two-span beam with lumped mass M and elastic support of stiffness k, as shown in Figure 5.1. Each span has same length of $L_{\text{span}}$. The beam has a moment of inertia of $I_{\text{super}}$. E is the elastic modulus of beam.
The natural frequency of the beam can be expressed as a function of stiffness and mass $m$ (per unit length) and $\lambda$ which represents the proportion effect from stiffness and mass, as given in following:

$$f_i = \frac{1}{2\pi} \frac{\lambda_i^2}{L_{span}^2} \sqrt{\frac{EI_{super}}{m}}$$  \hspace{1cm} \text{Equation 5-1}$$

where $\lambda$ are roots of the frequency equation. For the two-span model indicated in Figure 5.1, the fundamental frequency equation $(i = 1)$ can be written in the implicit form [Filippov, 1970], given by:

$$\frac{2}{\lambda} \cosh \lambda \cos \lambda \cosh \lambda \sin \lambda - \sinh \lambda \cos \lambda = \alpha - \frac{1}{2\lambda^4} \cdot k^*$$  \hspace{1cm} \text{Equation 5-2}$$

where $\alpha = \frac{M}{ml_{span}}$, $k^* = \frac{kl_{span}^3}{EI_{super}}$ denoted as the mass ratio and stiffness ratio, respectively.

Clearly $\lambda$ is as a function of stiffness ratio and mass ratio. Equation 5-2 is solved for various combinations of stiffness ratio and mass ratio, and the frequency roots are plotted versus stiffness ratio and mass ratio, as shown in Figure 5.2. It can be seen that $\lambda$ is an increasing function of stiffness ratio, and a decreasing function of mass ratio.
Inspired by the two-span beam with lumped mass and vertical spring support, the concept of stiffness ratio and mass ratio is adopted in the current study. The VBDD method uses the change in flexibility deflection damage indicator, which is calculated from the horizontal mode shapes and corresponding natural frequencies. The theoretical model of a bridge subject to scour damage can be simplified into a uniform beam with lumped masses on horizontal elastic supports. In the simplest case, like the two-span idealized laboratory bridge created by Elsaid (2011), the theoretical model is comprised of only two spans, one lump mass in the middle of the beam and one horizontal elastic support, as shown in Figure 5.3. The elastic support comes from the horizontal stiffness of the pile support and the lumped mass also comes from the pile support. It must be mentioned that the reason why the pile support can be simulated as a lumped mass and elastic spring is that the current study focuses on the response of the superstructure, which is influenced by the mass and stiffness.
of the pile support. Also, the lumped mass from the pile depends on the mass allocation in the pile rather than simply considering the total mass of pile. The mass and stiffness allocation in the substructure would also determine the vibration mode shape of the pile, which is beyond the current research scope.

![Figure 5.3 Two-span beam with lumped mass and horizontal elastic support](image)

As is known, scour damage reduces the stiffness of the support pile by increasing the effective height. The experimental works and FEM studies conducted by Elsaid (2011) and Clark (2012) both showed that the flexibility-based deflection increases while the pile height increases.

Based on the above discussion of frequency roots $\lambda$, it can be found that the value of $\lambda$ becomes smaller while the stiffness ratio decreases or mass ratio increases, which could happen at the same time due to the pile height increasing when scour damage occurs. According to Equation 5-1, the natural frequency would become smaller since $\lambda$ decreases. This is also verified by Elsaid (2011) and Clark (2012). As for mode shapes, or the eigenvectors in the modal analysis problem, which could be determined by substitute the
corresponding eigenvalues into the modal equations. The scour damage would change the vibration shape along the nodes of the superstructure, rather than the magnitude since the mode shapes are indeterminate quantities. Therefore, the influence of scour on mode shapes are not as straightforward as natural frequencies. It has to be noted here that either natural frequency or mode shape alone cannot be a good damage indicator, as explained in Chapter 2.

5.2.3 Assumptions

Based on the above discussion on natural frequency and mode shape, it is proposed that similar structures should have similar dynamic responses. If the stiffness ratios and mass ratios between structural components in each structural system is identical, then the mode shape and $\lambda$ from different structural systems should be the same. Under this assumption, a generalized damage indicator for consistent comparison is possible for all kinds of bridge structures subject to scour damage including different materials, structural types, sections, or any other factors influencing the stiffness ratio and mass ratio.

However, the natural frequencies from different structural systems, according to Equation 5-1, are different since the absolute value of stiffness and mass govern as well. On the other hand, certainly it is inappropriate to specify one numerical model for one particular structure to relate the change in flexibility deflection to the extent of scour. Therefore, there is a need to develop a generalized model regardless of type of structure and material to quantify the relative effect of scour on structural dynamic behavior. In other words, the flexibility-based deflection needs to be modified or normalized in a way to represent a consistent damage indicator for all kinds of bridge structures.
5.3 Model Development

Based on the previous theoretical study, the next step is to develop a method to relate the extent of scour to the flexibility deflection. Following the assumption that similar structures should have similar dynamic response, the new relationship can theoretically be used for any type of bridge structure, although for now, all the work is restricted to symmetrical two-span bridges. The vibration based damage detection technique based on the change in flexibility deflection has been shown capable of identifying the scour location, which is at the middle point of the bridge. Thus, current analysis mainly focuses on the magnitude of flexibility deflection at the middle of the superstructure. As mentioned previously, the connection between the superstructure and the substructure is assumed rigid in Chapter 5. A numerical relationship is proposed to predict the effective height based on the flexibility deflection value at the scour location for this rigid connection case.

5.3.1 Modified Deflection

The flexibility based deflection is calculated by multiplying the flexibility matrix with a unit load vector. By substituting the natural frequency from Equation 5-1, the flexibility based deflection can be expressed by the following:

$$D = 4\pi^2 \cdot \frac{m}{EI_{super}} \cdot L_{span}^4 \cdot \left[ \sum_{i=1}^{n} \frac{1}{\lambda_i} \phi_i \phi_i^T \right] \{load\}$$  \hspace{1cm} (5-3)

where $\lambda_i$ is the root of the frequency equation for the $i$th mode; $I_{super}$ is the moment inertia of the superstructure in the horizontal direction; and $L_{span}$ is the length of one span.

Equation 5-3 indicates that the mass and stiffness from the superstructure also determine the magnitude of flexibility deflection besides $\lambda_i$ and $\phi_i$. As discussed previously,
is a function of stiffness ratio and mass ratio. And, \( \phi_i \), as the \( i \)th mass-normalized mode shape, also only depends on these two ratios.

Therefore Equation 5-3 can be normalized with mass and stiffness from the superstructure to obtain a quantity which only depends on these two ratios. This modified flexibility deflection is expressed as the following:

\[
D' = D \cdot \frac{EI_{\text{super}}}{mL_{\text{span}}^4}
\]  
Equation 5-4

In the following modal analysis, since the same material is used for superstructure and substructure, and length of span is considered as constant, the flexibility deflection can be normalized with only area and moment of inertia, given by following,

\[
D' = D \cdot \frac{I_{\text{super}}}{A_{\text{super}}}
\]  
Equation 5-5

5.3.2 Parametrical Study

As is known, for those structures who have multi-degrees-of-freedom, the eigenvalue problem could be difficult to solve due to the nature of the eigen matrix determinant, which is a high order polynomial, and can only be solved with numerical iteration methods, such as direct iteration and inverse iteration. Thus, the modal analysis is conducted using the finite element software SAP2000 to study the dynamic behavior of bridge superstructures when subject to stiffness and mass changes in the substructure.

An idealized two-span bridge structure is created, with only one beam and only one column. The beam-column connection is assumed rigid for now. For the boundary condition, the bottom of the column is assumed fixed to the ground. The beam is supported by a pin and roller at either end in the vertical plane. Rotations are allowed in both horizontal and vertical
planes, but the horizontal translation movement at the end abutments is prevented. The beam element is used in the finite element model, and the mesh is shown in Figure 5.4.

![Figure 5.4 Idealized two-span bridge structure created for modal analysis](image)

For the purpose of model validation with Elsaid's experimental result [2011], the idealized structure has a similar size in terms of span length and column height. The total span length is constant at 18 ft. But the column has different heights (3.5 ft, 4 ft, 4.5 ft, 5 ft, 5.5 ft, 6 ft) to simulate various scour levels (0 in, 6 in, 12 in, 18 in, 24 in, 30 in). The beam section is selected from 30 W-shape steel sections. While the column section is selected from 10 S-shape steel sections and 10 round HSS steel sections.

