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ABC Closure Joints: Detailing, Design and Service Life

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FLORIDA INTERNATIONAL UNIVERSITY

Miami, Florida

ABC CLOSURE JOINTS: DETAILING, DESIGN AND SERVICE LIFE

A dissertation submitted in partial fulfillment of

the requirements for the degree of

DOCTOR OF PHILOSOPHY

in

CIVIL ENGINEERING

by

Azadeh Jaber Jahromi

2019

To: Dean John L. Volakis
College of Engineering and Computing

This dissertation, written by Azadeh Jaber Jahromi, and entitled ABC Closure Joints: Detailing, Design and Service Life, having been approved in respect to style and intellectual content, is referred to you for judgment.

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Florida International University, 2019

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DEDICATION

I dedicate this dissertation to my beloved mother, and my caring sisters for their
unconditional love and endless support.

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ABSTRACT OF THE DISSERTATION

ABC CLOSURE JOINTS: DETAILING, DESIGN AND SERVICE LIFE

by

Azadeh Jaber Jahromi

Florida International University, 2019

Miami, Florida

Professor Atorod Azizinamini, Major Professor

Accelerated Bridge Construction (ABC) is a paradigm change in delivering bridges, which significantly minimizes onsite construction, enhances the safety of the travelling public and construction workers, and uses innovative materials and technologies that can increase the service life of bridges. The most widely used ABC technique utilizes prefabricated modular bridge deck elements, however, there are concerns regarding the structural performance and durability of cast-in-place joints between these elements. Due to the limitation in experimental and numerical results on the performance of closure joints, there is a lack of helpful information for detailing, design, and service life of these joints. To address these concerns, research was conducted to develop a comprehensive design specification for closure joints in ABC projects. As part of this initiative, a new detail is proposed to connect the adjacent prefabricated modular deck elements in the closure joint region. The new detail consists of a 90° hooked bar in the closure joint using normal strength concrete.

The primary goal of this research is to develop design provisions that could be adopted by governing design and construction specifications. This goal is achieved through non-destructive testing, experimental, and numerical programs focused on these joints. The

performance of the existing closure joints was also evaluated using nondestructive testing methods. Impulse response testing was identified as an effective method to evaluate joint defects, and testing was carried out on bridges in-service and laboratory specimen. The performance of the closure joints was evaluated in terms of mobility. To evaluate the structural performance of the proposed detail, specimens with different bar sizes and lap splice lengths were tested under flexural loading. The ductility ratio was used to assess the performance of different 90° hooked bar details. It is concluded that a ductility ratio between 3 to 4 can be used as a metric for joint performance. Based on the outcomes of the experimental study, design recommendations are provided for different bar sizes. A detailed finite element (FE) analysis was carried out to complement the results of experimental study, and a parametric study was performed to understand behavior of the joint.

The results show that these longitudinal connections detailed with 90° hooked bars can be a viable alternative for the design and construction of closure joints. Closure joint construction can be costly in ABC projects, which can offset the advantages of modular construction. The simplicity of the proposed new detail provides a cost-effective alternative to expensive details which use headed bars or Ultra-High Performance Concrete. The detail is not proprietary; it uses normal strength concrete and is economical.

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CHAPTER 1 INTRODUCTION

1.1 An Introduction to Accelerated Bridge Construction (ABC)

A major portion of the bridge infrastructure in the United States is approaching the end of their design life and may need retrofitting or total replacement. Design life is the period for which each component, element, or bridge is expected to be functional for its designated purpose when designed, constructed, maintained as per standards [1]. Service life, on the other hand, is the period for which the component, element, or bridge provides the desired function and remains in service [1]. By the year 2030, almost 400,000 of the 600,000 bridges in the national inventory will be exceeding their design life [2]. Many of these bridges are located over crowded roadways, and closure of these sections for an extended time is not an option. To minimize traffic disruptions, Accelerated Bridge Construction (ABC) techniques can be utilized. Minimizing the construction activities performed in the field not only decreases detour time and traffic jams but also increases safety of workers, vehicles, and the travelling public [3]. According to FHWA, “ABC is a bridge construction method that uses innovative planning, design, materials, and construction methods in a safe and cost-effective manner to reduce the onsite construction time that occurs when building new bridges or replacing and rehabilitating existing bridges.”[3]. There are three most popular ABC methods that are used in the U.S. such as, prefabricated elements, lateral slides, and Self Propelled Modular Transporter (SPMT).

1.2 Prefabricated Bridge Elements and Systems (PBES)

The prefabricated bridge element and system is the most popular among the three methods mentioned above. In this approach, large segments of bridges are prefabricated offsite, transported to final site and are connected together, using dry or wet connections. Construction of these prefabricated elements in a controlled environment can significantly improve the long term performance and durability of these elements [4]. The prefabricated elements can be deck modules, pile caps, abutments, approach slabs, and intermediate bents.

Prefabricated deck elements eliminate the extensive forming, scaffolding, reinforcing steel, and concrete placement in the conventional construction methods [5],[6]. Examples of deck elements include [3]:

- Partial-depth precast deck panels
- Full-depth precast deck panels with and without longitudinal post-tensioning
- Lightweight precast deck panels
- Fiber Reinforced Polymer (FRP) deck panels
- Steel grid (open or filled with concrete)
- Orthotropic deck
- Other prefabricated deck panels made with different materials or processes

Figure 1-1 shows an example of a prefabricated deck module, consisting of steel girder and deck cast on top and ready for shipment to the final site.

Figure 1-1 Examples of prefabricated deck element in ABC projects [3] (Photo Courtesy of Azizinamini).

As mentioned earlier, these systems require joints between elements, and some agencies have expressed concern regarding the durability and structural integrity of these joints [3]. Some of the most commonly observed problems in modular connected bridges is leaking and cracking at joints [7]. Possible causes of the leakage and cracking have been identified as shrinkage, material quality, type of details, and overlay type. Additionally, factors that impact the service life of conventionally built bridges must also be considered. This may include corrosion and deterioration due to factors such as chloride ion intrusion. To this end, joints between prefabricated deck elements have been identified to be an important detail which if not properly designed and constructed could result in reduced service life of the bridge.

1.3 Closure Joint

According to FHWA, closure joint is defined as a segment which connects two prefabricated deck elements by casting concrete in the spliced reinforcement or mechanical connector between them [3].

In a modular bridge superstructure, the shear and moment are transferred through the longitudinal closure joints which run parallel to the direction of traffic as shown in Figure 1-2. These joints are required to connect one modular unit to the adjacent units, however, the performance of these joints can be affected by environmental attacks and structural degradations [8]. Durability issues have been encountered in these longitudinal joints with the use of welded steel connectors. In addition, cracking has been observed in bridge overlays of prestressed concrete bridges. These cracks are continuous along the length of the bridge and are prone to deterioration of both superstructure and substructure elements. A leaking crack can be a potential hazard for vehicular traffic on bridges over highway underpasses [8],[9]. Water leaking through the deck causes serious issues, such as corrosion of deck reinforcement, concrete spalling and expansion due to water freezing and corroding of cross-frames if water leaked through the closure joints.

In order to control deck cracking, several researchers proposed the use of distributed reinforcement [10]. The use of closely spaced reinforcing bar provides better stress distribution when compared to widely spaced reinforcing bars [11]. In addition, it is desirable to keep the joint width small, mainly to reduce durability issues related to shrinkage of cast-in-place concrete.

Figure 1-2 Schematic view of longitudinal closure joints.

1.4 Current Details Used for Closure Joints

This section provides a description of various closure joint types used in practice. For various closure joint types, this section also provides a summary of essential information related to each closure joint type in a tabular format.

Figure 1-3 provides a summary of different closure joint types used in practice. In general, closure joint types could be divided into four major categories, as indicated in Figure 1-3. Garber et al. conducted a survey to investigate the different types of closure joint detail used by the State Department of Transportations (DOTs). Figure 1-4 shows a summary of results obtained from the survey [12].

Post-Tensioned	<p>Post-Tensioned</p>	
Mechanical	<p>Welded</p>	<p>Grouted Dowel</p>
Ultra-High Performance Concrete	<p>Self-Forming w/Straight Bar</p> <p>Straight Bar</p>	<p>Headed Bar</p> <p>Hoop Bar</p>
Conventional Concrete	<p>Hoop Bar</p> <p>Headed Bar</p>	<p>Straight Bar</p> <p>Hooked Bar</p>

Figure 1-3 Common joints detail categorized based on connection types and materials [12].

Figure 1-4 Types of joint details [12].

In the sections to follow, each major connection type is described in detail, and important information is summarized. For each connection type, a summary table is also provided, which provides essential information.

1.4.1 Post-Tensioned Connections

Figure 1-5 Post Tension detail.

One of the connection types for joining prefabricated deck units is the use of post-tensioning method. As shown in Figures 1-5 and 1-6, the longitudinal post-tensioning can use female-to-female connections. The longitudinal post-tensioning is applied through either strands or high-strength steel bars which are set through the deck panels. The splicing

of ducts is achieved using duct tape or heat shrink wrap. Initially, the deck panels are placed together, and the gap between them is filled with grout. Then the connection is achieved by applying post-tensioning, and finally, the ducts are filled with grout.

The composite behavior between the deck panels and girders is achieved by pockets cast in the slab (also known as blockouts). The shear studs from the beam extend into the pockets which are typically grouted, as shown in Figure 1-7.

The panel-to-panel joints are subjected to compression when using transverse or longitudinal post-tensioning. This compressive force can help mitigate tensile stresses, which may result from live loads. Although the post-tensioning connection is advantageous in many aspects, it generally requires better quality control and a qualified contractor. As a consequence, the post-tensioning connection may have a higher cost. Besides the cost, a lack of practical quality control during splicing and grouting may render the post-tensioning ducts to corrosion related issues [13,14]. Another concern is the prestress loss over time. These concerns have encouraged bridge designers to develop details that do not need post-tensioning. Table 1-1 lists a detailed description of post-tensioned closure joints.

Figure 1-6 Typical longitudinal post tension detail [7].

Figure 1-7 Shear pocket used to create composite action between beam and deck [15].

Table 1-1 Description of closure joints with post-tensioning

Connection Description: Post-tensioned	
Schematic view of the detail	An example used detail

Element Incorporating	Transverse closure joints using precast deck
Description	The connection was established by using grouted shear key with longitudinal post-tensioning.
Challenges	This connection is used in conjunction with a grouted shear key. It is very important to seal this connection since the PT is pulled after the grout has been placed. There have been problems with pushing the PT strand through if the maximum size and number of strand are used.
Performance History	Speed of Construction*:8 Constructability**:6 Inspection Access***:9 Durability:8 Cost:3 Future Maintenance:9
Project Example	Replacement of I-287 Viaduct over the Bronx River Parkway, NY
Design	The design was based on providing minimal tension in deck slab. It is a good option for viaducts with steel box girders. The connection was designed to transmit shear and moment forces with proper PT application.
Photo
First Year the Detail Was Used	1997
Source	[5,7,15–17]
* 0 Very Slow, 10 Very Fast, **0 Difficult, 10 Easy, *** 0 Not Visible, 10 Easily Inspected	

1.4.2 Mechanical Connections

Figure 1-8 Mechanical connection detail.

Using welded or bar coupler is the other option for connecting prefabricated deck elements. Because of the long-term performance issues due to corrosion, this connection (as shown in Figure 1-8) is not popular.

The Utah DOT [18] reported that the welded tie connection performed the worst among the other connections. The inspection reports also found that the bridges with welded tie connections experienced leakage and efflorescence between deck panels, as shown in Figure 1-9.

Figure 1-9 Typical joint leakage at deck panels (I-84 WB over Weber Canyon with welded-tie connections from 2009 inspection) [18].

One of the observed problems in this type of joint was poorly grouted shear keys between panels. An example of a welded-tie connection with poor grout is shown in Figure 1-10.

Figure 1-10 Cracks with efflorescence in parapet over the deck panel joint and Poorly bonded grout in shear pocket, I-84; US-89 to SR-167, Weber Canyon (Built 2008), Joint connection: Welded Tie connections [19].

The other option to connect precast deck panels is using grouted reinforcing dowels placed in a slotted connection. The structural performance of this connection was investigated by NCHRP project 12-65. The results of the study allowed using shorter development length, as a result of confinement provided by a steel box utilized in the connection. This connection type could be constructed without longitudinal post-tensioning. Figure 1-11 shows the use of this connection in the Live Oak Creek Bridge in Texas. A detailed description of this closure joint is listed in Table 1-2.

Figure 1-11 Transverse connection at Live Oak Creek Bridge, Texas [7].

Table 1-2 Description of closure joints with shear key with HSS (Hollow Structural Sections)

Connection Description: Grouted dowels in steel box inserts	
Schematic view of the detail	An example used detail

Element Incorporating	Transverse closure joints using precast deck
Description	The connection details developed and tested under NCHRP Project 12-65. Longitudinal post-tensioning is not required to connect the deck panels in the field. Short reinforcing bars are placed in reinforced blackouts and grouted in place.
Challenges	Selecting a non-shrink grout to fill a 2 ft wide pocket.
Performance History	Speed of Construction*:9 Constructability**:7 Inspection Access***:5 Durability:9 Cost:9 Future maintenance:9
Project Example	Live Oak Creek Bridge, TX
Design	The connection was designed to transmit shear and moment forces based on AASHTO LRFD Bridge Design Specifications (Section 9).
Photo
First Year the Detail Was Used	2008
Source	[5,7,13,15,16,20]
* 0 Very Slow, 10 Very Fast, **0 Difficult, 10 Easy, ***0 Not Visible, 10 Easily Inspected	

1.4.3 Ultra-High Performance Concrete Connections

Figure 1-12 Schematic view of current joints

Ultra-High Performance Concrete (UHPC) is a cementitious composite material with very high compressive strength, high tensile strength, and excellent durability properties. UHPC can gain compressive strength of more than 22 ksi, as well as tensile strength greater than 0.7 ksi [21], [22].

Extensive experimental tests were conducted by Yuan and Graybeal in 2014 [23] to investigate the bond behavior of reinforcing steel in UHPC. Test results have indicated that UHPC can develop reinforcing bars over a short length. The minimum embedment length of reinforcing bar in UHPC was found to be $8d_b$ (where d_b is the diameter of the bar) [23]. This allows a reduction in the width of the closure joint.

In addition, long term performance and durability of UHPC make this material an ideal alternative for connection in closure joints [24]. Sketches for different types of closure joints utilizing UHPC are shown in Figures 1-12 and Figure 1-13. A comprehensive investigation was conducted to study the performance of UHPC in deck connections by FHWA. These investigations included testing specimens under cyclic and static loading to

study the structural behavior of connections. The test results show an acceptable performance of these joints in the field [25]. Several projects were constructed using UHPC in closure joints by New York State DOT. Using UHPC in closure joints makes the width of connection smaller compared to normal strength concrete. However, the use of UHPC is associated with higher costs, therefore limiting its widespread use. It is anticipated that the development of non-proprietary UHPC mixes will reduce the cost in the near future.

Figure 1-14 shows the UHPC closure joint between prefabricated deck panels as utilized by NYSDOT on CR47 over Trout Brook [25]. A detailed description of UHPC closure joints is listed in Table 1-3.

Figure 1-13 Closure joints detail using UHPC [24].

Figure 1-14 Placement of UHPC into longitudinal connections between deck bulb-tee girders in Route 31 Bridge in Lyons, New York (Photo courtesy of NYSDOT) [25].

Figure 1-15 Field-casting of UHPC for Route 23 Bridge in Oneonta, New York (Photo courtesy of NYSDOT) [25].

Table 1-3 Description of closure joints with straight bars and UHPC

Connection Description: a straight bar with UHPC	
Schematic view of the detail	An example used detail

Element Incorporating	Longitudinal closure joints using precast deck
Description	The connection was established by using straight reinforcing bar and ultra-high strength concrete (UHPC). The connection is 8 inches wide with lapped splice bar
Challenges	Required lap splice length is considerably less than normal strength concrete. UHPC is a durable material. Cost is currently high, but by more usage development of non-proprietary UHPC mixes, it is expected that cost will be reduced.
Performance History	No leakage reported. Excellent performances
Project Example	UHPC connection developed by NYSDOT on CR47 over Trout Brook
Design	The connection was designed to transmit shear and moment forces based on AASHTO LRFD Specifications for ABC.
Photo
First Year the Detail Was Used	N/A
Source	[5,7,15,16,24,26], [27], [28]
* 0 Very Slow, 10 Very Fast, **0 Difficult, 10 Easy, ***0 Not Visible, 10 Easily Inspected	

1.4.4 Connections Using Normal Strength Concrete

Figure 1-16 Schematic view of current joints

This section provides a summary of different bar detailing using normal strength concrete. These joints include the hooped bar (full hooked or 180° hooked bar), straight bar, headed bar, and 90° hooked bar as shown in Figure 1-16.

1.4.4.1 Headed Reinforcement Detail

The alternative option to connect prefabricated deck modules is using headed rebar in closure region with normal strength concrete. The research conducted by NCHRP (project 10-71) studied the use of headed bar in closure joints. The results indicated the possibility of developing No.5 headed bar in narrow closure joints with satisfactory structural performance [29]. NCHRP 12-69 recommends the use of headed bars for ease of constructability [30]. However, there are several challenges associated with headed bars. Among the many shortcomings of headed bars, which may cause service life issues, the primary concern is the increased size of the head at the end, which will reduce concrete cover at this location. The headed bars require careful detailing to avoid interfering with adjacent headed bars during placement of two modular units (Figure 1-17) [26]. In addition,

the relative cost of headed bars is higher. Figure 1-18 shows a typical detail of closure joints using headed bars. A detailed description of closure joints with head bars and normal strength concrete are listed in Table 1-4.

Figure 1-17 Closure joints using headed bar [31] (Photo Courtesy of Azizinamini).

Figure 1-18 Closure joints detail using headed bar [32].

Table 1-4 Description of closure joints with headed bars and normal strength concrete

Connection Discription: a Headed bar with normal strength concrete	
Schematic view of the detail	An example used detail

Element Incorporating	Longitudinal closure joints using precast deck
Description	The connection was established by using the headed reinforcing bar and normal strength concrete.
Challenges	This connection need careful detailing to prevent misalignments. Limited supplier, high cost
Performance History	Speed of Construction*:7 Constructability**:10 Inspection Access***:10
Project Example	262 nd street over I-80, NE
Design	The connection was designed to transmit shear and moment forces based on AASHTO LRFD Specifications for ABC.
Photo
First Year the Detail Was Used	2008
Source	[5,7,15,16,30-32]
* 0 Very Slow, 10 Very Fast, **0 Difficult, 10 Easy, *** 0 Not Visible, 10 Easily Inspected	

1.4.4.2 Full Hooked Reinforcement Detail

The full hooked bars offer another detail of closure joint, which are economical and constructible. The full hook detail provides a solution to the clearance problem experienced with headed bars. However, there are other challenges with full hooks details which include the following:

The typical detailing of deck reinforcement at top and bottom consists of different bar sizes. However, for full hooked detailing, the bars need to be the same size. In case the bars are similar, the reinforcement may be overdesigned.

For hooked bar, ACI-318-11 sections 7.1 and 7.2, for detailing purposes, specify bend diameter [33] which result in deck thickness to be greater than 9.5 in. However, the deck thickness is typically less than 8.5 in. Consequently, the use of full hooked bars will result in increasing the deck thickness for merely detailing purposes. Figure 1-19 shows a typical detail for closure joints using full hooked bars. Figure 1-20 shows the actual construction of closure joints with full hooked bars in Brooklyn, NY. A detailed description of closure joints with full hooked bars, and normal strength concrete are listed in Table 1-5.

Figure 1-19 Closure joints detail using full hooked bar [31].

Figure 1-20 The belt parkway bridge in Brooklyn, NY [34].

Table 1-5 Description of closure joints with full hooked bar and normal strength concrete

Connection Description: Hooked bar with normal strength concrete	
Schematic view of the detail	An example used detail

Element Incorporating	Longitudinal closure joints using precast deck
Description	The connection was established by using reinforced closure pour and high early strength non-shrink concrete.
Challenges	Spacing Issue related to minimum bend requirement.
Performance History	Speed of Construction*:7 Constructability**:10 Inspection Access***:10
Project Example	Boone County IBRC Project over Squaw Creek, IA
Design	The connection was designed to transmit shear and moment forces based on AASHTO LRFD Specifications for ABC.
Photo
First Year the Detail Was Used	2006
Source	[5,7,15,16], [27]

* 0 Very Slow, 10 Very Fast, **0 Difficult, 10 Easy, ***0 Not Visible, 10 Easily Inspected

1.4.4.3 Straight Reinforcement Detail

One of the common methods of connecting prefabricated deck units is the use of straight spliced bars. However, straight spliced bars require wider closure joints for tension development of bars in closure joints. Use of wide closure joints must be avoided when possible. However, the primary reason for the use of wide closure joints is to reduce the width of the pre-topped deck in modular units; therefore, the cost during shipping. Figure 1-21 shows a typical detail for closure joints with straight spliced bars with normal strength concrete.

Regardless of the type of “Non-Shrink” concrete mixes used for longitudinal closure joints, there is a high risk of developing longitudinal and transverse cracks in the closure joints. When fresh concrete is placed in the closure joints, it will be restrained from shrinking by existing hardened concrete that is at the edges of closure joints (edges of the deck on modular beam deck units). This restraint results in the development of tensile stresses that can ultimately produce longitudinal cracks along the closure joints and transverse cracks at some intervals (about one foot on center) along the closure joints. Figure 1-22 shows the actual construction of closure joints with straight bars with normal strength concrete. A detailed description of closure joints with straight bars, and normal strength concrete are listed in Table 1-6.

Figure 1-21 Closure joints detail using straight bar [35].

Figure 1-22 Closure joints detail using straight bar [36].

Table 1-6 Description of closure joints with straight bars and normal strength concrete

Connection Description: Straight bar with normal strength concrete	
Schematic view of the detail	An example used detail

Element Incorporating	Longitudinal closure joints using precast deck
Description	This connection was used to account for the crown of the bridge deck. This connection can also be used for stage construction joints. High early concrete is commonly used. This connection was constructed in one day and opened to traffic the next day.
Challenges	Cracking Issue due to Shrinkage of wide joint.
Performance History	Speed of Construction*:7 Constructability **:7 Inspection Access **:10 Durability:9 Cost:7 Future Maintenance:10
Project Example	Route 8 Viaduct, Seymour, CT
Design	The connection was designed to transmit shear and moment forces based on AASHTO LRFD Specifications for ABC.
Photo
First Year the Detail Was Used	2001
Source	[2], [4], [6], [14], [27], [33]
* 0 Very Slow, 10 Very Fast, **0 Difficult, 10 Easy, ***0 Not Visible, 10 Easily Inspected	

1.4.4.4 Spiral Reinforcement Detail

One of the possible methods of connecting modular units is the use of spiral reinforcement. This kind of detailing is a modification to the straight bar detail. The straight bars from adjacent slabs can be wound with spiral reinforcement in closure joint (Figure 1-23). However, this detail can be labor intensive, and the installation of spirals can create congestion over a limited width of closure region. Compared to the available methods, the constructability issues of this detail makes it a less popular choice [10].

Figure 1-23 Closure joints detail using spiral reinforcements [10].

1.5 Problem Statement

As discussed, several details have been used for closure joints in practice, but none of these methods have entirely solved construction issues such as reduced concrete cover in the case of the use of headed reinforcing bars; wider closure joints which cause shrinkage cracks in the case of the use of straight reinforcing bars; construction cost in case of the use of UHPC; and corrosion and steel relaxation issues as in the use of posttensioning tendons. An ideal closure joint detail should address all issues mentioned above while satisfying the structural performance such as load-carrying capacity and ductility.

1.6 Objective and Methodology

The main objective of this research is to develop a new closure joint detail for Accelerated Bridge Construction, which can overcome the challenges in existing joints. Moreover, the designers are to be provided with complete information which allows them to select the most appropriate type of closure joint detail for a given project and which is designed for service life. This objective was achieved through the following steps:

1. Development of a comprehensive review of the current practice and compilation of all available ABC methods for constructing closure joints.
2. Evaluation of field performance of the current closure joints through non-destructive testing.
3. Development of new closure joint details using normal strength concrete with 90° hooked bars.
4. Conducting numerical and experimental studies to develop design criteria for the new proposed closure joint connection.

5. Development of design guidelines for service life for the proposed closure joint connection compared to the current connections.

Development of this dissertation has relied on extensive experimental and numerical programs. Moreover, information from various sources, including existing literature, survey of experts and stakeholders, input from ABC-UTC Advisory Board members, and other domain experts.

1.7 Organization of Dissertation

This dissertation is organized in four chapters, as follow:

Chapter 1 (herein) describes the definition of closure joints and current available options.

Chapter 2 describes and evaluates the service life performance of existing closure joint details through non-destructive testing. To this end, a field investigation was carried out on existing closure joints of two bridges that were constructed with ABC technology. Although field investigation gave insight to the performance of the closure joints, but further investigation into common joints issues was deemed necessary. Therefore, laboratory specimens were constructed which simulated different defects and results showed that Impulse Response method is a promising technique to detect damages. However, based on the NDT testing and literature review it was observed that some of the existing joints have complicated details which inhibits inspectability, and, tight tolerances for construction of these joints may cause service life issues. Therefore, it was realized that a simplified new detail is required which is economical and have longer service life.

Chapter 3 discusses development of proposed closure joint detail through experimental and numerical investigation. This chapter discusses a fundamental of load transfer mechanism through development length and lap splice lengths in addition to providing the current code provisions for development length of straight and hooked bars. Furthermore, this chapter summarizes an extensive parametric study using calibrated nonlinear finite element model developed based on the results of the previous studies in literature. Furthermore, findings from the numerical study are used to comprehend and develop the experimental program which assesses the structural performance of the proposed closure joint detail such as the mode of failure, load capacity, and ductility ratio. As a result of extensive experimental, numerical and field testing, a design recommendation was developed which will provide bridge engineers with a guideline to select most suitable closure joint for their projects.

Chapter 4 summarizes the conclusions, recommendations, and suggestions for future studies.

CHAPTER 2 SERVICE LIFE PERFORMANCE OF EXISTING CLOSURE

JOINTS

2.1 Introduction

Precast concrete and cast-in-place decks share many performance issues including cracking, corrosion of reinforcement, spalling, delamination, and concrete deterioration evidenced by scaling, wear, and abrasion. These detrimental effects may affect service life which is defined as the period for which the component, element, or bridge provides the desired function and remains in service [1]. Closure joints are the source of many bridge deck service life issues [38]. ABC bridges share these issues as leaking and cracking at closure joint locations are some of the most commonly observed performance issues [18]. Possible causes of the leakage and cracking have been identified as shrinkage, material quality, details of the joint (e.g., shape and reinforcing), and overlay type. Additionally, the sealant material applied to construction joints can break down, also causing cracking, leakage, and eventually reinforcement corrosion [38].