These sections are identified in Table 5.1. The round HSS sections can represent those stiff substructures with light-weight because the section has higher stiffness-to-mass ratio compared to S-shapes.
Table 5.1 Beam and column sections used in idealized structural analysis

<table>
<thead>
<tr>
<th>Section type</th>
<th>Section selection</th>
</tr>
</thead>
<tbody>
<tr>
<td>Beam (W-shape)</td>
<td>W18X97; W16X100; W18X86; W16X89; W18X76; W16X77; W16X67; W14X61; W12X53; W21X93; W8X67; W21X83; W8X58; W21X73; W21X68; W8X48; W21X62; W8X40; W21X55; W8X35; W21X48; W8X31; W8X28; W8X24; W8X21; W8X18; W8X15; W8X13; W6X9; W6X8.5</td>
</tr>
<tr>
<td>Column (S-shape)</td>
<td>S24X121; S24X106; S20X96; S20X75; S18X70; S15X50; S12X35; S8X23; S6X12.5; S3X7.5</td>
</tr>
<tr>
<td>Column (round HSS)</td>
<td>HSS5.500X.500; HSS5X.312; HSS5X.375; HSS5X.500; HSS5X.375; HSS4X.313; HSS4.500X.237; HSS4.500X.188; HSS4.500X.125; HSS4X.188</td>
</tr>
</tbody>
</table>

Ideally it is possible to manually vary section properties such as moment of inertia and area for both superstructure and substructure. In that case, there is at least five variables in the parametric study, which could significantly increase analysis time. Inappropriate combinations, such as very flexible substructure, could result in no output for desired horizontal mode shapes of the superstructure. This is because in modal analysis, the calculation limit for mode is set as constant in the parametric study. Once the desired mode passes the calculation limit since the flexible substructure introduces too many local vibration modes in the substructure, these desired mode shapes would not be calculated. Furthermore, SAP2000 has a database for standard sections normally used in industry. Thus, it is convenient to import section properties directly from SAP2000 by section name. In that case, only three variables are considered in the parametric study, which includes the section name for superstructure and substructure, and the pile length. Considering all these combinations of beam and column sections and column height, there are a total of 3600 modal analysis combination. Due to the large amount of modal analyses, a MATLAB program was written...
to process the modeling and calculation via the Application Program Interface (API) provided by SAP2000.

The first five horizontal mode shapes of the superstructure and corresponding natural frequencies were extracted from each modal analysis to calculate the proposed modified flexibility deflection using Equation 5-5. The substructure-to-superstructure stiffness and mass ratios are calculated as the following, given by Equation 5-6 and Equation 5-7, respectively,

\[ k^* = \frac{3EI_{sub} / L_{pile}}{EI_{super} / L_{span}} \]  \hspace{1cm} \text{Equation 5-6}

\[ a = \frac{\rho \cdot A_{sub} \cdot L_{pile}}{\rho \cdot A_{super} \cdot L_{span}} \] \hspace{1cm} \text{Equation 5-7}

where \( A_{sub}, A_{super} \) are sectional area for the column and beam, respectively; \( I_{sub}, I_{super} \) are moment of inertia in horizontal direction for the column and beam, respectively; and \( L_{pile} \) is the height of column. \( E, \rho \) are steel modulus and steel density, respectively.

5.3.3 Results and Interpretation

5.3.3.1 Results plotted in 3D Diagram

The modified flexibility deflection was calculated for 3600 modal analyze. The deflection values at the middle of beam, which is the location of scour, are plotted versus corresponding stiffness and mass ratios, presented in the three dimensional diagram shown in Figure 5.5. The blue data points represent those structures that have S-shaped column sections, while the green data points represent those structures that have round HSS column. From the results, the stiffness ratio ranges from 0.01 to 750, and the mass ratio ranges from 0.03 to 9.5.
These 3600 data points in Figure 5.5 form a three-dimensional surface, which converges to the same point at $D' = 0.02818$, and becomes asymptotic to the $k^* - \alpha$ plane at $D' = 0$. More detailed discussion is provided later in Figure 5.8 and Figure 5.9.
5.3.3.2 Sample Stiffness and Mass Ratio of Typical Bridges

A rough calculation of stiffness and mass ratio of the substructure to the superstructure gives values of 5.45 and 0.12, respectively, for a typical two-span bridge supported by several columns like the Chicken Road Bridge which has 135 ft long span (one span) with 1 ft thickness and 40 ft width. The intermediate support has 40 in diameter columns that are 26 ft high. The bridge is shown in Figure 5.6.

Figure 5.6 Chicken Road Bridge [Mosavi, 2010]

Simplified bridge geometry details and calculations are provided in Table 5.2. In this rough calculation, the materials modulus and density are assumed identical for simplicity. And the bent cap is ignored during calculation, only columns are taken into account for stiffness and mass of the substructure. Reinforcement of the structure are also ignored. It has to be mentioned that Equation 5-6 calculates the stiffness of the substructure as $3E_{sub}/L_{pile}^3$ since the column is in bending behavior as a cantilever due to little restraint at the top. For the Chicken Road Bridge, due to the restraint from the bent cap and superstructure, the theoretical stiffness of one column can be expressed as $12E_{sub}/L_{pile}^3$ since the both ends can be considered as fixed. The total stiffness of the substructure is taken as the summation of the stiffness from the three columns.
Table 5.2 Calculation example of stiffness and mass ratio of Chicken Road Bridge

<table>
<thead>
<tr>
<th>Superstructure</th>
<th>Substructure</th>
</tr>
</thead>
<tbody>
<tr>
<td><img src="image" alt="Diagram" /></td>
<td><img src="image" alt="Diagram" /></td>
</tr>
</tbody>
</table>

**Chicken Road Bridge**

- **L\text{span}** = 135 ft
- **width** = 40 ft
- **s** = 10 ft
- **d\text{deck}** = 12 in
- **d\text{beam}** = 60 in
- **A\text{beam}** = 77 in\(^2\)

### Calculation and results

- \( I\text{sub} \approx \frac{\pi \cdot d_e^4}{64} \), \( A\text{sub} = \frac{\pi \cdot d_e^2}{4} \)
- \( I\text{super} \approx \frac{d\text{deck} \cdot \text{width}^3}{12} + 2A\text{beam} \cdot \left[ \left( \frac{3s}{2} \right)^2 + \left( \frac{s}{2} \right)^2 \right] \), \( A\text{super} = d\text{deck} \cdot \text{width} + 4A\text{beam} \)
- \( k^* \approx \frac{3 \cdot 12 \cdot E\text{I}_{\text{sub}} / L^3\text{pile}}{E\text{I}_{\text{super}} / L^3\text{span}} \), \( \alpha \approx \frac{3 \cdot \rho \cdot A\text{sub} \cdot L\text{pile}}{\rho \cdot A\text{super} \cdot L\text{span}} \)
- \( k^* \approx 5.45 \), \( \alpha \approx 0.12 \)

- **L\text{pile}** = 26 ft
- **d_e** = 40 in
Some bridges with substructure in the form of shear walls would have higher stiffness ratio than those bridges supported by columns. For example, Northeast Creek Bridge has strong shear walls as intermediate supports, as shown in Figure 5.7.