Regular bridge inspections are crucial to extending service life, and presently, visual inspection is the primary method used to discover deterioration and damage. However, visual methods only identify the problems after damage progresses and influences the system reliability [39]. Phares et al. discuss aspects of visual bridge inspection that need significant improvement [40]. When routine inspections are performed by different inspectors, condition ratings, element-level inspection results, inspection notes, and photographs vary significantly. The assignment of condition ratings, which is one of the most important results of a bridge inspection, also has a high degree of

variability. Additionally, when an in-depth inspection is prescribed, the inspection may not yield any findings beyond those noted during a routine inspection [40]. These types of issues have led to significant advances in developing nondestructive testing (NDT) technologies to assist in assessing bridge conditions.

2.2 Nondestructive Testing

Bridge owners and operators have embraced traditional nondestructive testing (NDT) technologies to complement visual inspections to provide a means to prioritize bridge rehabilitation, maintenance, and emergency repairs. Some of the examples of NDT technologies include mechanical wave, electromagnetic waves, penetration radar, magnetic methods, and others. Figure 2-1 shows different NDT methods available for assessing the condition of the structure. Among the listed methods, mechanical wave vibration methods have been widely used for assessment of closure joints.

Figure 2-1 Overview of NDT methods [41].

Farhandoust et al. evaluated some of the technologies (shown in Table 2-1), which have been commonly used for closure joints and provided metrics for selection of the technologies [42].

Table 2-1 Comparison and preliminary rating of NDT methods for ABC closure joints – Good=G, Fair=F, Poor=P [42]

Capability Type NDT Method	Test Speed	Surface Scanning	Internal Detection	Accuracy	Analyzing Speed	Cost	Ease of Use	Safety	Skill	Repeatability
Impact Echo Testing	F	F	G	F	G	G	G	G	G	G
Ground Penetrating Radar	G	F	G	F	G	G	G	G	G	F
Ultrasonic Testing	F	P	G	G	G	F	G	G	F	G
Phased Array Ultrasonic Testing	F	P	G	G	G	G	F	G	F	G
Infrared Thermography Testing	G	G	F	F	G	G	G	G	G	F
Impulse Response Testing	F	G	F	F	G	G	G	G	G	F

Impact Echo testing, and Ground Penetrating Radar testing have been successful in identifying and locating defects in concrete bridge decks [43]. These methods are described briefly below. Recent work with the Impulse Response test has shown improvements in accuracy and repeatability to levels that compare favorably with Impact Echo testing [44].

2.2.1 Impulse Response Testing

One of the NDT methods that can provide a better understanding of the behavior and performance of closure joints is Impulse Response (IR) testing. This method becomes a tool to assist in the evaluation of structures, including bridges. IR test help to clarify the factors that reduce the effectiveness of the closure joint, which in turn leads to developing strategies to mitigate these factors. The service life of ABC bridges can be increased with

this understanding and by developing details that could provide better service life performance [45].

IR testing has been used successfully to assist in the general evaluation of structures, and can typically identify areas associated with concrete damage, such as honeycombs, voids, and cracks.

IR testing measures the mobility at each test location. Areas with high mobility are associated with structural damage. Comparison of mobility values obtained shortly after construction with values obtained during subsequent inspections will help with identifying areas that are beginning to show evidence of damage. This, in turn, will allow proper planning and implementation of appropriate repair or rehabilitation procedures.

Figure 2-2 IR equipment and schematic of test setup [46,47].

The IR method consists of hitting the test surface with a rubber tipped hammer instrumented with a load cell to measure the force applied to the surface. A geophone placed nearby measures the surface velocity of the stress wave resulting from the hammer blow. The geophone is placed at various points along a predefined grid, with the hammer impacting the test surface near each geophone location (see Figure 2-2). The mobility is then defined as the ratio of the velocity amplitude at the test location to the force amplitude

at a given frequency. Mobility is expressed in units of velocity/force. This method is illustrated in Figure 2-3 and 2.4 and Equation 2-1.

$$Mobility_{(f)} = \left| \frac{FFT(Velocity_{(t)})}{FFT(Force_{(t)})} \right| \quad (2-1)$$

Where:

Mobility_(f) = mobility in the frequency domain (in / s / lb).

FFT = Fast Fourier transform algorithm

Velocity_(t) = recorded velocity signal (in / s)

Force_(t) = recorded force signal (lb)

Figure 2-3 IR testing procedure [48].

The IR test method provides a quick evaluation of overall structural conditions. Often, it is combined with a more detailed investigation performed with Impact Echo or Ground Penetrating Radar methods [46]. IR testing was performed for this study to generate initial profiles of the test specimens. Additional methods may be applied to further refine the data obtained and to gain insight into closure joint behavior.

Mobility of a concrete slab is a function of the material properties, concrete quality, geometry (particularly thickness), and support conditions of the element, as well as the duration and location of the impact [46]. IR results are considered to be relative, as the many factors involved make it impractical to compare absolute values. Data is often presented as contours or in 3-D figures. Figure 2-5 shows an example of a mobility spectrum obtained from an IR test of a plate-like concrete element [47].

Figure 2-4 Typical force time waveform and amplitude spectrum plots from an IR test [47].

Figure 2-5 Example of a mobility spectrum obtained from an IR test of a plate-like concrete element [47].

2.2.2 Impact Echo Testing

The Impact Echo (IE) method has been applied to determine the location and extent of cracks, delamination, voids, honeycombing, and debonding in concrete bridges. The method is based on impact-generated sound waves. The waves propagate through the concrete and are reflected by discontinuous surfaces such as cracks and voids. The method has also been applied to locate voids in grouted tendon ducts of posttensioned structures with limited success. Concrete slab thickness measurements have been made with a high degree of accuracy [43].

A stress pulse is introduced at the concrete surface with a mechanical impact. As shown in Figure 2-6, the stress or sound wave propagates through the concrete and is reflected between regions of different densities and/or elastic moduli. A second transducer monitors the reflected waves. The travel time of the wave is determined. If the wave velocity of the concrete is known, the location of the defect or interface can be determined

[49].

Figure 2-6 Schematic view of Impact Echo testing [49].

2.2.3 Ground Penetrating Radar

Ground Penetrating Radar (GPR) is a single side scanning technique that detects internal discontinuities, voids, and cracks within materials with electromagnetic waves. GPR has been successfully applied to locate reinforcing bars and other inclusions in reinforced concrete and is often combined with other NDT methods. GPR is also applicable to bridge decks and other bridge elements for detecting damage, delamination, cracks, and voids [50].

In this technique, electromagnetic waves propagate through the concrete deck through an antenna, and are reflected back to the receiver. The electromagnetic energy travels at different velocities through different materials. Internal defects are identified with moderate accuracy by analyzing and interpreting the reflected waves.

As shown in Figure 2-7, the radar antenna will detect internal anomalies at various depths in the elements by sending and receiving the electromagnetic signals. By considering the wave velocity, the system determines the characteristics of each defect based on the depth and time of the wave reflection [50].

Figure 2-7 GPR methods for defect identification [51].

IR is the most commonly used NDT method for evaluating concrete slabs, and it has been used in this study to evaluate the performance of closure joints. Developing IR signatures for a variety of closure joints and manufactured discontinuities will also provide insight for IR testing to become a common structural condition evaluation tool. This evaluation will provide insight into the behavior of closure joints.

2.3 Testing Program

IR testing was performed on bridge structures in service and on test specimens constructed in the laboratory. The laboratory test specimens were constructed with three typical closure joint configurations specified and constructed for ABC structures. Two of the closure joints were constructed with normal strength concrete, and the third with UHPC. The lab specimens incorporated areas with artificial discontinuities in both the concrete and reinforcing steel to study the resultant IR test signatures. Details of the field and laboratory testing are presented in the following sections.

2.4 Field Testing

To better comprehend service life of existing bridges, site visits were conducted to inspect a select number of bridges. Visual inspection, together with a review of available design and maintenance documents, were carried out to identify and document the service life performance of bridges. Attempts were made to identify the possible causes of any observed service life issues.

Two in-service bridges, hereafter referred to as Bridge A and Bridge B, were field tested with IR method. Both the bridges were constructed with ABC methods and included closure strips to connect deck modules. These bridges have a closure joint detail that is about 3-ft wide, with similar strength concrete as the adjacent deck sections. As discussed further in this chapter, this closure joint detail was modeled in one of the laboratory test specimens.

2.4.1 Bridge A

Bridge A is a two-lane, two-span bridge carrying traffic over an interstate highway. It was constructed in 2006. On the bridge surface, the roadway width measured from barrier inside edges is 30 ft. The skew angle of the bridge is about 12.5 degrees. Bridge includes six precast deck modules, each with a steel box beam supporting the module. The box beams are about 5 ft. in width and 4 ft. in depth and are spaced at 6 ft. Three modules were placed side by side in each span. Each module is 132 ft. long. The interior module is 9.83 ft. in width, and the exterior modules are 10.25 ft. in width. The thickness of precast concrete deck is about 6.5 in. The bridge deck also has a 2-in. concrete overlay. The width of longitudinal closure joints is 1 ft. Reinforcing steel consists of headed reinforcing steel

within the closure joint. Transverse closure joints are included at each abutment and over the center pier (see Figure 2-8).

Figure 2-8 Pictorial view of Bridge A.

Figure 2-9 Typical section of Bridge A showing initial test node locations.

The IR test node locations were laid out with transverse section lines perpendicular to the centerline. Initially, the points were laid out with the schematic as shown in Figure 2-9. After a few sections were tested, it was decided to move the points closer together, and the distance between each point was cut in half, resulting in 49 points on each section.

Finally, each line included 49 test locations. The southernmost section line was set about 2 ft. from the abutment, and three section lines were tested. Each subsequent line was located 4 ft. to the north of the preceding line. The northern section was located near the mid span of the north span. This section also consisted of three section lines, spaced at

4 ft. apart. The measured mobility values are shown graphically in Figure 2-10. The plot in Figure 2-10 represents a section about 12 ft wide, or three section lines.

Figure 2-10 3-D Mobility plot of Bridge A, near the middle of the south span.

The longitudinal closure joint was also tested. The tested section was about 40 ft. long centered over the center pier. The geophone locations included placing one point near the flange of the box beam, two points between the box beam and the closure pour, one point in the middle of the closure pour, two points between the closure joint and the next beam, and a final point near the top of the flange of that beam. The points closest to the closure joint were placed near the edge of the joint. As the deck had a concrete overlay, it was not possible to see the actual joint location. Points were located by measuring across the deck from the interior face of the barrier wall. The location of the beams and closure joints were assumed to be in accordance with the bridge plans. Figure 2-11 shows the

measured mobility values along the length of the closure joint for the section centered on the pier.

Figure 2-11 3-D mobility plot of eastern longitudinal closure joint on Bridge A.

2.4.2 Bridge B

Bridge B is located along an unpaved county road, crossing over a small creek. This bridge consists of two lanes with a roadway width of 28 ft. measured from the inside edges of each guardrail. The single-span bridge is 57 ft. long, including the abutments, with no skew (see Figure 2-12). Folded plate girders (FPGs) are used in the bridge. The bridge modules consist of a precast concrete deck cast on top of the girder. The FPGs were delivered to the site and placed on the abutments. A total of four FPGs were placed, resulting in three longitudinal closure joints. The closure joints consist of UHPC and are 8 in. wide. Reinforcing steel in the closure joint consists of loop bars extending out of the precast concrete, with No. 5 bars placed lengthwise, inside the overlapping loops, extending the length of the joint. Transverse closure joints are also located at the abutments.

Figure 2-12 Pictorial view of Bridge B.

Figure 2-13 Typical section of Bridge B showing test node locations.

The approach roads are gravel topped, and the gravel surface extends over the bridge deck. Gravel was removed from the section lines with a small garden spade and push broom. The IR test node locations were laid out with transverse section lines perpendicular to the centerline. Each line included 31 test locations, as shown in Figure 2-13. The transverse sections were spaced about 4 ft. apart. Mobility plots are presented in Figures 2-14 and 2-15.

Figure 2-14 3-D Mobility plot of Bridge B.

Figure 2-15 2-D Mobility plot of Bridge B.

2.4.3 Field Testing Results

Low mobility regions are apparent at locations where the deck is connected to the supporting steel girders and at the support abutments and piers. High mobility regions are apparent between the girders, and in the case of Bridge A, between the flanges of the box beams. Bridge B shows low mobility near and on top of the abutments. Bridge A was not

tested on top of the abutments, but lower mobility was measured where the deck connects to the center pier. The laboratory specimens also exhibited higher mobility at the ends of the decks, where there is no continuation of the deck and no abutment to provide support. For each bridge, mobility increased by a factor of about 2 to 2.5 between the girders when compared to the mobility at the girder locations.

Figure 2-10 shows slightly higher mobility at the edges of the closure joint than in the center. The edge measurements were obtained on the precast concrete, with the center measurement obtained at the center of the closure joint. A possible explanation is that the headed reinforcing steel increases the stiffness of closure joint resulting in lower mobility.

Figure 2-11 also shows higher mobility at the north end of the bridge span of Bridge A than at its south end. This is unexpected as the spans are the same length. No apparent distress in the bridge deck was observed from either the top of the deck or by looking up from underneath the bridge to the exposed areas of the deck.

2.5 Laboratory Testing

The second phase of the study included construction of test specimens at the Structures Laboratory at Florida International University (FIU). Three specimens were constructed; each specimen is 15 ft. in length with wide-flange (WF) 30x99 steel girders spaced at 6 ft. on center. Shear studs were welded to the top flange of the girders. The girders were set on top of concrete New Jersey traffic barriers and rested on elastomeric pads, as shown in Figure 2-16.