![Northeast Creek Bridge](image)

**Figure 5.7 Northeast Creek Bridge [Clark, 2012]**

A rough calculation shows the stiffness and mass ratio has values of 42.25 and 1.20, respectively. Similar to the calculation for Chicken Road Bridge, the material modulus and density are assumed identical. Reinforcement of the structure is also ignored. The submerged piles are the foundation structure, therefore the shear wall is considered as the only component of the substructure. However, the rigidity of a fix-ended shear wall $R_f$ is calculated instead of the bending stiffness for the substructure at the top of shear wall in the horizontal direction. The reciprocal of the total deflection is defined as the rigidity of the wall (Taly 2010), including the component from flexure behavior and shear behavior. Simplified geometry of the bridge and calculations are shown in Table 5.3.
Table 5.3 Calculation example of stiffness and mass ratio of Northeast Creek Bridge

<table>
<thead>
<tr>
<th>Superstructure</th>
<th>Substructure</th>
</tr>
</thead>
<tbody>
<tr>
<td><img src="image1" alt="Diagram" /></td>
<td><img src="image2" alt="Diagram" /></td>
</tr>
</tbody>
</table>

Northeast Creek Bridge

<table>
<thead>
<tr>
<th>Calculation and results</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>$R_f = \frac{E \cdot t}{\left(\frac{h}{d}\right)^3 + 3 \left(\frac{h}{d}\right)}$</td>
<td>[Taly 2010]</td>
</tr>
<tr>
<td>$I_{super} \approx \frac{d_{deck} \cdot width^3}{12} + 2A_{beam1} \cdot s_1^2 + 2A_{beam2} \left(s_1 + s_2\right)^2$, $A_{super} = d_{deck} \cdot width + 2A_{beam1} + 2A_{beam2}$</td>
<td></td>
</tr>
<tr>
<td>$k^* \approx \frac{R_f}{EI_{super} / L_{span}}$, $\alpha \approx \frac{\rho \cdot d \cdot h \cdot t}{\rho \cdot A_{super} \cdot L_{span}}$</td>
<td></td>
</tr>
<tr>
<td>$k^* \approx 42.25$, $\alpha \approx 1.20$</td>
<td></td>
</tr>
</tbody>
</table>
5.3.3.3 Deflection versus Stiffness Ratio

The modified flexibility deflection corresponding to small stiffness ratios tends to converge to same point. However with large stiffness ratio, the deflection tends to be very small. A better understanding of this can be obtained by rotating the view from Figure 5.5 into a 2-D diagram of the deflection versus stiffness ratio plane, as shown in Figure 5.8. In general, stiffness ratio of bridge structures would not likely exceeds 100, where the modified flexibility deflection reaches the flat plateau, as shown in Figure 5.8.

![Figure 5.8 Modified flexibility deflection plotted versus stiffness ratio](image.png)

Small values of stiffness ratio means little resistance is provided by the support in the horizontal direction. In the limit of this case the stiffness ratio is zero and the bridge structure behaves as a simply supported beam with no intermediate restraint as illustrated in Figure 5.8. And in the case of very large stiffness ratio, the intermediate support could be considered as fixed and rigid, and the flexibility deflection should be approach zero. For those two-span
bridge structures with any intermediate stiffness ratio, the substructure can be considered as a spring providing horizontal resistance.

### 5.3.3.4 Deflection versus Mass Ratio

For comparison, the data in the deflection versus mass ratio plane is presented in Figure 5.9.

![Figure 5.9 Modified flexibility deflection plotted versus mass ratio](image)

From a mathematical point of view, there exists multiple values of deflection for the same value of mass ratio, as shown in Figure 5.9. Therefore, the deflection is certainly not a monotonic function of mass ratio in the two dimensional presentation.

As demonstrated previously, the mass ratios of two typical bridge structures are approximately 0.12 and 1.20. For those bridges having columns type substructures, the mass ratio will likely be in the range of 0~1, as indicated by the red line in Figure 5.9.
5.3.4 Primary Parameter and Regression Analysis

The general trend of the deflection surface was illustrated in the previous discussion. The next step is to establish a mathematical relationship between the modified flexibility deflection and structural parameters. Theoretically, the modified flexibility deflection is determined by both stiffness ratio and mass ratio in the case of the rigid connection between the beam and pile. However, in this study, stiffness ratio is the primary parameter to be considered in the development of the mathematical relationship.

There are four reasons to simplify the relationship to consider only the stiffness ratio and modified flexibility deflection: ① the stiffness ratio and mass ratio in bridges are not independent variables. Unlike the idealized spring-mass system in Figure 5.3, changes in sectional geometry and effective height of piles change both stiffness and mass at the same time; ② the stiffness of the pile varies according to a cube height, higher than the linear variation for mass; ③ as mentioned previously in the spring-mass system in Figure 5.3, only partial mass from the pile can be considered as lumped mass on the superstructure. The mass allocation of the pile would mainly determine the mode shape of the pile; ④ the monotonic shape shown in Figure 5.8 shows the feasibility and potential for developing a numerical expression to relate stiffness ratio and modified flexibility deflection.

As described in the idealized spring-mass system, the analytical equation to solve frequency roots $\lambda$ is very complicated even for the fundamental mode. Besides, the implicit expression of the equation is an obstacle in deriving the relationship analytically. Since the numerical results from 3600 modal analyses show a monotonic curve shape between the modified flexibility deflection and stiffness ratio, the regression analysis method is selected to establish the mathematical relationship.
5.3.5 Proposed Mathematical Relationship

By substitute Equation 5-5 into Equation 5-3, the modified flexibility deflection can be expressed as:

\[
D' = 4\pi^2 \cdot \frac{P}{E} \cdot L_{\text{span}}^4 \cdot \left[ \sum_{i=1}^{n} \frac{1}{\lambda_i^4} \{\phi_i\} \{\phi_i\}^T \right] \{\text{load}\}
\]  
Equation 5-8

By expanding the load vector into the calculation, the modified flexibility deflection at the middle point of the superstructure \(D'_{\text{mid}}\), the same location as the scour location, can be expressed as following:

\[
D'_{\text{mid}} = 4\pi^2 \cdot \frac{P}{E} \cdot L_{\text{span}}^4 \cdot \left[ \sum_{i=1}^{n} \left( \frac{1}{\lambda_i^4} \phi_{i,\text{mid}} \sum_{j=1}^{q} \phi_{ij} \right) \right]
\]  
Equation 5-9

where \(n\) is the number of horizontal modes considered, in this case, \(n = 5\); \(\phi_{i,\text{mid}}\) is the value at the middle location of \(i\)th mass-normalized mode shape; \(q\) denotes the total number of locations to calculate the mode shape, ideally \(q \to \infty\), but in the case of actual field monitoring, \(q\) should equal the number of sensor location.

In order to help understand the derivation of convergence point \((D' = 0.02818)\) presented later, Equation 5-9 can be rewritten in the form of the summation of deflection from fundamental mode, \(D'_{1,\text{mid}}\) and deflection from all other modes \(D'_{2n,\text{mid}}\), as shown in following:

\[
D'_{\text{mid}} = D'_{1,\text{mid}} + D'_{2n,\text{mid}}
\]  
Equation 5-10

where

\[
D'_{1,\text{mid}} = 4\pi^2 \cdot \frac{P}{E} \cdot L_{\text{span}}^4 \cdot \frac{1}{\lambda_1^4} \phi_{1,\text{mid}} \sum_{j=1}^{q} \phi_{1j}
\]  
Equation 5-11

\[
D'_{2n,\text{mid}} = 4\pi^2 \cdot \frac{P}{E} \cdot L_{\text{span}}^4 \cdot \sum_{i=2}^{n} \left( \frac{1}{\lambda_i^4} \phi_{i,\text{mid}} \sum_{j=1}^{q} \phi_{ij} \right)
\]  
Equation 5-12
5.3.5.1 Derivation of Convergence Point

As mentioned previously, the value of the modified flexibility deflection at the mid-span tends to converge when the stiffness ratio approaches zero. The modified flexibility deflection at the convergence point can be calculated as for a simply supported beam. The fundamental natural frequency of a simply support beam is given by Humar (2005), and shown in Equation 5-13

\[ f_1 = \frac{\pi}{2} \sqrt{\frac{EI_{\text{sup}er}}{L_{\text{span}}}} \]  

Equation 5-13

The corresponding frequency root \( \lambda_1 \) can be obtained by substituting \( \omega_1 \) in Equation 5-13 with Equation 5-1, then \( \lambda_1 \) simply equals to \( \pi \). Then Equation 5-11 can be rewritten as:

\[ D'_{1,\text{mid}} = 4\pi^2 \frac{P}{E} \frac{L^4_{\text{span}}}{\pi^4} \cdot F(\phi) \]  

Equation 5-14

where

\[ F(\phi) = \phi_{1,\text{mid}} \sum_{j=1}^{q} \phi_{ij} \]  

Equation 5-15

For simply supported beams, the first mode shape is given by Humar (2005), as shown in the following:

\[ \phi(x) = \sin\left(\frac{\pi x}{L}\right) \]  

Equation 5-16

where \( x \) is the distance along the beam, and \( L \) is the length of the beam.

By substituting the mode shape in Equation 5-15 with Equation 5-16 and transforming the expression mathematically, \( F(\phi) \) can be rearranged as a problem of finding the limit shown in following:
\[ F(\phi) = \lim_{q \to \infty} \frac{\sin\left(\frac{\pi}{2}\right) \cdot \sin\left(\frac{q + 1}{2q} \cdot \pi\right) \cdot 2\cos\left(\frac{\pi}{2q}\right)}{\frac{1}{4} (1 + 2q) \cdot \sin\left(\frac{\pi}{q}\right) - \sin\left(\frac{\pi}{q} (1 + 2q)\right)} \quad \text{Equation 5-17} \]

By solving the above limit problem, \( F(\phi) \) is simply expressed as:

\[ F(\phi) = \frac{4}{\pi} \quad \text{Equation 5-18} \]

By substituting Equation 5-18 into Equation 5-14, the final expression of \( D'_{1,\text{mid}} \) is:

\[ D'_{1,\text{mid}} = \frac{16L_{\text{span}}^4}{\pi^3} \cdot \frac{\rho}{E} \quad \text{Equation 5-19} \]

For the example case \( L_{\text{span}} = 18 \text{ ft} \) due to simply supported beam; \( \rho = 7.345 \times 10^{-4} (\text{lb} \cdot \text{s}^2/\text{in}^4) \) and \( E = 29500 \text{ ksi} \) for the steel material being considered in both the superstructure and substructure, \( D'_{1,\text{mid}} = 0.02797 \).

It must be noted that this theoretical result is derived from only the fundamental mode. Based on the calculation for \( D'_{\text{mid}} \) with SAP2000 including all five modes, this convergence point has a value of modified flexibility deflection \( D'_{\text{mid}} \) at middle of the superstructure of 0.02818. \( D'_{\text{mid}} \) can also be expressed by following:

\[ D'_{\text{mid}} = \alpha \cdot \frac{16L_{\text{span}}^4}{\pi^3} \cdot \frac{\rho}{E} \quad \text{Equation 5-20} \]

where \( \alpha = 0.02818/0.02797 \), which is the adjust ratio for including first five modes.

It can be seen from Equation 5-20 that this convergence value depends on the length and material of superstructure. Therefore, for structure with different span length and materials, this convergence point no longer be the same as 0.02818, but can be calculated through Equation 5-20.
5.3.5.2 Curve Fitting and Equation

Regression analysis on previous modal analysis data is performed here to establish a numerical relationship between modified flexibility deflection and stiffness ratio. As mentioned previously, the stiffness ratio from realistic bridges is unlikely to be larger than 100, thus the regression analysis is limited to those data points within the stiffness ratio range of 0~100. As a basic requirement, the equation should satisfy the condition of convergence, that is, the deflection value needs equal to 0.02818 when the stiffness ratio approaches zero. This convergence condition can help reduce one fitting parameter.

As the first step, the potential form of the equation can be obtained by using the function finder feature in the software LABFIT, which has a built-in database of fitting functions including 208 functions with two fitting parameters $A$ and $B$. The function finder feature also lists the best 20 possible equations in the order of goodness of fit. Table 5.4 present the top 3 equations identified.

<table>
<thead>
<tr>
<th>Order</th>
<th>Equation form</th>
<th>Convergence condition</th>
</tr>
</thead>
</table>
| 1<sup>st</sup> | $Y = \frac{1}{A \cdot X + B}$ | when $X = 0$
| | | $Y = \frac{1}{B} = 0.02818$
| | | $B = 35.486$
| 2<sup>nd</sup> | $Y = A \cdot B^X$ | when $X = 0$
| | | $Y = A = 0.02818$
| 3<sup>rd</sup> | $Y = A \cdot \exp(B \cdot X)$ | when $X = 0$
| | | $Y = A = 0.02818$
Although the function finder feature can compare the goodness of equation fitting, the comparison conclusion may not be correct once the convergence condition is applied. Thus, MATLAB curve fitting tool is used here to determine which equation is most suitable for the mathematical relationship.

The results of the curve fitting for those three possible equations from Table 5.4 are presented in Figure 5.10, Figure 5.11 and Figure 5.12. The parameter A (or B) and goodness of fit are also shown within these figures.

Figure 5.10 Equation 5-21: parameter and goodness of fit
Figure 5.11 Equation 5-22: parameter and goodness of fit

Figure 5.12 Equation 5-23: parameter and goodness of fit
As expected, Equation 5-21 fits these data points best. It must be mentioned the fitting curve seems not converge to the point when deflection equals to 0.02818, that is because the default setting of curve fitting tool in MATLAB includes negative x axis.

By substituting parameter \( \beta \) for \( A \) and the substituting the convergence value of deflection 0.02818 with Equation 5-20, the proposed equation to relate the stiffness ratio \( k^* \) and modified flexibility deflection \( D' \) can be expressed as Equation 5-24:

\[
D' = \frac{1}{\beta \cdot k^* + \frac{\pi^4 \cdot E}{16L_{span}^4 \cdot \rho \cdot a}}
\]

where \( \beta = 4.632 \) for the rigid connection between the superstructure and substructure.

With Equation 5-24, the stiffness ratio \( k^* \) can be calculated inversely once the modified flexibility deflection is obtained. Since the change in pile height is the only factor that influences \( k^* \), it is straightforward to get the pile height by the definition of stiffness ratio given in Equation 5-6.

5.4 Model Validation

The next step is to examine the ability of Equation 5-24 in predicting the pile length based on the modified flexibility deflection value. Since the proposed mathematical relationship between the modified flexibility deflection and extent of scour is derived for the case of rigid connections between beam and column, there is no available experimental data to validate this relationship. Therefore, the same modal analysis is performed for beam-and-column structures similar to those used in the 3600 modal analyses, to calculate the modified flexibility deflection. The sections used for the superstructure and substructure are different from those used in the previous parametric study, as well as the pile length. The span length
is kept unchanged for simplicity, and the connection between beam and column is assumed rigid.

For each structure, modal analysis is performed to calculate the modified flexibility deflection $D'$. Based on the inverse calculation from Equation 5-24 and Equation 5-6, the length of pile can be determined. Table 5.5 shows the comparison of prediction value and actual value used in the modal analysis, together with the sections details. The combinations give three stiffness ratios, two of which correspond to the two typical bridge structure indicated previously in Table 5.2 and Table 5.3, and the smallest value can represent bridge with substructure less stiffer than superstructure. The proposed method shows reasonable accuracy in predicting pile length for the simplest bridge structure. Actually, the accuracy improves when the stiffness ratio reduce, most likely due to the reducing scatters at small stiffness ratio range in the fitting curve shown in Figure 5.10.

Table 5.5 Prediction of pile length

<table>
<thead>
<tr>
<th>Section combination</th>
<th>$k^*$</th>
<th>$D'$ from FE analysis</th>
<th>L pile (in) Prediction</th>
<th>L pile (in) Actual</th>
<th>Error</th>
</tr>
</thead>
<tbody>
<tr>
<td>Column='W6X8.5'</td>
<td>0.45</td>
<td>0.02649</td>
<td>58.5</td>
<td>60</td>
<td>2.47%</td>
</tr>
<tr>
<td>Beam='S24X106'</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Column='W8X15'</td>
<td>2.02</td>
<td>0.02165</td>
<td>57.4</td>
<td>60</td>
<td>4.32%</td>
</tr>
<tr>
<td>Beam='S20X75'</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Column='W14X61'</td>
<td>63.46</td>
<td>0.00263</td>
<td>56.9</td>
<td>60</td>
<td>5.13%</td>
</tr>
<tr>
<td>Beam='S20X75'</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

5.5 Conclusions

This chapter focused on the development of the relationship between the dynamic damage feature and the extent of scour for the case of rigid beam-column connections. At first, a modified flexibility deflection was proposed based on the theoretical study used in the
mathematical relationship. Based on the same theoretical study, the idea of stiffness and mass ratio of substructure-to-superstructure was adopted in the development. Modal analyses of idealized beam-and-column structures with various combinations were conducted, and the results of modified flexibility deflection were presented together with stiffness ratio and mass ratio to study the trend of the deflection surface. Then a regression analysis was performed against the modal analysis results. In the end, a mathematical equation was proposed to relate the modified flexibility deflection at the scour location and the stiffness ratio, based on which the pile height can be easily obtained in the inverse direction of the proposed equation. Throughout the validation with results from modal analysis, the proposed mathematical relationship shows reasonable accuracy in predicting the pile height.