Figure 2-16 Laboratory test specimen girders.

Each of the three specimens was constructed with a different closure joint detail. These include the two details described above for Bridges A (Specimen 2) and B (Specimen 3). The closure joint details are discussed in the following sections.

2.5.1 Specimen 1

Specimen 1 was built with a 3-ft. wide closure joint with standard overlapped reinforcing bars. The concrete strength matches the concrete in the adjacent deck section. This is similar to the closure joint detail constructed for the Mass 14 projects in Massachusetts, as shown in Figures 2-17 and 2-18. The deck has two layers of reinforcing steel. The top and bottom steel layers consist of longitudinal bars of #4 at 7.5 in. on center, and transverse bars of #4 at 6 in. on center on top and #5 at 8 in. centers on bottom.

Figure 2-17 Mass 14 closure detail under construction.

Figure 2-18 Specimen 1, Mass 14 closure joint detail (note bars are not extended through joint).

2.5.2 Specimen 2

Specimen 2 was built with a 12-in. wide closure joint reinforced with headed reinforcing steel extending into the closure joint. The concrete strength matches the concrete in the adjacent deck section. This detail replicates the closure joint detail in Bridge A. The deck has two similar layers of reinforcing steel at top and bottom of the deck, with transverse steel consisting of #5 headed bars at 9 in. on centers and the longitudinal steel

is #8 bars at 12 in. on center as shown in Figures 2-19, 2-20 and 2-21.

Figure 2-19 Construction of the headed bar closure joint detail.

Figure 2-20 Construction of the closure joint detail with headed bars.

Figure 2-21 Specimen 2, headed bar detail (note that forms have not been completely removed).

2.5.3 Specimen 3

Specimen 3 was built with an 8-in. wide closure joint, with hook shaped reinforcing bars forming overlapping loops in the closure joint (see Figure 2-22 and 2-23). Longitudinal bars extend through the hooks. UHPC was used for the closure joint. This detail replicates the closure joint detail in Bridge B. The deck has 2 layers of reinforcing steel. The top layer consists of longitudinal bars of #5 at 12 in. and transverse of #5 at 8 in. The bottom layer consists of longitudinal bars of #5 at 8 in. and transverse bars of #5 at 8 in. Hooked bars in closure joint are # 5 at 8 in.

Figure 2-22 Specimen 3, hook detail (note the shear key is present on this end).

Figure 2-23 Hook detail (note lack of shear key on this end).

Construction of the laboratory specimens included simulated cracks, debonding, and missing reinforcing steel which replicates the corroded bars. The following features were included in each specimen:

- A short section with a transverse manufactured discontinuity;
- A discontinuity of the shear key between the closure joint and the adjacent deck, and;
- A short section where the reinforcing steel is discontinuous through the joint between the deck and the closure joint.

The locations of these manufactured discontinuities are shown schematically in Figure 2-24.

Figure 2-24 Schematic of manufactured discontinuities in lab test specimens.

2.5.4 Impulse Response Testing on Laboratory Specimens

IR testing was performed on three laboratory specimens. As discussed in earlier sections, each laboratory specimen was constructed with different closure joint detail. Specimen 1 simulates the Mass 14 bridge details, which included straight bars with normal

strength concrete; Specimen 2 included headed reinforcing bars and normal strength concrete, and Specimen 3 has hook reinforcing steel and UHPC in the closure strip.

2.5.4.1 Specimen 1

Specimen 1 represents the closure joint detail that was constructed for the ABC bridges built as a part of the Mass 14 project. The test specimen included longitudinal closure joints that are 36 in. wide, and are located between previously cast concrete deck modules supported on the wide flange beams. Reinforcing steel consists of straight bars within the closure joint.

The IR test locations were laid out with transverse section lines perpendicular to the centerline. The section lines were located about 1.5 ft apart. Each section line was laid out with one IR point located over each beam (Locations 1 and 5), one at each side of the closure joint (Locations 2 and 4), and one at the centerline of the closure joint (Location 3).

The measured mobility values are shown graphically in a 3-D representation in Figure 2-25. Figure 2-26 shows the mobility ratio, a normalized 2-D plot of the measured values for the section shown longitudinally. The test values were normalized to the average of the mobility values measured at the steel beam, in accordance with ASTM C 1740 [47].

Figure 2-25 Mobility plot of Specimen 1 with Mass 14 detail.

Figure 2-26 Normalized mobility plot for Specimen 1.

As mentioned previously, variations and differences in the data are more important than the absolute values. Several items can be noted in the initial review of the data

obtained in this first round of testing. The two lines designated as Locations 1 and 5 were obtained directly above the steel beam. The other three lines include the centerline (red, Locations 3), and the closure joints (orange and yellow, Locations 2 and 4).

The edge effects can be observed at each end. This is shown by the significant increase of the mobility values at each end of the plot at rows 1 and 11. The increase in mobility is easily attributed to the edge effect or boundary condition at the slab edge. The rise in row 8 along the centerline shows the manufactured discontinuity, representing a full-depth crack in the concrete. The high point is located between the two discontinuities, and the mobility is expected to be higher. The concrete is discontinuous by casting the concrete against visqueen.

In the vicinity of row 10, the closure joint detail was changed from a keyway to a straight side. Based on this initial data, there does not seem to be a significant change in mobility related to the type of joint incorporated into the closure joint. This can be attributed to the presence of the reinforcing steel extending into the closure joint and some concrete bonding at the pour interface. The small rise in the mobility seen in the closure joint at row 5 in Locations 2 corresponds to the approximate location where the reinforcing steel was not carried through the joint. While higher mobility was expected, the keyway in this location may limit the mobility, moderating the effect of the lack of reinforcement at this location.

2.5.4.2 Specimen 2

Specimen 2 represents the same detail as Bridge A, previously tested and reported in Section 2.4.1. The test specimen included longitudinal closure joints 12 in. wide, located

between previously cast concrete deck elements supported on the wide flange beams. Reinforcing steel consists of headed reinforcing steel within the closure joint.

The IR test locations were laid out with transverse section lines perpendicular to the centerline. The section lines were located about 1.5 ft apart. As used in Specimen 1, each section line was laid out with one IR point located over each beam (Locations 1 and 7), one at each side of the closure joint (Location 3 and 5), and one at the centerline of the closure joint (Location 4). Additional points were located between the beam and closure joints (Locations 2 and 6).

The measured mobility values are shown graphically in a 3-D representation in Figure 2-27. Figure 2-28 shows a 2-D plot of the mobility ratio along the length of the bridge. The test values were normalized to the average of the mobility values measured at the steel beam.

Figure 2-27 Mobility plot of Specimen 2.

Figure 2-28 Normalized mobility plot for Specimen 2.

Several items are observed in the review of the data obtained in this first round of testing. The data shown in Locations 1 and 7 are the values obtained above the steel beam. The other three lines include the centerline (yellow, Locations 4), and the closure joints (red and purple, Series 3 and 5). The two lines (green and orange, Locations 6 and 2) between the beams and the closure joint lines represent the lines between the beams and the closure joint.

The edge effects are obvious at each end. This is shown by the increase of the mobility values at each end of the bridge section at rows 1 and 11. The increase in mobility at row 9 for the centerline (Locations 4) corresponds to the manufactured discontinuity location. The increase is not as pronounced as in Specimen 1, likely due to the narrower width of the closure joint and shorter extent of the manufactured discontinuity.

In the vicinity of row 10, the closure joint detail was changed from a keyway to a straight side. As in Specimen 1, there does not seem to be a significant change in mobility related to the type of joint incorporated into the closure pour. The variations in mobility at rows 5, 6, and 7 for Locations 3, 4, and 5 correspond to the approximate locations where the reinforcing steel was not continued into the closure joint. While higher mobility was expected, the presence of the keyway and the narrow width in this location may provide stiffness, moderating the effect of the lack of reinforcement at this location.

2.5.4.3 Specimen 3

Specimen 3 represents the same detail as Bridge B, previously tested and reported in Section 2.4.2. The test specimen included 8 in. wide longitudinal closure joints located between previously cast concrete deck elements supported on the wide flange beams. Reinforcing steel consists of hooked bars with longitudinal bars threaded within the hooks, inside the closure joint.

The IR test locations were laid out with transverse section lines perpendicular to the centerline. Again, the section lines were located 1.5 ft apart. As with Specimen 2, each section line was laid out with one IR point located over each beam (Locations 1 and 7), one at each side of the closure joint (Location 3 and 5), and one at the centerline of the closure joint (Location 4). Additional points were located between the beam and closure joints (Locations 2 and 6).

The measured mobility values are shown graphically in a 3-D representation in Figure 2-29. Figure 2-30 shows a 2-D plot of the mobility ratio along the length of the bridge. The test values were normalized to the average of the mobility values measured at

the steel beam.

Figure 2-29 Mobility plot of Specimen 3.

Figure 2-30 Normalized mobility plot for Specimen 3.

The data obtained in this round of testing shows several similarities to the other two

specimens. The data shown in Locations 1 and 7 are the values obtained above the steel beam. The other three lines include the centerline (yellow, Locations 4), and the closure joints (red and purple, Locations 3 and 5). The two lines (green and orange, Locations 2 and 6) between the beams and the closure joint lines represent the lines between the beams and the closure joint.

The edge effects are obvious at each end, which was also seen with Specimens 1 and 2. The mobility values increase at each end of the bridge. The rise in mobility at row 9 at the centerline (Locations 4) corresponds to the manufactured discontinuity location. The variation in mobility is similar to that shown in Specimen 1 and more pronounced than in Specimen 2. This may be due to the higher strength and stiffness of the UHPC in the closure joint. This may magnify the effect of the discontinuity with the interruption of the stiffer UHPC. The high mobility ratio is located between the two manufactured joints.

In the vicinity of row 10, the closure pour detail was changed from a keyway to a straight side detail. Again, the configuration seems to have little effect on the mobility. More study is needed to ascertain the phenomena involved here, as one expectation was for the straight-sided joint to have higher mobility than the keyway joint. The scatter in mobility values is greater than was seen in the other two specimens, and may be related to the relatively high concentration of steel in the closure joint combined with the higher strength and stiffness of the UHPC concrete.

Overall, the range in mobility ratio is somewhat lower for Specimen 3 as the mobility ratio values range from about 1.5 to 2.0. The range in ratio for Specimen 1 is from about 1.5 to 2.5 and for Specimen 2, about 1.6 to 2.3. This is excluding the values measured at the manufactured defects locations.

2.6 Discussion on Results

The laboratory specimens all showed a distinct increase in mobility in the general vicinity of the manufactured transverse discontinuities. An increase in mobility was also observed at the location of the discontinuous reinforcing steel. When areas of increased mobility are noted, further study may be desired and could consist of invasive testing (such as coring) or NDT with alternate methods. Although it is concluded that the results of the field and laboratory testing do not correlate since their stiffness, geometry, and material properties differ, the global behavior of closure joints exhibit increased mobility, which was observed in both modes of testing.

2.7 Conclusions

Mobility is indicative of the flexibility of the structural element at the tested location [52]. The relative mobility values obtained during this study reflect the anticipated behavior of the bridges and test specimens. The conclusions of this study are presented below.

- Mobility was lower at locations where the deck was supported by the girders, and about 2 to 2.5 times higher at locations between the girders.
- Mobility was lower at the connections to the abutments and supporting piers.
- The closure joint at Bridge A had slightly lower mobility at the center of the joint than at the edges, which is likely due to the location of the headed reinforcing steel within the joint.
- Discontinuities in the reinforcing steel and concrete in the laboratory specimens had higher mobility than locations without the discontinuities.

- IR signatures were developed for the two bridges, and provide a baseline for future testing and inspections.
- IR testing is a promising tool in evaluating the integrity of bridge decks and closure joints.

2.8 Future Studies

The test results presented above represent results from the first phase of IR testing on the three laboratory specimens. Evaluation of these results will need to be supplemented. Additional IR tests with finer grids can be performed to better fine-tune the process and to look at some of the data variations seen in this initial testing. However, it can be seen from the initial data obtained both from the field and laboratory specimens, that IR testing can be a tool to help in identifying and locating potential areas of concern in bridge decks, as the manufactured discontinuity was observed in all three specimens.

As expected, the field results of the IR testing show that mobility values increase with distance from supporting girders and bents. Continued IR testing with the laboratory specimens may prove useful in identifying issues such as cracking, debonding, and loss of reinforcing steel due to corrosion. These kinds of additional studies may lead to the development of a guideline incorporating the use of IR testing to assess the structural integrity of closure joints immediately after completion of construction. This could provide bridge owners with a tool for inspecting the quality of closure regions, where visual inspection is not effective. It is believed that the assessment of many details used in ABC projects, such as closure joints, is more effective using NDT, such as the IR test.

CHAPTER 3 DEVELOPMENT OF NEW CLOSURE JOINT DETAIL

3.1 Introduction

As discussed in Chapter 1, various methods to connect one modular unit to the adjacent units have been used in practice. One of the possible methods of connecting separate units is the use of straight lap bars in closure joints. However, straight lap bars require wider closure joints to develop sufficient development length based on the AASHTO-LRFD design specifications [53]. The other issue with straight lap bars in wide closure joint is the shrinkage cracking. Alternatively, headed bars are used which are currently in practice for construction of modular bridge systems. NCHRP 12-69 recommends using of headed bars for easier construction [30]. Among the many shortcomings of headed bars, which may cause service life issues, the primary concern is the increased size of the head at the end, which can cause issues of reducing concrete cover. The headed bars require careful detailing to avoid interfering with adjacent headed bars during placement of two modular units [31]. In addition, the cost of headed bars is relatively higher.