The mathematical relationship considers the pile height as the only variable and is of a very simple form, these are considered as great advantages in solving engineering problems. However, this mathematical relationship was developed for the simplest two-span symmetrical bridge structure, of which the beam-column connection is assumed rigid. In realistic bridges, the connection between the superstructure and substructure cannot always be assumed to be rigid. Chapter 6 presents the influence of this connection on the dynamic behavior of bridge superstructures and modifies the proposed mathematical relationship by considering the reduction of total stiffness of the substructure due to non-rigid connections.
Chapter 6. Relationship between Flexibility-Based Deflection and Scour Extent for Non-Rigid Superstructure-to-Substructure Connections

6.1 Introduction

Chapter 5 presented the development of a relationship between flexibility-based deflection and scour extent for the case of rigid beam-column connections. This Chapter continues the development of this relationship considering the more realistic case where the bridge structure has a non-rigid superstructure--to-substructure connection. Again, the mathematical relationship developed in this research is based on a symmetrical two-span bridge structure.

In this chapter, firstly, an investigation on beam-column connections of the laboratory two-span steel structure created by Elsaid (2011) is conducted to demonstrate its influence on the dynamic behavior of superstructures by reducing the total horizontal stiffness of the substructure. Then a modification is applied to the theoretical mathematical model developed in Chapter 5 based on results of modal analysis considering the connection stiffness. A practical solution is proposed to easily apply the approach in the field to determine the extent of scour. Then the ability of this practical solution to predict pile height is examined against finite element model (FEM) analysis and experimental results.

6.2 Investigation of Connection Stiffness

In this section, the comparison between experimental results of the idealized two-span laboratory bridge created by Elsaid (2011) and his FEM is reinvestigated, leading to the issue of connection stiffness which was not been considered in his research. Then the stiffness of the beam-column connection for this particular case is analyzed to obtained an equivalent stiffness as an internal spring between the beam and column, which is applied in an improved
FEM simulation of the laboratory two-span structure. As expected, the results of natural frequencies and horizontal mode shapes from this modified FEM matches experimental results better than the original FEM simulation by Elsaid (2011).

### 6.2.1 Experimental Results vs Original FEM Analysis

Natural frequencies and mode shapes were extracted from the experimental impact testing through signal processing, and compared with the same quantities obtained from eigenvalue analysis using an FEM [Elsaid, 2011]. Figure 6.1 shows the first five horizontal mode shapes as a result of the eigen value analysis. The natural frequency of the structure with four levels of scour are shown in Table 6.1. For the 1st mode, the difference between the natural frequencies of the FEM and the experimental results is at least 10%. While for higher modes, such as the 5th mode, the difference of natural frequencies is less than 4%. Elsaid's research primarily focused on the experimental work, therefore he did not further investigate the factors that cause this difference.
The natural frequencies of the 2nd and 4th mode shape were not sensitive to change in pile height since the support piles were located at a stationary point of the mode shape. This can be verified with either experimental results or FE analysis, as shown in Table 6.1. However, the magnitude of the natural frequencies of the 2nd and 4th mode shapes from the FE analysis [Elsaid, 2011] is smaller than that from his experimental results. This is because in Elsaid's FE analysis, the structure is modeled with shell elements, which allow rotational degrees of freedom between elements. On the other hand, the idealized structure can be considered as a very stiff beam member under small impact load. But the shell element increases the flexibility of the beam, eventually resulting in smaller natural frequency magnitude.
In the current study, a new FE analysis using beam elements instead of shell elements gives more accurate predictions of natural frequencies for the 2nd and 4th modes. The results are now very close to (1% difference) experimental results, as shown in Table 6.1.

Table 6.1 Natural frequencies of laboratory bridge from experiment and finite element model

<table>
<thead>
<tr>
<th>Type</th>
<th>Scour (in)</th>
<th>Horizontally-displaced mode shapes frequencies (Hz)</th>
<th>1st</th>
<th>2nd</th>
<th>3rd</th>
<th>4th</th>
<th>5th</th>
</tr>
</thead>
<tbody>
<tr>
<td>EXP [Elsaid, 2011]</td>
<td>0</td>
<td>18.71, 20.64, 34.99, 52.07, 63.32</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>8</td>
<td>16.45, 20.88, 32.54, 51.92, 63.44</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>16</td>
<td>14.86, 20.86, 31.55, 52.67, 62.79</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>24</td>
<td>14.47, 20.96, 31.32, 52.37, 62.40</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>FEM Shell element [Elsaid, 2011]</td>
<td>0</td>
<td>21.10, 18.65, 44.37, 47.64, 65.83</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>8</td>
<td>19.92, 18.64, 38.43, 47.64, 61.20</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>16</td>
<td>18.35, 18.64, 34.14, 47.64, 59.38</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>24</td>
<td>16.58, 18.64, 31.52, 47.64, 58.45</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>New FEM Beam element</td>
<td>0</td>
<td>23.68, 20.62, 52.55, 52.84, 81.58</td>
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<td></td>
<td></td>
<td></td>
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</tr>
<tr>
<td>(Rigid connection)</td>
<td>8</td>
<td>22.85, 20.61, 47.53, 52.83, 71.61</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>16</td>
<td>21.74, 20.60, 42.29, 52.82, 67.32</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>24</td>
<td>20.35, 20.59, 38.17, 52.81, 65.20</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Although the FE model using beam elements improved the predicted natural frequencies corresponding to the 2nd and 4th horizontal mode shapes, the frequencies for 1st, 3rd, and 5th are worse compared to the experimental results, which may be attributed to the higher estimation of structural stiffness in the new model.

In fact, Elsaid (2011) did take into account of the connection between the beam and pile in his FE model. The rotation of the steel grid was allowed over the intermediate pile in his FE model. However, the connection was simulated as rigid constraints in the three
translational directions which overestimate the rigidity of the connection. The estimation of connection stiffness is presented in the following.

### 6.2.2 Analysis of Equivalent Connection stiffness

An L-angle was welded to the side of the beam girder and the top surface of the pile to provide the horizontal resistance against translation. The L-angle has approximate dimensions as shown in Figure 6.2.

![Figure 6.2 Detail of angle connection [Elsaid, 2011]](image)

Stiffness can be calculated via the moment-area theorem, according to its definition, the amount of force required to produce unit displacement. Figure 6.3 presents the idea of how to calculate the equivalent horizontal stiffness of the angle connection.
The expression of equivalent connection stiffness is given in Equation 6-1.

\[ k_{connec} = \frac{6EI_o}{2d^3 + 3dh^2} \]  \hspace{1cm} \text{Equation 6-1}

where \( I_o = \frac{ab^3}{12} \). For those parameters specified in Figure 6.2, which are measured from the L-angle connection directly, the horizontal stiffness \( k_{connec} \) has a value of 6.99 kip/in. To be noted that both columns are connected to beam girders with L-angle. The actual horizontal stiffness is likely smaller than this value since some additional flexibility exists due to the discontinuous beam being connected with bolts. In addition, the actual boundary condition of the L-angle at the welds is not totally fixed, which would also reduce the calculated stiffness.
For comparison, the expression of stiffness at the top of the pile is given in Equation 6-2,

\[ k_{\text{pile}} = \frac{n \cdot EI_y}{L^2} \]  

Equation 6-2

where \( n = 6 \) for two cantilever columns, \( I_y = 17.3 \text{ in}^4 \) for the W12x26 steel section, and \( L = 42 \text{ in} \) for the unscoured case. With these parameters, the stiffness of pile \( k_{\text{pile}} \) has a value of 41.34 kip/in.

Hence, the superstructure can be considered to be supported by springs in series for a total spring stiffness given by Equation 6-3,

\[ \frac{1}{k_{\text{sub}}} = \frac{1}{k_{\text{pile}}} + \frac{1}{k_{\text{conn}}} \]  

Equation 6-3

It is easy to conclude that the total stiffness of the substructure is reduced due to the non-rigid connection, which means the resistance from the pile against the horizontal vibration of the superstructure is also reduced due to the non-rigid connection.