Another common option to reduce the closure joint width is the use of Ultra-High Performance Concrete (UHPC) between the modular deck units. This material offers high compressive strength, high tensile strength compared to normal strength concrete. In addition, the joints have low permeability, which improves the service life of the closure joint. The higher tensile strength of this material makes it possible to provide adequate strength over short lap splice length [54]. However, the higher cost of this material compared to other available concrete mixtures limits its widespread use.

Lastly, the 180° hooked bars provide detailing of closure joint, which are economical and constructible. The 180° hooked detail provides a solution to the concrete cover problem experienced with headed bars. However, this detail limits the designers to use the same reinforcement size for the top and bottom layers, which in some cases, is not the situation for bridge decks [31]. Moreover, meeting the bent bar requirements as outlined in AASHTO-LRFD section 5.10.2 (Section 7.1 and Section 7.2 of the ACI318-08) [53],[55] causes the deck thickness to be greater than 9.5 in., which is a function of girder spacing which is typically around 8.5 in.

To overcome the shortcomings of the existing details, a new connection detail is proposed in this research by using 90° hooked bars which overcome the concrete cover issue associated with head bars, deck thickness issue associated with 180° hooked bars, and the wide closure joint in case of the use of straight bars. The new detail should be designed for adequate structural performance and service life for ABC applications.

3.2 Description of the New Closure Joint Detail

In the proposed closure joint, normal strength concrete along with 90° hooked bar is used to connect the pre-topped deck elements in the ABC application. As a part of the research performed by the University of Tennessee, Knoxville, a survey was sent to various bridge professionals to determine concerns with current bar details. The primary concerns were the overall width of the closure joint and constructability of each of the details [31],[56]. In order to reduce the closure joint width and to achieve better constructability, hooked bars are commonly used. The objective of the proposed new detail in this study is to investigate the performance of 90° hooked bars in closure joint. The hooked bars may

be obtained from any local steel fabricator which greatly reduces the time and cost of fabrication and shipment to the work site [57]. AASHTO specifies the face of closure joint to be roughened or have shear keys to ensure a proper bond and shear strength between segments [58],[59]. However, the worst-case scenario is when no surface preparation is conducted on adjacent segments, which leads to a weaker shear interface due to face smoothness. Based on this assumption, details of the proposed closure joints were designed using a smooth interface, as shown in Figure 3-1.

Figure 3-1 Details of hooked bar in the proposed connection (elevation view and section).

The main advantage of a narrower closure joint is the reduction in shrinkage cracking. The hooked bar can be developed within the deck thickness over a shorter length. Also, the hooked bars have a comparatively lower fabrication cost, and their ease of construction saves time, which agrees with the ABC goals.

Although the hooked bars have been used extensively in different applications, their use in closure joints is limited due to lack of experimental and numerical data. The objective of this chapter is focused on studying the hooked detail in the closure joint region through experimental and numerical studies. To understand the mechanism of development of 90° degree hooks, a detailed experimental program was conducted by incorporating different parameters such as bar size, lap splice length, the lateral distance between reinforcement, and transverse reinforcement. The experimental research incorporating these parameters will fill the gaps in the existing studies and will provide additional details that will help to understand the mechanism of failure in closure joints. This mechanism, along with numerical study, will be used for revising existing design provisions and possible incorporation of 90° hooked details in construction applications.

The following sections provide the fundamental of bond mechanism along with the results of an experimental and numerical studies which were conducted to formulate a design procedure for calculating tension development length and tension lap splice length when the hooked bar is used in closure joints in ABC projects. The bond mechanism is provided for straight and hooked bars in normal strength concrete. Also, a critical review of existing specifications for development length is provided.

3.3 The Mechanics of Bond

Reinforced concrete is an extensively used composite material in bridge engineering. Composite materials are described as a combination of two or more materials to form a new material with different mechanical properties to those of the individual components. Reinforced concrete as a composite material is composed of concrete and reinforcing steel. Plain concrete has a high compressive strength but is weak and brittle in tension. In reinforced concrete structures, concrete mainly resists compressive stresses while steel reinforcing bars resist against tensile stresses.

Generally, to consider the reinforced concrete as a composite material, both concrete and reinforcing steel must have enough interaction so that they can carry the load together. This interaction between reinforcing steel and surrounding concrete, which transfers tensile stress from the concrete into the bar, is defined as a bond mechanism. Figure 1-1 illustrates simple concept of bond stress for a straight bar embedded in a block of concrete. The term ‘developed’ is used for steel bar when the bond stress is sufficient to resist the maximum tensile strength of the bar, as shown in Figure 3-2. The steel bar development length (L_d) is defined as the anchorage length required for embedment [60].

Figure 3-2 Concept of bond stress [60].

The transfer of tensile forces between deformed reinforcing bar and surrounding concrete is accomplished through three mechanisms [61]:

- Chemical adhesion between the bar and the concrete;
- Frictional forces originating from the roughness of the interface,
- Mechanical anchorage or bearing of the ribs against the concrete surface.

The main component to transfer the forces is bearing component of the bar deformation (ribs) from mechanical anchorage, as shown in Figure 3-3.

Figure 3-3 Bond force transfer mechanism [61].

Bearing forces in the concrete, at the bar deformations act in a direction approximately normal to the face of the rib and an angle, θ_{bond} , with respect to the bar longitudinal axis (see Figure 3-4 part i).

These bearing forces can be divided into two parts, parallel and perpendicular components, as shown in Figure 3-4 (part ii). The parallel component provides the bond which is required to resist tensile forces in the bar. On the other hand, the perpendicular component acts outward on the surrounding concrete as a radial splitting force (Figure 3-4 part iii). This force is resisted by the tensile capacity of the concrete, and if, this force exceeds the tensile capacity of concrete, then splitting failure will control.

Figure 3-4 Bond and splitting components of rib bearing stresses [60].

Two different types of failure modes including pull-out failure due to concrete crushing at the faces of the reinforcing ribs and splitting failure due to the radial tensile stresses may happen in reinforced concrete which is defined as a bond failure (see Figure 3-5). The pull-out failure occurs when bond stresses exceed the shear capacity of the concrete between bar ribs. In this case, shear cracks propagate along the length of bar and concrete will crush in front of the bar ribs. On the other hand, splitting failure happens when the space between the bars or/and the cover are small compared with the bar diameter.

Both types of bond failures can be prevented if sufficient embedment length, bar spacing, and cover are provided. In case these conditions are satisfied, the failure may occur due to yielding of bar, in which case it will be considered as a flexure failure.

Figure 3-5 Bond failure types (a) Splitting failure, (b) Shear crack and/or concrete crushing due to pullout [61].

The simplest way to provide anchorage is extending straight bars beyond the point of maximum stress demand. However, extending straight bars is not always possible option due to element size. The alternative option is use of end anchorages such as heads or hooks at the end of reinforcing bar to ensure proper transferring the forces from the steel to the surrounding concrete.

The mechanism for bond failure for hooked bars differs from the straight bars. The difference in behavior is attributed to the presence of hook. The slip between the straight bar and the surrounding concrete initiates the anchorage capacity of the hook. The mechanism of anchorage for a 90° hooked bar is shown in Figure 3-6. The slip in the straight portion of bars causes loss of bond around the outer radius while the inner radius is subjected to compressive stresses. If these stresses exceed the compressive strength of concrete, crushing can occur at the bearing of inner radius of the hook [62].

The modes of failure of 90° and 180° hook differ significantly. When subjected to tension, a 90° hook tend to straighten causing bearing against the concrete on the tail end of the hook. When the tail cover is sufficiently small, the hook tail can “kickout” the

concrete cover, causing the concrete to spall. However, in the closure joint, the adjacent concrete provides large confinement to the hook, and the spalling of concrete may not occur. The failure mechanism for 180° hooked bar is through movement of the hook as a whole, which leads to crushing of concrete on the inside edges of the hook [62–65].

Figure 3-6 Behavior of 90° hooked bar subjected to tensile force [64].

The stress transfer mechanism of a hooked rebar is presented in Figure 3-7. A hooked bar typically consists of a straight portions and a bend diameter. Typically, at full development, the portion of the hook between the straight and at the onset of bend is crushed. The load applied on the hook tend to straighten the bend; therefore, it is important that the hook extension is confined at the bend or else the extension during straightening out may lead to spalling behind the hook. On the contrary, a 180° bend pulls forward without slipping at the bend. The common failure in hooked bars is spalling of concrete cover, as shown in Figure 3-8.

Figure 3-7 Stress transfer in a hooked bar [60].

Figure 3-8 Side spall failure of a hooked bar [60].

Minor and Jirsa compared the slip behavior of hooks and straight bars and concluded that the 180° hooks undergo more slip than 90° hooks, but both these hooks experience more slip than a straight bar during load application [65]. Additional study by Marques showed that the anchorage capacity of the hook is not significantly improved by the compressive pressure within the plane of the bend [63]. The effect of lateral pressure

applied perpendicular to the plane of the bend of U-bent bars was studied by Thro and concluded that the increase in this pressure improved the anchorage strength [66].

Mattock carried out a study on the U-bent bars having an applied lateral pressure. The study found that for bars with minimum allowable bend diameter ($6d_b$), a larger lateral pressure increased the anchorage capacity. Based on the results, Mattock formulated an expression which was proportional to $(f_n/f_{ct})^{0.7}$, where f_n is the applied lateral pressure and f_{ct} is the tensile strength of the concrete [67].

3.4 Code Provisions for Development Length

Design equations in the code provisions suggest the required bond stress in terms of the minimum development length so that a steel bar is fully yielded. In the following sections calculating the minimum development lengths of the straight and hooked bars are explained based on the ACI 318 code as well as the AASHTO Bridge Design Specifications.

3.4.1 ACI 318-14

The ACI 318-14 Building Code Requirements for Structural Concrete [68] provides provisions for calculating the minimum tension development length of straight deformed steel reinforcement. When the available space is not sufficient for extending the straight bars to be fully developed, using hooks are the most common and economical method to anchorage steel reinforcing bar. The following sections discuss the minimum tension development length equations for both straight and hooked bar.

3.4.1.1 Straight bar

The ACI 318-14 [68] recommends two methods to calculate the minimum tension development length for straight bars. Table 25.4.2.2 in ACI 318-14 provides the equations under certain conditions including given bar size, spacing, cover, etc. The minimum requirements of transverse reinforcement along the development length in ACI is originated from a study conducted by Azizinamini et al. [69]. ACI suggests that the shear transverse reinforcement and this minimum requirement can be considered simultaneously. Section 25.4.2.3 suggests general equation, which is more accurate to calculate development length as below:

$$l_d = \frac{3}{40\lambda} \frac{f_y}{\sqrt{f'_c}} \frac{\psi_t \psi_e \psi_s}{\left(\frac{c_b + K_{tr}}{d_b}\right)} d_b \quad (3-1)$$

$$\frac{c + K_{tr}}{d_b} \leq 2.5 \quad (3-2)$$

$$K_{tr} = \frac{40A_{tr}}{sn} \quad (3-3)$$

Where:

l_d = development length of bar (same units as d_b , typically inches)

d_b = nominal bar diameter (inches)

f_y = yield stress of reinforcing steel being developed (psi)

f'_c = cylinder compressive stress of concrete (psi, limited to 10,000 psi maximum in section 25.4.1.4)

Ψ_t = reinforcement location factor (1.3 if 12" of concrete cast below bar)

Ψ_e = coating factor (1.5 for epoxy-coated bars with minimum clear dimension $\leq$ 3db, 1.2 for all other epoxy-coated bars)

Ψ_s = reinforcement size factor (0.8 for # 6 bars and smaller)

λ = lightweight aggregate factor (0.75 when lightweight concrete is used)

c = minimum of half the center to center bar spacing or the cover dimension measured from the center line of the bar to the surface of the concrete

K_{tr} = transverse reinforcement index

s = maximum spacing of transverse reinforcement within L_d , center-to-center (in)

n = number of bars or wires being developed along the plane of splitting

A_{tr} = total area of transverse reinforcement within the spacing, s , that crosses the plane of splitting through the reinforcement being developed (in²)

The minimum development length for straight bars is restricted to 12 inches. The mentioned equations are based on experimental studies conducted by Orangun, Jirsa, and Breen [70,71]. They developed an equation based on nonlinear regression analysis of test results of beams specimens with lap splice. They considered the effect of various parameters on the strength of lap length. These parameters are lap length, cover, spacing, bar diameter, concrete strength, and transverse reinforcement.

3.4.1.2 Standard hooks

The term of standard hooks is defined in the ACI 318 Building Code [68] as a hooked rebar with certain radius of bend and tail extension which is applicable for both 90° and 180° bars (see Figure 3-9).

Figure 3-9 Standard hooks details for development length [33]

The ACI 318-14 recommended an equation for calculating the development length of deformed hooked bar in tension (l_{dh}) as determined in equation 3-4 [68]. This equation shows that various parameters such as the yield strength of the bar, the square root of concrete compressive strength, and diameter of the bar can change the development length of hooked bar. The calculated development length (l_{dh}) with corresponding modification factors, should be larger than $8d_b$ and 6 in.

$$l_{dh} = \frac{f_y \psi_e \psi_c \psi_r}{50 \lambda \sqrt{f'_c}} d_b \quad (3-4)$$

Where:

l_{dh} = development length of hooked bar (same units as d_b , typically inches)

d_b = nominal bar diameter (inches)

f_y = yield stress of reinforcing steel being developed (psi)

f'_c = cylinder compressive stress of concrete (psi, limited to 10,000 psi maximum)

Ψ_e = coating factor (1.2 for epoxy-coated bars)

Ψ_c = concrete cover factor (0.7 for No. 11 bar and smaller hooks with side cover $\geq 2\text{-}1/2$ in.)

Ψ_r = confining reinforcement factor

λ = lightweight concrete factor (0.75 when lightweight concrete is used)

The mentioned equation is based on test results of 38 typical beam-column joints conducted by Marques and Jirsa (1975) and Pinc et al. (1977) [63,72]. They investigated the effect of different confinement, axial load and side cover on the anchorage strength of standard 90° and 180° hooked bar.

3.4.2 AASHTO LRFD Bridge Design Specifications (7nd Ed., 2014)

The AASHTO LRFD Bridge Design Specifications [53] suggested equations for calculating tension development length of straight and hooked bar.

3.4.2.1 Straight bar

There are three different equations for tension development of straight bar in the AASHTO LRFD. These equations are applicable to different bar sizes as illustrated below. Note that these equations are valid for No. 11 bars and smaller in specified concrete compressive strength (f'_c) up to 15 ksi. It is important to note that for slabs, including bridge decks, no minimum requirement is suggested.

$$l_d = \frac{1.25 A_b f_y}{\sqrt{f'_c}} \geq 0.4 d_b f_y \quad \text{For No.11 bar and smaller} \quad (3-5)$$

$$l_d = \frac{2.7 f_y}{\sqrt{f'_c}} \quad \text{For No.14 bar} \quad (3-6)$$

$$l_d = \frac{3.5 f_y}{\sqrt{f'_c}} \quad \text{For No.18 bar} \quad (3-7)$$

Where:

l_d = development length (inches)

A_b = area of bar (in²)

f_y = yield stress of bar (ksi)

f'_c = compressive strength of concrete (ksi)

d_b = diameter of bar (inches)

For considering different conditions, AASHTO provides modification factors as described below [53]:

- Top horizontal cast concrete (12" of concrete below bar): 1.4
- Lightweight concrete used: 1.3
- Sand lightweight concrete used: 1.2
- Epoxy-coated bars with clear dimension $\leq 3d_b$: 1.5
- All other epoxy-coated bars: 1.2

3.4.2.2 Standard hooks

AASHTO LRFD uses the term “standard hooks” for the hook bars with a certain radius of bend and extension tails. The tension development length of standard hooked bars (l_{dh}) with the maximum yield strength (f_y) of 60 ksi, including both 90° and 180° hooks,

can be calculated by equation (3-8). Figure 3-9 shows the tension development length of standard hooked bars in detail.

$$l_{dh} = \frac{38}{\sqrt{f'_c}} d_b \geq 8d_b \text{ and } 6 \text{ in.} \quad (3-8)$$

Where:

l_{dh} = development length (inches)

f'_c = compressive strength of concrete (ksi)

d_b = diameter of bar (inches)

For considering different conditions, AASHTO provides modification factors as illustrated below [53]:

- f_y more than 60.0 ksi : $f_y/60$
- Side cover $\geq 2.5''$ and cover over 90° hook extension $\geq 2''$: 0.7
- confining reinforcement used: 0.8
- lightweight concrete used: 1.3
- Epoxy-coated bars: 1.2

The modification factors, mentioned above, do not provide any factor for top-cast bars because the mechanism for anchorage of hooks is contributed mainly through direct bearing and not by bond. Compared to straight bars, the hooked bars develop over shorter lengths, particularly when low strength concrete is used.

3.5 Lap Splice Mechanism

Due to limitations on available length of reinforcing bars, field splices are often inevitable in construction. Although 60-ft length reinforcing steel is stocked by steel

fabricators, it is often convenient to transport and work in the field with bars of shorter lengths, thus necessitating the use of rather frequent splices.

From the standpoint of constructability and economy, lap splices are easiest. However, there are some disadvantages related to the use of the lapped splices such as the complicated nature of the resulting stress transfer and the local cracks that frequently occur in the vicinity of the bar ends. Bond stresses play an important part in transferring the forces from one bar to another. Thus the required splice lengths are closely related to development lengths [73].

For lapped splice, the force from one bar transfers to the adjacent bar through the concrete. The crack pattern in the concrete in Figure 3-10b clearly shows the force transfer mechanism between the bars. When the force is transferred between adjacent bars, an outward radial pressure (see Figure 3-10c) is exerted against the concrete; this pressure causes splitting cracks along the bar. The occurrence of such cracks is the failure point, and the splice no longer transfers any force, as shown in Figure 3-11. The cracks initiate at the end where the radial splitting pressure is greater than the middle portion. This increased pressure causes large transverse cracks which occur at the end of the lap spliced bar, as shown in Figure 3-10b [74].

(a) Forces on bars at splice.

(b) Internal cracks at splice.

(c) Radial forces on concrete and splitting stresses shown on a section through the splice.

Figure 3-10 Tension lap splice [74].

Figure 3-11 Failure of a tension lap splice [74].

A commonly used technique to delay the occurrence and propagation of splitting cracks is through the provision of transverse reinforcement in the splice region.

Many researchers, including Orangun, Jirsa, and Breen, report no difference in development length requirements for single and lapped bars [60]. However, the ACI Code (Section 25.5) requires multipliers for lapped bars in most scenarios [68].

As described earlier, a 90° hooked bar in a reinforced concrete joint need to meet AASHTO LRFD requirements. To develop a bar in concrete, the AASHTO LRFD code specifies a length of tail extension of $12d_b$ for a 90° hook [53]. Previous studies on typical beam-column connections were used as bases for AASHTO LRFD and ACI requirement and have shown that the bend portion of the hooks can pop out when either short length of hook or smaller concrete cover are used [63],[72],[75]. However, this scenario is not common in closure joint regions where large confinement is provided by the adjacent concrete.

3.6 Numerical Study

3.6.1 Introduction

It is not always possible to experimentally study the effects of all variables due to cost and time limitations. In structural engineering, Finite Element (FE) modeling can complement results of experimental study and help in comprehending structure behavior. A reliable model can show the stress and strain distribution and predict the behavior of the system. Furthermore, a calibrated model can identify critical locations which can be used to optimize the structure. Although FE analysis may be computationally intensive than

analytical methods, it offers several advantages such as increased accuracy, design improvement, and a better insight into the design parameters.

Reinforced concrete is one of the most common used composite material in structures, which is composed of a steel bar and concrete matrix. From a structural standpoint, to achieve a composite action, loads should transfer from one material to another material through bond mechanism. Due to the importance of bond mechanism on the general behavior of structural member, it is essential to fully understand this mechanism [76]. However, there are a number of difficulties to measure bond through experimental studies. The primary issue is that bond is an internal mechanism and cannot be monitored through external observatory methods. In addition, using the strain gauges to measure the bond requires the application of adhesives and waterproofing materials which decimates the bond interface, which is the object of the study. Considering these issues, a numerical FE technique can help to comprehend the bond behavior. There are several commercially available FE software that can be adopted for numerical study such as ABAQUS, ANSYS, and ATENA. Each of these software consider different material models to simulate the real behavior of concrete and steel bars [77].

The objective of this section is to develop FE analysis modeling techniques, which is applicable for the bond mechanism. In this study, only three-dimensional, and static model with a monotonic loading were used. ATENA can simulate concrete with Fracture-Plastic Model, which includes the following effects of concrete behavior [78] (see Figure 3-12):

- Non-linear behavior in compression including hardening and softening,
- Fracture of concrete in tension based on the nonlinear fracture mechanics,

- Biaxial strength failure criterion,
- Reduction of compressive strength after cracking,
- Tension stiffening effect,
- Reduction of the shear stiffness after cracking (variable shear retention),
- Two crack models: fixed crack direction and rotated crack direction.

Based on ATENA documentation, the reinforcement can be modeled in two forms, smeared and discrete. In smeared concept, a perfect bond between concrete and steel bars is considered, therefore bond-slip cannot be directly modeled [78], unlike the discrete model. In both forms of modeling reinforcement, ATENA assumes the uniaxial stress state and a multi-linear stress-strain diagram, as shown in Figure 3-13.

Figure 3-12 Uniaxial stress-strain law for concrete in ATENA [78]

Figure 3-13 The multi-linear stress-strain law for reinforcement in ATENA [78].

The compressive failure for concrete is assumed as a biaxial stress failure criterion based on research conducted by Kupfer et al. (1969) [79].

Figure 3-14 Biaxial failure function for concrete [78].

For modeling of the cracks, ATENA considers two options; the fixed crack model and the rotated crack model, as shown in Figure 3-15. Both the models show crack formation when the principal stresses exceed the tensile strength, which are uniformly distributed.

Figure 3-15 Stress and strain state a) Fixed crack model b) Rotated crack model [78].

3.6.2 Modeling the Bond Behavior of the Straight Bars in Pull Out Test

An extensive FE investigation was employed to model the bond behavior. In this study, solid elements were employed for both concrete and steel reinforcement. For simplicity, the reinforcing bar is modeled as a line element embedded in concrete assuming full contact. However, while considering bar as a line element, the bond-slip behavior

cannot be captured. To simulate real behavior of reinforcing bar embedded in concrete, solid elements including the ribs are more useful. However, the solid model is more numerically intensive.

ASTM A615 specifies the properties of deformed and plain billet-steel bars for concrete reinforcement in detail [80]. The standard includes the chemical composition, deformation geometry, tensile strength, bending strength, and weight of reinforcing bars. Rib spacing, angle, gap, and height are the main characteristic of the geometry in the deformed reinforcement (see figure 3-16). Table 3-1 describes the requirements of deformed bar based on ASTM A615 specification.

Figure 3-16 Deformation geometry for reinforcing bar specifications [60].

Table 3-1 ASTM A615 requirements for reinforcing bar deformations [80]

Bar Size	Maximum Avg. Spacing (in)	Minimum Avg. Height (in)	Maximum Gap (in)
#3	0.262	0.015	0.143
#4	0.350	0.020	0.191
#5	0.437	0.028	0.239
#6	0.525	0.038	0.286
#7	0.612	0.044	0.334
#8	0.700	0.050	0.383
#9	0.790	0.056	0.431
#10	0.889	0.064	0.487
#11	0.987	0.071	0.540
#14	1.185	0.085	0.648
#18	1.580	0.102	0.864

To calibrate the FE model, an experimental study conducted by Eligehausen et al. [81] was used. They evaluated the bond stress-slip relationships between the #8 deformed reinforcement and concrete with a strength of 4.35 ksi using pullout test. The tested reinforcement was embedded only for a short length of $5d_b$ (Figure 3-17). The results of the test were presented as their ultimate capacity of the specimens. Finally, they concluded that the bond resistance was approximately proportional to the square root of concrete compressive strength ($\sqrt{f'_c}$).

Figure 3-17 Schematic representation of Elgehausen's tests [82].

ATENA software was used to simulate the test specimen with the same geometric and boundary conditions as that of the experimental test study. The FE model was subjected to a displacement-controlled loading, and, to simulate a quasi-static analysis, the load was applied in small increments.

To simulate the actual boundary conditions, the simulated model was restrained in all three directions. A quadratic tetrahedral solid element (four-node) was used to model both the concrete and embedded reinforcement bar. To reduce the computational run time of analysis, a quarter of the model was modeled (Figure 3-18 (b)). Figure 3-18 (a) shows the comparison between calibrated FE modeling and the experimental results. The experimental study and the FE model shows a good correlation. The load displacement plot shows a higher stiffness and load carrying capacity of the FE model when compared with experimental study. This difference is attributed to variation in boundary condition defined in the FE model.

Figure 3-18 Modeling results of Eligehausen test a) Comparison between calibrated FEM results and experimental test b) FE modeled specimen.

Several researchers found that the bond strength was proportional to the square root of concrete compressive strength ($\sqrt{f'_c}$). The calibrated FE model was used to investigate the effect of compressive strength of surrounding concrete. Eligehausen model was used for different concrete strength and the results are shown in Figure 3-19. The parametric study results show that the bond strength increases proportionally to the compressive strength of concrete. The results indicate that increasing the strength of surrounding concrete will increase the bond resistance. Using curve-fitting for the data obtained from FE analysis, a formula is derived which closely matches the findings of the study conducted by Eligehausen et al.

Figure 3-19 Relationship between strength of concrete and bond strength.

3.6.3 Modeling the Bond Behavior of the Hooked Bars in Pull Out Test

The common way to anchor a steel bar in reinforced concrete members is extending the bar beyond the maximum stress demand point. However, in some structural members, the available space is not sufficient to extend the straight bars. In these cases, using hooked bars and headed bars are an alternative option to properly transfer the forces. Generally, these type of end anchorage bars shows different mechanism of force transfer compared to straight bars. Due to the complicated behavior of bond mechanism through the end anchorages, it is important to have a clear understanding of this behavior. A reliable FE analysis can provide useful information to predict stress distribution along the bond interface and, in particular, the failure mode of member.

To calibrate the proposed FE model, the experimental results of Minor and Jirsa conducted at the University of Texas at Austin were used [65]. In that study, the anchorage capacities of hooked deformed reinforcing bars were evaluated. They investigated the factors which influence the strength of hooked bar anchorages. The 80 specimens were loaded in a pullout setup containing 37 different bar configurations until the hooked bar

failed (Figure 3-20). Bond length, angle included in the bend, inside radius of bend, and bar diameter were the main parameters. The dimensions of concrete block were selected to prevent fracture of the block by splitting during testing. For the bar size, Minor and Jirsa tested #5, #7, and #9 deformed grade 60 reinforcing bars. The average compressive strength of test specimens was between 3300 psi to 5500 psi in their study [65].