### 6.2.3 Results from Improved FE Analysis

Based on the better understanding of the role of the connection, the FE modal analysis of Elsaid's idealized laboratory structure can be improved by considering the connection as an internal spring with stiffness less than 6.99 kip/in.

Table 6.2 presents the natural frequencies for various scour case (0, 8, 16, 24 in) by modeling internal springs with magnitudes (kip/in) which gives the best prediction of the experimental results. The experimental magnitudes can be different since the connections were re-welded every time the column height changed to simulate a different scour case.
Table 6.2 FE modal analysis of idealized two-span structure considering connection as spring.

<table>
<thead>
<tr>
<th>Type</th>
<th>Spring stiffness (kip/in)</th>
<th>Scour (in)</th>
<th>Horizontally-displaced mode shapes frequencies (Hz)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td>1&lt;sup&gt;st&lt;/sup&gt;</td>
</tr>
<tr>
<td>New FEM</td>
<td>4.5</td>
<td>0</td>
<td>18.50</td>
</tr>
<tr>
<td>Beam element (realistic connection)</td>
<td>2.3</td>
<td>8</td>
<td>16.69</td>
</tr>
<tr>
<td></td>
<td>2.3</td>
<td>16</td>
<td>15.07</td>
</tr>
<tr>
<td></td>
<td>2.3</td>
<td>24</td>
<td>14.78</td>
</tr>
<tr>
<td>EXP [Elsaid (2011)]</td>
<td></td>
<td>0</td>
<td>18.71</td>
</tr>
<tr>
<td></td>
<td></td>
<td>8</td>
<td>16.45</td>
</tr>
<tr>
<td></td>
<td></td>
<td>16</td>
<td>14.86</td>
</tr>
<tr>
<td></td>
<td></td>
<td>24</td>
<td>14.47</td>
</tr>
</tbody>
</table>

The predictions shown in Table 6.2 are in high agreement with the values extracted from the experiment data, especially for the 1st mode. The frequency of the 3rd and 5th modes also slightly improved. The frequency of the 2nd and 4th modes did not change significantly since the support piles are on the stationary node.

Figure 6.4 shows comparison of the 1st~5th mass normalized mode shapes from experimental results (scour = 8 in) and the three FE simulations mentioned previously. The new FE simulation with realistic connection shows a much better match with experimental results than other two FE simulations for the 1st mode. As expected, the 2nd and 4th did not exhibit much difference. For the 3rd mode, the new FE simulation has slightly improved the mode shape result, as indicated by the blue dashed line compared to the experimental black curve. However, it is hard to see any improvement for the 5th mode shape likely due to the insensitivity of higher modes to the change in stiffness of the connection.
(a) 1st mode shape

(b) 2nd mode shape
(c) 3rd mode shape

(d) 4th mode shape
6.3 Model Development Considering Connection Stiffness

In Chapter 5, a mathematical relationship between the modified flexibility deflection and the stiffness ratio was developed for the case of rigid beam-column connections. However, a truly rigid connection between the superstructure and the substructure in a typical bridge is rare. Thus, the mathematical relationship developed in Chapter 5 needs to be modified to allow for different connection stiffness levels. In order to do so, a parametric study based on FEM modal analyse on the same beam-column structure was performed. Similar to previous work in Chapter 5, this parametric study also considered different section combinations and pile length. In addition, a relative connection-to-substructure stiffness ratio $\eta$ was used to represent different connection stiffness level, as given by:
where \( k_{\text{pile}} = \frac{3EI_{\text{pile}}}{L^3} \) is the stiffness of a cantilever column. Then in each FE model, the connection can be simulated as an internal spring link element with stiffness equal to \( n \cdot k_{\text{pile}} \).

Table 6.3 presents all four parameters varied in the parametric study, which includes 20 beam sections, 10 column sections, 12 different relative stiffness ratios, and 3 pile heights. The total number of FE analyses is 7200. With these parameter combinations, the substructure-to-superstructure stiffness ratio has a range of 0.01~700.

### Table 6.3 Parameter matrix for combination of idealized beam-and-column structure

<table>
<thead>
<tr>
<th>Section type</th>
<th>Section selection</th>
</tr>
</thead>
<tbody>
<tr>
<td>Beam (W-shape)</td>
<td>W18X97; W18X86; W18X76; W16X67; W12X53; W21X93; W21X83; W8X58; W21X68; W8X48; W21X62; W8X40; W8X35; W21X48; W8X31; W8X24; W8X21; W8X18; W8X15; W6X8.5</td>
</tr>
<tr>
<td>Column (S-shape)</td>
<td>S24X121; S24X106; S20X96; S20X75; S18X70; S15X50; S12X35; S8X23; S6X12.5; S3X7.5</td>
</tr>
<tr>
<td>Relative stiffness ratio ( \eta )</td>
<td>0.2; 0.4; 0.6; 0.8; 1; 5; 10; 25; 50; 100; 500; 1000</td>
</tr>
<tr>
<td>Pile height</td>
<td>48 in; 54 in; 60 in</td>
</tr>
</tbody>
</table>

### 6.3.1 Theoretical Adjusted Mathematical Relationship

From the above parametric study, the modified flexibility deflection can be calculated based on the natural frequencies and mode shapes of the first five horizontal modes extracted from each FE analyse. The results are plotted against the stiffness ratio defined in Chapter 5, as shown in Figure 6.5. The curves correspond to the 12 relative stiffness ratios. Only the data that has stiffness ratio smaller than 150 is shown since typical bridges are unlikely to have larger substructure-to-superstructure stiffness ratios, as indicated in Chapter 5.
A regression analysis has been conducted for those 12 curves, which indicate that the form of expression given in Equation 6-5 could describe the family of curves as a function of connection stiffness. It also can be seen from these solid lines that the regression curve tends to better fit the relationship with less stiff connections.}

![Figure 6.5 Modified flexibility deflection versus stiffness ratio for various levels of connection stiffness](image_url)

\[ D' = \frac{1}{\beta \cdot k^* + \frac{1}{D_o'}} \quad \text{Equation 6-5} \]

where \( D_o' = \frac{16L^3_{\text{span}} \cdot \rho \cdot a}{\pi^3 \cdot E} \) and represents the modified flexibility deflection at the convergence point. The stiffness ratio \( k^* \) can also be rewritten as function of pile height for bridges with columns as the substructure, given by:
$k^* = \gamma \left( \frac{1}{L_{\text{pile}}} \right)$  

Equation 6-6

where $\gamma = \frac{n \cdot E \cdot I_{\text{sub}}}{E \cdot I_{\text{super}} / L_{\text{span}}}$ is constant for a given bridge structure, and $n$ varies depending on the number of columns and boundary conditions, for the idealized beam-column structure $n = 3$.

Different $\beta$ values for each Equation 6-5 are calibrated for corresponding connection relative stiffness ratios $\eta$ shown in Table 6.4.

Table 6.4 Calibrated values of $\beta$ from regression analysis for 12 levels of connection stiffness

| Connection relative stiffness ratio $\eta$ |
|---|---|---|---|---|---|---|---|---|---|---|---|
| 0.2 | 0.4 | 0.6 | 0.8 | 1 | 5 | 10 | 25 | 50 | 100 | 500 | 1000 | Rigid |
| $\beta$ | 0.698 | 0.983 | 1.220 | 1.569 | 1.762 | 3.467 | 4.170 | 4.438 | 4.524 | 4.575 | 4.621 | 4.626 | 4.632 |

By plotting the best calibrated value of $\beta$ versus relative connection stiffness ratio $\eta$, as shown in Figure 6.6, it is easy to observe that $\beta$ quickly converges to the value of rigid case when $\eta > 10$.

![Figure 6.6 Calibrated value $\beta$ versus relative connection stiffness 0~1000](image-url)
β can be calibrated as a function of η, given by Equation 6-7.

\[
\beta = 4.632 \cdot \left(1 - e^{-0.154\eta}\right)
\]

Equation 6-7

Figure 6.7 shows the fit curve for β - η relationship.