Figure 3-20 Plan view of the test by Minor and Jirsa [65].

Considering the material properties of the tested specimens by Minor and Jirsa, the concrete block with a deformed hooked bar #7 was modeled and employed to calibrate the FE model. Figure 3-21 shows the geometry of the calibrated FE model.

Figure 3-21 Hooked bar numerical model details.

The geometry of the ribs on a deformed hooked bar and the surrounding concrete at the interface of the ribs were modeled as 3-D solid elements.

The geometry of the ribs, including rib spacing and rib height in the FE model were assumed based on the ASTM A615 requirements for reinforcing bar deformations. (Table 3-1). All the modeling elements properties of the model are the same as straight bar illustrated in section 3.6.2. The FE model is shown in Figure 3-22.

Figure 3-22 Stress distribution of the modeled hooked bar in the concrete block of the hooked bar.

As mentioned earlier, the hooked bar shows different stress distribution compared to straight bar. Figure 3-23 illustrates the mechanism of stress transfer in hooked bar.

Figure 3-23 Figure Crack formations and rebar stresses in the numerical model

It was found that generally, when the 90° hook is fully developed, concrete in front of the hook, in the inner radius of the hooked bar, will crush and consequently the tail extension of the hooked bar tends to pulled straight around the bend as shown in Figure 3-24 [60] and Figure 3-25. However, this scenario is not common in closure joint regions where large confinement is provided by the adjacent concrete.

Figure 3-24 Stress transfer in the hooked bar a) FE model, b) Thompson et al. model [60].

Figure 3-25 Crack propagation in FE model.

Figure 3-26 shows the calibrated FE modeling results compared to the experimental results. The results of the numerical model indicate that the calibrated model can predict the behavior of the deformed hooked bar within a range of 4%.

Figure 3-26 Comparison between calibrated FEM results and experimental test.

3.6.4 Modeling the Bond Behavior of Spliced Bars in Beam Test

The bond of steel bar to surrounding concrete is comprised of three components, i) chemical adhesion ii) friction, and iii) mechanical interaction [83,84]. The bond mechanism is a function of these components and may affect the performance of bond behavior. Many material properties and geometric design characteristics affect these bond mechanisms, including bar size (i.e. diameter), rib configuration (e.g., height, angle, spacing, etc.), presence of coating on the bar, concrete strength, bar strength, confining pressure provided by transverse reinforcement, and the length of the anchorage or splice zone [85].

The results of experimental efforts on bond behavior can be augmented by FE simulations. An extensive numerical and experimental data is available, however, new

developments in material technology and design warrant further investigation on bond behavior. These studies are essential to understand the mechanism of force transfer and have implications on the design of structures. Although testing and measurement technologies have improved but FE simulations remain an important tool to understand the bond behavior, which would otherwise be impossible to observe without compromising the validity of the tests. It is imperative that the FE model is calibrated, which requires careful consideration, especially for bond-zone response. FE model for bond has been developed in a way that represents bond behavior in beam specimens.

3.6.4.1 Description of FE Model

Geometric Configuration

In order to better understand the bond response, steel reinforcement in splice region should be modeled with 3-D rib scale modeling approach. In this approach, the geometry of the ribs on a deformed steel bar was represented based on ASTM A615 specification as described in section 3.6.2. (See Table 3-1 and Figure 3-16). Figure 3-27 shows the geometric configuration of the bond region, including the reinforcing bar, ribs, and adjacent surrounding concrete.

Figure 3-27 Geometric configuration of the bond region: (a) 2D view and (b) 3D view of reinforcing bar.

To simplify the FE model, ribs were defined as a series of discrete rings oriented perpendicular to the bar axis, as shown in Figure 3-27 (b). This modeling assumption deviates from the as milled condition consisting of a continuous spiral placed at an angle to the bar axis. However, this modeling discrepancy is assumed to have minimal impact on the behavior of the bond.

Many experimental tests observed that bond response at rib scale is dominated by the bearing of the ribs and shear interface had little effect on the bond behavior [84–87]. Therefore, the contact model between steel and concrete was simulated as a fixed contact. In fixed contact, adhesion and friction are not taken into consideration, and the bond mechanism is governed by the steel rib bearing against the surrounding concrete. [85].

Using fixed contact, the FE model, with appropriate mesh size, can capture the stress distribution of the bar and associated concrete crack opening at different load levels. The literature review indicates that a three dimensional rib scale model can potentially capture the mechanism of bond behavior. Another modeling approach can assume an interface medium, but which can be computationally expensive and calibration of such model may require additional testing to determine adhesion or friction coefficients.

Loading and Boundary Condition

The numerical model of the concrete beam consists of concrete beam, and a pair of lap spliced longitudinal bars. The rib scale modeling approach mentioned earlier was used for modeling purpose. As shown in Figure 3-28, the beams were supported on rigid plates which were constrained for vertical displacement. The loading plates on top of the beam were modeled with fixed contact surface with the concrete beam.

Figure 3-28 Typical beam splice test specimen in FE model.

Element Type and Material Model

The material model of the concrete used quadratic tetrahedral element with four nodes. The reinforcement bar was also modeled with the same element. The rib scaled modeling requires a more refined mesh to capture the performance at the interface. For modeling purpose, a mesh sensitivity analysis was carried out based on which a mesh size of 0.2 was used. The total number of elements resulting from this mesh size was 1,241,593.

Figure 3-29 Mesh view of spliced region.

In this study, the Fracture-Plastic model was used to simulate normal strength concrete (NSC), high strength concrete (HSC) and ultra-high performance concrete (UHPC), which are presented in Table 3-2. The bi-linear stress-strain diagram was used to simulate the reinforcement; Table 3-3 shows the material properties employed in FE analysis for reinforcement. (see Figure 3-30)

Table 3-2 Concrete properties

Parameter	UHPC	HSC	NC
Compressive strength	22.1 ksi	15 ksi	5 ksi
Modulus of elasticity	7200 ksi	6381 ksi	4400 ksi
Poisson ratio	0.2	0.2	0.2
Tension strength	1.4	0.71	0.1

Table 3-3 Reinforcement properties

Parameter	
Yield strength	60 ksi
Modulus of elasticity	29000 ksi
Poisson ratio	0.3
Hardening Modulus	1450 ksi

Figure 3-30 The bi-linear stress-strain law for reinforcement in ATENA.

3.6.4.2 Parametric Study

As previously mentioned, the behavior of a bond is a function of many variables. Among these variables, the bond behavior can be affected by the material properties of concrete and steel, the geometric properties of reinforcing steel and the lap splice length. Therefore, the performance of the bond has direct bearing on the simulated response of the specimen.

Consequently, a parametric study that can investigate the variables affecting bond behavior in splice region could assist to better understand the general behavior of the reinforced concrete structures.

In FE modeling, concrete and steel have many input parameters among which compressive and tensile strength of concrete and lap splice length were expected to have significant impact on simulated beam response. These two parameters were examined as study variables. A series of FE analyses were carried out to investigate the sensitivity of simulated beam response to variation in these parameters.

Effect of Compressive Strength of Concrete

The FE model, calibrated based on previous research, was used to investigate the effect of concrete compressive strength on the beam behavior with different lap splice lengths. The compressive strength of concrete in the FE model was varied from 5 to 22 ksi. This range of strength was intended to simulate NSC, HSC, and UHPC. For each material strength, two different lap splices were used in the modeling, i.e. 4-in and 18-in. These two lengths represent adequate and inadequate lap splices in order to see two different modes of failure. The tensile strength of concrete varied as a function of compressive strength. The details of these six cases are tabulated in Table 3-4.

Table 3-4 Specimens geometric and material variables

Specimen	Bar size	Splice length(in.)	Compressive strength (ksi)	Tensile strength (ksi)
I	#6	4	5 (NSC)	0.1
II	#6	4	15 (HSC)	0.71
III	#6	4	22 (UHPC)	1.4
IV	#6	18	5 (NSC)	0.1
V	#6	18	15 (HSC)	0.71
VI	#6	18	22 (UHPC)	1.4

The von-mises stress distribution is shown in Figure 3-31 and Figure 3-33 for 4-in and 18-in lap splice, respectively. The insufficient lap splice results in high stresses resulting from bearing of concrete on the ribs and the specimen. The load deflection curves for 4-in splice for different concrete shows an improvement in load values as the concrete strength improves. This observation can be noticed in NSC and HSC specimens, which show a sudden drop in load after peak load, as shown in Figure 3-32.

In the three models with 18-in lap splice, the stresses are more uniformly distributed along the length of spliced bar, as shown in Figure 3-33. The load displacement curves for different concrete strengths show improved post-yielding behavior. All FE models exhibit sufficient ductility before failure, as shown in Figure 3-34.

Figure 3-31 Rebar stresses (von-mises) in the numerical model when the lap splice length is 4 in.

Figure 3-32 Effect of compressive strength of concrete when lap splice length is 4 in.

Figure 3-33 Rebar stresses (von-misses) in the numerical model when the lap splice length is 18 in.

Figure 3-34 Effect of compressive strength of concrete when lap splice length is 18 in.

Effect of Lap Splice

The FE model, calibrated based on previous research was used, to investigate the effect of lap splice lengths. The lap splice in the FE model was varied from 4 and 18 inches. The load displacement curves for each concrete strength with different lap splice are shown in Figure 3-35 through Figure 3-37. The curves indicate that at initial stages the specimen behave similarly with the same initial stiffness for both 4-in and 18-in lap splices. After cracking of concrete, the steel starts to yield. Due to insufficient lap splices, the specimen with insufficient lap splices (4-in, lap splice length) fail with an abrupt loss of strength. The high mechanical strength of UHPC allows improved ductile behavior when compared to high strength and normal strength concrete.

Figure 3-35 UHPC comparison between 4" and 18" lap splice.

Figure 3-36 HSC comparison between 4" and 18" lap splice.

Figure 3-37 NSC comparison between 4'' and 18'' lap splice.

3.6.4.3 Discussion

The objective of numerical study was to evaluate the performance of straight and hooked lap splices. A parametric study was carried out on models calibrated with experiments carried out by previous researchers. The parameters evaluated in this study included concrete strength and lap splice. The results of the study show that an increase in concrete compressive and tensile strength results in improved performance of the bond. The tensile strength of UHPC is higher than conventional and high strength concrete, which results in significant increase in ductility of the specimens.

As discussed in Chapter 1, a number of options have been developed and implemented in the field to connect bridge deck elements with closure joints. These details include mechanical connections such as lap spliced bars, headed bars, and hooked bars. Also, material strength of closure joint has been shown to improve the ultimate capacity and ductility. This has led to the development of closure joints using enhanced materials such as UHPC which allows a reduction in development length while providing a durable

detail. However, the commercial value of UHPC is many folds that of normal concrete. Therefore, keeping in view an economical joint, a hooked bar is investigated, which is an alternative option reducing the joint width while providing adequate structural capacity.

3.7 Development of Design Criteria

Although the hooked bars have been used extensively in different applications, especially in buildings, their use in closure joints is limited due to the lack of design recommendations. To this extent, an extensive experimental study on new closure joint detail using a hooked bar was conducted to fill the knowledge gap in current provisions and develop design criteria.

3.7.1 Experimental Study

To evaluate the structural performance of the proposed closure joint detail, an experimental study was conducted at Florida International University (FIU). The test specimen is designed for unit width of the deck in the transverse direction where moments are transferred through closure joint. The thickness of the slab was chosen to be 8 in. for all specimens, which is a typical thickness for bridge decks, as shown in Figure 3-38.

Figure 3-38 Schematic view of test specimens.

Based on the test program, the experiments were conducted in three phases. Each phase was investigated for different parameters, including lap splice (L_p) length, the lateral distance between hooked bars (Gap Space (GS)) and the use of transverse bar (TR) (each variable is defined in Figure 3-39). In this detail, lap splice (L_p) was measured from outside to outside of hooked bars, as shown in Figure 3-39. The gap space in Figure 3-39 represents the transverse spacing of noncontact splices, which is limited by the smallest value of one-fifth of lap splice length or 6.0 in. [33]. For this reason, a gap space of 2.0 in. and 4.0 in. are considered as lower and upper limits, respectively. The empirical design of deck typically uses No. 4 bar, but No. 6 bar was also considered as an upper limit. To investigate the optimal lap splice length, a splice length of 2-in. to 10-in. was considered with a 2-in. increment.

Figure 3-39 Test specimen variables.

To meet the requirement of the AASHTO-LRFD Bridge Design Specifications [59] for the confinement of hooks, the side cover is considered as 2 in. For each bar size, the range of L_p is incrementally varied from 2 in. to 10 in. with 2-in. increment. For bar size (BS), No. 4 and No. 6 are considered which are commonly used in bridge decks. Each group consists of 12 beams with a length of 8 ft. and a depth of 8 in. The design procedure for the design of modular bridge deck is similar to a conventional cast-in-place decks based on the AASHTO-LRFD Bridge Design Specifications [59]. The specimens were tested in a reverse test setup where the tension side was located at the top of the specimens due to some limitations in test setups. Figure 3-40 shows an elevation view of the reversed test setup.

Elevation View

Figure 3-40 Specimen geometry, reinforcement, and reversed test setup.