![Figure 6.7 Fit β - η relationship](image)

The flexibility deflection D can be calculated based on the natural frequencies and mode shapes extracted from the field data. Then an accurate estimation of equivalent stiffness and mass of superstructure is required, in order to normalize D into the modified flexibility deflection D'. According to Equation 6-5, the pile height is the only unknown variable, because β can be calibrated through Equation 6-7, which is still a function of pile height, and all other parameters can be estimated theoretically. Then, the unknown pile height can be predicted by solving Equation 6-5.

However, it is not likely feasible to accurately estimate the relative connection stiffness η for a bridge structure. The determination of connection equivalent stiffness, even for the laboratory two-span structure presented previously, has many
uncertainties. Therefore, an accurate estimation for the connection stiffness is very difficult to achieve. Thus, Equation 6-5 and the idea of calculating $\beta$ for a real in-service bridge structure will be difficult to apply in practice.

### 6.3.2 Practical Solution

Inspired by the typical trend for the family of curves with varying connection stiffness Figure 6.5, a practical method is proposed here to overcome the difficulty of predicting connection stiffness in most bridge structures. Based on the expression shown in Equation 6-5, only one data point is needed to determine one curve corresponding to the relative connection stiffness $\eta$, since the curve will converge to a point representing a simply support beam when stiffness ratio approaches zero.

Theoretically, this Equation 6-5 cannot be directly applied because the relative connection stiffness $\eta$ varies as scour changes the effective pile height, assuming that the connection does not suffer from degradation or damage during the monitoring period.

But based on the expression for the previous family of curves, the pile effective height can still be predicted by the idea of a "moving function" as presented in Figure 6.8. For a particular bridge structure under investigation, the relative connection stiffness ratio $\eta$ increases when the pile height increase from $L_1$ to $L_2$ due to scour, which leads the original status $P_1$ to move to $P_2$ on the curve corresponding to $\eta_2$, then move to $P_3$ on the curve corresponding to $\eta_3$. The moving function should exhibit a trend of increasing $D'$ due to the reducing of pile stiffness caused by scour.
Since the variation of $\beta$ value tends to converge quickly when $\eta > 10$, as shown in Figure 6.6 and Figure 6.7, the moving function can be derived for these two ranges of $\eta$, respectively.

When $\eta < 10$:

The moving function can be obtained by substituting $\beta$ with Equation 6-7 into Equation 6-5 and replacing $\eta$ with Equation 6-4, given by,

$$\frac{1}{D'} = M \cdot \left(1 - e^{N \cdot L'}\right) \cdot \left(\frac{1}{L'}\right) + \frac{1}{D_o}$$  \hspace{1cm} \text{Equation 6-8}

where $M = 4.632\gamma$, and $N = \frac{-0.74}{3EIl_{pile}}$. $N$ is the only unknown parameter since only the connection stiffness cannot be accurately estimated. But $N$ can be determined by substituting the original status $P_1$ into Equation 6-8, given by
When $\eta > 10$:

For a given bridge before any scour damage occurs, $\beta$ converges to the value of the rigid case and the increasing $\eta$ due to scour is not significant, and likely unnoticeable. In this case, the moving function is very close to the original curve corresponding to $\eta_1$, as shown in Figure 6.9. Therefore, the modified flexibility deflection-pile height curve can be approximated as the moving function, which can be expressed by Equation 6-5 with only one unknown variable $\beta$.

$$N = \frac{1}{L^3} \ln \left(1 - \frac{1}{\frac{D_1}{D_c} - \frac{1}{M \cdot \frac{1}{L_1}}}\right)$$

Equation 6-9

Figure 6.9 Approximation of moving function when $\eta > 10$
The expression of moving function can be determined by substituting the known point $P_1$ into Equation 6-5 and replacing stiffness ratio $k^*$ with Equation 6-6. Then the multiplication of $\beta$ and $\gamma$ can be calculated from $L_1$ and $D'_1$ of the original status, given by,

$$\beta \cdot \gamma = \frac{\frac{1}{D_1} - \frac{1}{D_o}}{\frac{1}{L_1} - \frac{1}{D_o}}$$

Equation 6-10

Then the moving function can be used to predict the future pile height after scour occurs at the pile location. Based on the experimental results from the VBDD testing in the field, the modified flexibility deflection can be calculated, then the pile height can be determined from Equation 6-11 with known $\beta \cdot \gamma$.

$$L = \left( \frac{\beta \cdot \gamma}{\frac{1}{D} - \frac{1}{D_o}} \right)^{1/3}$$

Equation 6-11

In the practical application of the VBDD technique, an assessment of the healthy structure (original status) has to be acquired under known conditions prior to a scour event, which gives the one necessary point to construct the curve. The moving function is then determined once pile length and modified flexibility deflection are obtained for the original condition of the bridge. It must be mentioned the basic assumption of this method is that the connection stiffness does not vary in the period of monitoring. Also an rough estimation of relative connection stiffness is needed in order to determine which expression should use for the moving function.
It should be noted that the calculation of the modified flexibility deflection \( D' \) required an accurate estimation of moment of inertia (transverse direction) and area for the superstructure of the bridge in order to calculate the correct modified flexibility deflection. This is important because the derivation of the convergence point is based on the modified flexibility deflection, which is independent of moment of inertia and area of the superstructure, as presented in Chapter 5. The application and validation of the practical solution is illustrated below.

6.4 Model validation

In this section, the practical solution is applied on the same idealized two span beam-column structure used in the previous parametric study. An FE model was created to calculate the modified flexibility deflection of the structure with different combinations of section and pile height. Two FE models are created for the two section combinations shown in Table 6.5. As mentioned previously, typical bridge stiffness ratios are in the range of 0~100. Therefore, two combinations of superstructure and substructure are used in the FE modal analysis. The stiffness of the two combinations shown in Table 6.5 correspond to the stiffness ratio calculated for the two typical bridge structures presented in Table 5.2 and Table 5.3. And the connection between the beam and column is simulated as a spring link element with relative stiffness ratio of \( \eta = 0.2, 10, 1000 \) to the substructure stiffness when the pile height is 48 in. Then increasing scour level is simulated by modeling the pile height as 54, and 60 in. The length of each span is 108 in. The section combinations are shown in Table 6.5.
Table 6.5 Beam-column structure parameter combinations

<table>
<thead>
<tr>
<th>Section combination</th>
<th>Relative connection stiffness $\eta$</th>
<th>Pile height (in)</th>
<th>Stiffness ratio range</th>
</tr>
</thead>
<tbody>
<tr>
<td>Column='W8X15' Beam='S20X75'</td>
<td>0.2, 10, 1000</td>
<td>48, 54, 60</td>
<td>2~4</td>
</tr>
<tr>
<td>Column='W14X61' Beam='S20X75'</td>
<td>0.2, 10, 1000</td>
<td>48, 54, 60</td>
<td>63~124</td>
</tr>
</tbody>
</table>

The structure with the pile height of 48 in is considered as the original reference status before scour. Then the modified flexibility deflection calculated based on the modal analysis results and the pile height of this original status can be used to determine the expression of the moving function. The predictions of pile height from the moving function for each structure are shown in Table 6.6.
Table 6.6 Moving function and prediction of pile height

<table>
<thead>
<tr>
<th>Section combination</th>
<th>Relative connection stiffness $\eta$</th>
<th>Modified flexibility deflection $D'$</th>
<th>L_{\text{pile (in) Actual}}</th>
<th>L_{\text{pile (in) Prediction}}</th>
<th>Error</th>
</tr>
</thead>
<tbody>
<tr>
<td>Column='W8X15'</td>
<td>$\eta = 0.2$</td>
<td>$D' = \frac{1}{1625054 \cdot \left[1 - e^{(-0.0116 \cdot L)^3}\right]} + 35.486$</td>
<td>48</td>
<td>Original status</td>
<td></td>
</tr>
<tr>
<td>Beam='S20X75'</td>
<td></td>
<td></td>
<td>0.02644 48</td>
<td>Original status</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>0.02664 54</td>
<td>62.5</td>
<td>16%</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>0.02685 60</td>
<td>71.3</td>
<td>19%</td>
</tr>
<tr>
<td>$\eta = 10$</td>
<td></td>
<td>$D' = \frac{1}{2210176 \cdot L^3} + 35.486$</td>
<td>48</td>
<td>Original status</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>0.01803 48</td>
<td>Original status</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>0.02017 54</td>
<td>53.9</td>
<td>0.2%</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>0.02193 60</td>
<td>59.8</td>
<td>0.4%</td>
</tr>
<tr>
<td>$\eta = 1000$</td>
<td></td>
<td>$D' = \frac{1}{2445067 \cdot L^3} + 35.486$</td>
<td>48</td>
<td>Original status</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>0.01736 48</td>
<td>Original status</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>0.01975 54</td>
<td>54.4</td>
<td>0.8%</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>0.02166 60</td>
<td>61.1</td>
<td>1.9%</td>
</tr>
</tbody>
</table>
From the results shown in Table 6.6, it can be seen that when the connection is relatively stiffer ($\eta > 10$), the approximation of moving function as described previously shows a very good prediction. However, if the connection is very flexible compared to the pile stiffness ($\eta = 0.2$), the moving function prediction error is greater. That is because the horizontal resistance from the substructure cannot be efficiently transferred to the superstructure due to the weak connection, which makes the
vibration of the superstructure less insensitive to the change of pile height. This insensitivity of modified flexibility deflection to the change of pile height is worse when the substructure stiffness is not significantly larger than superstructure stiffness. But the predictions for this case ($\eta = 0.2$) over predict the pile height, which is conservative since the actual residual capacity of pile would be higher than that predicted.

The $\beta - \eta$ relationship is based on a curve fit of the best calibrated value of $\beta$ in Equation 6-5 for different relative connection stiffness, that can also be a reason for the inaccuracy in the predicted of pile height.

6.5 Conclusion

This chapter completes the development of a relationship between modified flexibility-based deflection and scour extent by considering the connection stiffness between the substructure and superstructure. Since direct calculation of connection stiffness is likely difficult in practice, a practical solution based on the concept of a moving function is proposed to overcome the difficulty, which does not need an accurate estimation of connection stiffness. This practical solution requires one known point to determine the curve expression. However, an accurate estimate of equivalent stiffness and mass of the superstructure is needed to calculate the modified flexibility deflection.

So far, the developed mathematical relationship is only applicable to two-span symmetrical bridge structures whose substructure is in form of columns since the column stiffness is used in the derivation of the equations. The accuracy of the predicted pile height is reasonable based on the validation with FE analyse on an idealized beam-column structure. However, the validation also shows the limitation of
this practical solution for bridges with weak connections between the superstructure and substructure, mainly due to the insensitivity of horizontal vibration behavior to the change of pile height. In the future, more work needs to be done in order to apply this method on multiple span bridges or bridges with substructure other than columns.
Chapter 7. Conclusions and Recommendations

A continued development of an existing VBDD technique was completed in this dissertation to improve its practicality to identify the existence and location of scour on coastal bridges, and to predict the effective height of piles due to scour based on the flexibility deflection of the bridge superstructure. Conclusions of this study are presented in this chapter, as well as recommended future work.

7.1 Summary and Conclusions

Chapter 3 and 4 described an experimental program conducted at Jennette’s Pier to evaluate the accuracy and practicality of the VBDD technique based on the flexibility deflection. The experimental program was designed to investigate the dynamic behavior of the superstructure from two portions of the pier with identical geometry except the intermediate pile height. One group of piles further from shore had longer pile height than the group of piles closer to shore, this was also verified in the field. Acceleration and impact loading was recorded, then was used to calculate the flexibility deflection. The damage feature was capable of identifying relative length of effective pile height. The following presents several conclusions related to the implementation of the current VBDD on the tested coastal structure:

- Accurate extraction of natural frequencies and horizontal mode shapes requires enough accelerometers covering the complete length of two spans under investigation.
- The histogram method is adopted in the signal processing which greatly reduced the time and effort of determining the correct modes using the peak-picking method.
• Impact location influences the exhibited mode shapes. To obtain the desired horizontal mode shapes, the impact test should mainly focus on the location at the intermediate support.

• The contribution from each of the first few horizontal modes depends on the contribution factor, which is the square of the ratio of \( i^{th} \) mode natural frequency to the fundamental natural frequency. Calculation of flexibility deflection based only on the fundamental horizontal mode could be acceptable if contribution factors from others modes are relatively small.

• The selected VBDD technique is able to differentiate the relative pile height from two portions of the structures based on the flexibility deflection of the superstructure from each portion.

Chapter 5 and 6 presented a complete development of a mathematical relationship between the proposed modified flexibility deflection and the pile height. The development of the relationship firstly focused on the idealized case when the connection between the superstructure and substructure is rigid. Then, an adjustment was applied on it to adapt the model for the more realistic case of non-rigid connection. Modal analysis based on FE analyses was adopted in this research to study the dynamic behavior of numerous idealized beam-column structures. Then regression analysis was applied on the results from FE output to establish the mathematical relationship. At the end of Chapter 6, a practical solution was proposed for the purpose of engineering use in the field which was shown to be reasonably accurate through validation against FE analysis. But it had to be mentioned that the proposed mathematical relationship was developed based on a two equal span bridge, and the substructure was assumed in the
form of columns. Therefore the output cannot be directly extrapolate to more complicated bridges such as multiple span, or substructure with shear wall, to predict the effective height of piles. The following provides several conclusions regarding the development of the mathematical relationship:

- The modified flexibility deflection of the bridge superstructure was proposed as a new damage feature by normalizing the flexibility deflection with the stiffness and mass of the superstructure. This new damage feature would be used in the development of a mathematical relationship and it allowed the comparison of dynamic behavior from all kinds of bridge structures.

- The "deflection surface" obtained from numerous FE modal analyse showed a decreasing trend when the substructure-to-superstructure stiffness ratio increases. Particularly, when the stiffness ratio approached zero, the deflection surface would converge to a structural status equal to a simply supported beam. The converged modified flexibility deflection value was given both theoretically and numerically.

- The stiffness ratio was considered as the primary parameter to influence the dynamic behavior of the bridge superstructure. A typical range of stiffness ratio was given as 0-100, and the range of mass ratio was given as 0-1.

- Through regression analysis, a mathematical relationship was established to relate the modified flexibility deflection and the stiffness ratio for the rigid connection case. This rational expression satisfied the convergence feature of the "deflection surface". Also, validation of the proposed relationship against FE analysis of an idealized two-span beam-column structure showed reasonable accuracy in predicting pile height.
Non-rigid connections between the superstructure and substructure could change the modal characteristics of the superstructure significantly, including natural frequencies and the horizontal mode shapes. An improved FE model considering the connection translational stiffness matched well with the experimental result from Elsaid (2011).

The relationship between modified flexibility deflection and stiffness ratio corresponding to different relative connection stiffness could be expressed by a family of curve with similar expression and converged to the same simply supported beam case. But the theoretical relationship would be difficult to directly use in realistic problems since the connection stiffness is difficult to estimate.

A practical solution based on the concept of a "moving function" was proposed to overcome the difficulty of estimating the connection stiffness. The mathematical relationship can be determined given a reference condition of the structure. Then, the calculation of connection stiffness could be avoided. The validation of the practical solution showed best accuracy when connection stiffness is 10 times larger than the stiffness of substructure. The validation also showed when connection stiffness is relative small, the dynamic behavior of the superstructure became insensitive to the change of pile height. In this case, the error of pile height prediction increases when the substructure itself was less stiff than the superstructure. The prediction of pile height tends to be higher than the actual height, that could be considered a conservative assessment.
7.2 Recommended Future Work

While the current study led to several new conclusions regarding the scour diagnosis by applying the selected VBDD technique, additional research would be beneficial. Following are some research interests recommended for future work:

- The design of experimental work in this study was based on the reasonable assumption that two portions of structure are similar except the intermediate pile height. The ideal situation for testing will be to conduct one test for the reference condition and others immediately after severe storms. Therefore, the VBDD technique could be implemented in the future in a more realistic way.

- The current study did not have appropriate experimental results to validate the proposed mathematical relationship between the modified flexibility deflection and extent of scour. Therefore, a related experimental program should be developed to validate the current mathematical relationship, either based on laboratory small scale structure or a two symmetrical span bridge.

- The mathematical relationship was developed based on a two span bridge. More work needs to be done to applied it on multi-span bridge structures.
References


LABFIT Version 7.2.48. LAB Fit Curve Fitting Software by Wilton and Silva.