During the first phase, hereafter, indicated by Group A, 12 test specimens were constructed with different lap splices ($2 \text{ in.} \leq L_p \leq 10 \text{ in.}$) with 2-in. increment. The bar sizes for lap splice were No. 4 and No. 6 with a constant gap space of 2 in. In the second phase, hereafter, indicated by Group B, another 12 specimens were constructed with the details identical to Group A, except for a gap of 4-in. between the longitudinal hooked bar in the closure joints region. Finally, Groups C and D test specimens were constructed with details identical to Groups A and B, with additional transverse reinforcement which is not used in Groups A and B. These transverse bars consisting of No. 3 bars at a spacing of 2.5 in. were placed over the hooked bars in the closure joint. This transverse reinforcement represents longitudinal reinforcement that is usually placed in the longitudinal closure joints. All the test specimens designated as S/ST-GS-BS- L_p

where: S is a notation for the specimens without transverse bars;

ST is a notation for the specimens with transverse bars in closure region;

GS is a notation for the lateral distance between the hooked bars (Gap Space) in inches;

BS is a notation for bar size in terms of bar number; and

L_p is a notation for lap splice length in inches.

For instance, S-2-4-2 specifies the specimen with no transverse reinforcement within closure joint, with a 2-in. lateral distance between the hooked bars, No. 4 bar size, and 2-in. lap splice length. Table 3-5 shows the specimen matrix details.

Table 3-5 Specimen Matrix, Unit: in.

Group	ID	G S (in.)	Bar Size	L _p (in.)	Transverse Bar	Joint Width (in.)
A	S-2-4-2	2	#4	2	No	6
	S-2-4-4	2	#4	4	No	8
	S-2-4-6	2	#4	6	No	10
	S-2-4-8	2	#4	8	No	12
	S-2-4-10	2	#4	10	No	14
	S-2-6-2	2	#6	2	No	6
	S-2-6-4	2	#6	4	No	8
	S-2-6-6	2	#6	6	No	10
	S-2-6-8	2	#6	8	No	12
	S-2-6-10	2	#6	10	No	14
	S-2-4-control	2	#4	Control	No	N/A
	S-2-6-control	2	#6	Control	No	N/A
B	S-4-4-2	4	#4	2	No	6
	S-4-4-4	4	#4	4	No	8
	S-4-4-6	4	#4	6	No	10
	S-4-4-8	4	#4	8	No	12
	S-4-4-10	4	#4	10	No	14
	S-4-6-2	4	#6	2	No	6
	S-4-6-4	4	#6	4	No	8
	S-4-6-6	4	#6	6	No	10
	S-4-6-8	4	#6	8	No	12
	S-4-6-10	4	#6	10	No	14
	S-4-4-control	4	#4	Control	No	N/A
	S-4-6-control	4	#6	Control	No	N/A
C	ST-2-4-2	2	#4	2	#3	6
	ST-2-4-4	2	#4	4	#3	8
	ST-2-4-6	2	#4	6	#3	10
	ST-2-4-8	2	#4	8	#3	12
	ST-2-4-10	2	#4	10	#3	14
	ST-2-6-2	2	#6	2	#3	6
	ST-2-6-4	2	#6	4	#3	8
	ST-2-6-6	2	#6	6	#3	10
	ST-2-6-8	2	#6	8	#3	12
	ST-2-6-10	2	#6	10	#3	14
	ST-2-4-control	2	#4	Control	#3	N/A
	ST-2-6-control	2	#6	Control	#3	N/A
D	ST-4-4-2	4	#4	2	#3	6
	ST-4-4-4	4	#4	4	#3	8
	ST-4-4-6	4	#4	6	#3	10
	ST-4-4-8	4	#4	8	#3	12
	ST-4-4-10	4	#4	10	#3	14
	ST-4-6-2	4	#6	2	#3	6
	ST-4-6-4	4	#6	4	#3	8
	ST-4-6-6	4	#6	6	#3	10
	ST-4-6-8	4	#6	8	#3	12
	ST-4-6-10	4	#6	10	#3	14
	ST-4-4-control	4	#4	Control	#3	N/A
	ST-4-6-control	4	#6	Control	#3	N/A

3.7.2 Construction of Test Specimens

The structural performance of a closure joint was replicated under a flexure test for a unit width. The closure joint was cast in two stages. During the first stage, the adjoining decks were cast in a formwork separated by a closure joint. Upon the completion of curing for a minimum of 28 days, the closure joint was cast. The concrete used for both the slab and closure joint had a compressive strength of a minimum of 5 ksi (FDOT CL II deck concrete). Before casting of the closure joints, no surface preparation was performed on cast deck portion. This condition represents the most unfavorable condition of a cold joint. In addition to the test group specimens, control specimens of monolithic construction were made. These control specimens replicate the cast-in-place slab of conventional bridges that have no closure joints and contain straight bars. The test specimens were painted in white to document crack initiation and propagation during the test. Construction procedure of test specimens is shown in Figure 3-41.

Figure 3-41 Specimen construction procedure a) formwork, b) casting deck portions, c) spliced hooked bars in closure joint, d) casting closure joint, e) final test setup.

3.7.3 Test Setup and Loading Procedure

The test specimens were tested under a four-point loading setup as the main scope of this study is to investigate the flexural behavior and lap splice length requirement. A displacement controlled monotonic loading was applied to determine the moment capacity of the closure joints. The distance between the hydraulic jacks was 6 ft. and the roller supports were spaced at 3 ft. apart as shown in Figure 3-42 (actual test setup) and Figure 3-43 (schematic test setup). The hydraulic jacks were reacted against a spreader beam, which was anchored to the strong floor. To evaluate the behavior of the specimens, the test setup was instrumented with string potentiometers, load cells, and pressure transducers. Each specimen was loaded to failure. The deflection was measured at the mid of the slab and loading points.

Figure 3-42 Actual experimental test setup.

Figure 3-43 Schematic experimental test setup.

3.7.4 Material Tests

Cylinder test of 4x8 in. based on ASTM C39/C39M [88] specification was used to determine the compressive strength of normal concrete at test days as shown in Figure 3-44. The compressive strength of conventional concrete for the slab and closure joint regions are summarized in Table 3-6. All compressive strength values passed 5,000 psi as required.

ASTM A615 Grade 60, No. 3, No. 4, and No. 6 steel reinforcing bars were used for longitudinal reinforcement in all specimens and transverse bars for specimens in groups C and D. Four segments of each bar were used for tensile testing as shown in Figure 3-45. The resulting values of yield and ultimate strength are mentioned in Table 3-7.

Table 3-6 Compressive Strength of Outer Sections of Slab Specimens

Region	Sample #	Compressive Strength (psi)
Slab	1	7744
	2	6230
	3	7794
	4	7189
	5	6080
	6	6974
	7	6852
	8	6021
	Average	6860
Closure joint	1	6692
	2	6273
	3	6130
	4	6127
	5	6772
	6	6525
	Average	6420

Table 3-7 Bar Test Strengths

Test Specimen	Yield Strength (psi)	Ultimate Strength (psi)
Straight/Hooked No. 4	64,500	103,800
Straight/Hooked No. 6	68,000	113,000
Straight No. 3	61,700	100,100

Figure 3-44 Cylinder test.

Figure 3-45 Reinforcing bar test.

3.7.5 Experimental Results

3.7.5.1 Mode of Failure and Crack Pattern

Resistance against pull-out of deformed reinforcing bars embedded in concrete is mainly provided by bearing of ribs against concrete. Although adhesion and friction are present when a deformed bar is loaded, these bond-transfer mechanisms are quickly lost, leaving the bond to be transferred by bearing on the deformations of the bar [89]. For most structural members, bond failure is governed by concrete splitting.

The mode of failure for a closure joint with an inadequate splice length occurs at the closure joint region. by increasing the applied load, the force is transferred to the closure joint and concrete splits before the yielding occurs due to insufficient lap splice length, which is referred to as a bond failure as shown in Figure 3-46. However, for the flexure failure, the spliced bars reach their yielding, and the subsequent cracks are distributed along the tension zone. The flexure failure is a preferred mode of failure for the closure joints

and implies that the splice length is sufficient for these specimens (Figure 3-47). The modes of failure for all test specimens are summarized in Table 3-8.

Figure 3-46 Bond failure.

Figure 3-47 Flexure failure.

The specimens with No. 4 bar in Group A failed in flexure with the exception of the specimen with 2-in. lap splice length, which is not surprising due to short lap splice length of only four times bars diameter for bar No. 4.

For specimens with No. 6 in Group A, all failure modes were bond failure except for specimen with 10-in. lap splice length. This concluded that even eleven times the bar diameter of No. 6 is not sufficient to yield the spliced No. 6 bars. The specimens in Group B with No. 4 bars failed in flexure except 2-in. and 4-in. splice length. Since the only difference between Group A and Group B that specimens in Group B have bigger lateral distance between reinforcement which led to the need of at least twelve times bar diameter instead of eight times bar diameter for the case of smaller lateral distance between reinforcement for bar No. 4 for specimens in Group A.

For specimens with No. 6 bars in Group B failed in bond which concluded that increasing the lateral distance between the spliced bars caused that lap splice length to be insufficient even with over 13 times the bar size.

The specimens with No. 4 bars in Group C failed in bond failure mode except 8-in. and 10-in. splice length. For specimens with No. 6 bars in the same group, all specimens failed in bond failure mode except for specimens with 8 in and 10 in. splice length. The specimens with No. 4 in Group D bars failed in bond except 10-in. splice length. For specimens with No. 6 bars in the same group, all specimens failed in bond. The reason Groups C and D failed in bond was due to a weak plane caused by the presence of transverse bars. The modes of failure for all test specimens are summarized in Table 3-8.

Table 3-8 Test Specimens Modes of Failure

Group	ID	Bar Size	G S (in.)	L _p (in.)	Transverse Bar	Mode of Failure
A	S-2-4-2	#4	2	2	No	Bond Failure
	S-2-4-4	#4	2	4	No	Flexural Failure
	S-2-4-6	#4	2	6	No	Flexural Failure
	S-2-4-8	#4	2	8	No	Flexural Failure
	S-2-4-10	#4	2	10	No	Flexural Failure
	S-2-4-control	#4	N/A	N/A	No	Flexural Failure
	S-2-6-2	#6	2	2	No	Bond Failure
	S-2-6-4	#6	2	4	No	Bond Failure
	S-2-6-6	#6	2	6	No	Bond Failure
	S-2-6-8	#6	2	8	No	Bond Failure
	S-2-6-10	#6	2	10	No	Flexural Failure
	S-2-6-control	#6	N/A	N/A	No	Flexural Failure
B	S-4-4-2	#4	4	2	No	Bond Failure
	S-4-4-4	#4	4	4	No	Bond Failure
	S-4-4-6	#4	4	6	No	Flexural Failure
	S-4-4-8	#4	4	8	No	Flexural Failure
	S-4-4-10	#4	4	10	No	Flexural Failure
	S-4-4-control	#4	N/A	N/A	No	Flexural Failure
	S-4-6-2	#6	4	2	No	Bond Failure
	S-4-6-4	#6	4	4	No	Bond Failure
	S-4-6-6	#6	4	6	No	Bond Failure
	S-4-6-8	#6	4	8	No	Bond Failure
	S-4-6-10	#6	4	10	No	Bond Failure
	S-4-6-control	#6	N/A	N/A	No	Flexural Failure
C	ST-2-4-2	#4	2	2	#3	Bond Failure
	ST-2-4-4	#4	2	4	#3	Bond Failure
	ST-2-4-6	#4	2	6	#3	Bond Failure
	ST-2-4-8	#4	2	8	#3	Flexural Failure
	ST-2-4-10	#4	2	10	#3	Flexural Failure
	ST-2-4-control	#4	N/A	N/A	#3	Flexural Failure
	ST-2-6-2	#6	2	2	#3	Bond Failure
	ST-2-6-4	#6	2	4	#3	Bond Failure
	ST-2-6-6	#6	2	6	#3	Bond Failure
	ST-2-6-8	#6	2	8	#3	Flexural Failure
	ST-2-6-10	#6	2	10	#3	Flexural Failure
	ST-2-6-control	#6	N/A	N/A	#3	Flexural Failure
D	ST-4-4-2	#4	4	2	#3	Bond Failure
	ST-4-4-4	#4	4	4	#3	Bond Failure
	ST-4-4-6	#4	4	6	#3	Bond Failure
	ST-4-4-8	#4	4	8	#3	Bond Failure
	ST-4-4-10	#4	4	10	#3	Flexural Failure
	ST-4-4-control	#4	N/A	N/A	#3	Flexural Failure
	ST-4-6-2	#6	4	2	#3	Bond Failure
	ST-4-6-4	#6	4	4	#3	Bond Failure
	ST-4-6-6	#6	4	6	#3	Bond Failure
	ST-4-6-8	#6	4	8	#3	Bond Failure
	ST-4-6-10	#6	4	10	#3	Bond Failure
	ST-4-6-control	#6	N/A	N/A	#3	Flexural Failure

Cracks were mapped during the tests on each specimen to observe the order of cracks formation. At the onset of longitudinal crack formation, the tests were stopped for safety concerns. All remaining cracks were mapped upon the completion of testing, as shown in Figure 3-48. Initiation of cracks with bond failure mode occurred at the cold joints, and subsequent crack formation occurred over supports.

Figure 3-48 Crack pattern in closure joints region.

3.7.5.2 Load-Displacement Relationship

The comparisons of load-deflection curves for all groups are plotted from Figure 3-49 to Figure 3-52 with fixed horizontal and vertical axes. The test results show that the variables such as the bar diameter, the lateral distance between reinforcement, and lap splice length have a significant influence on the load carrying capacity of specimen.

In Group A, the deflection specimen with No. 4 bar increased with an increase of lap spliced length, while the maximum load carrying capacity of specimens remained the same which is due to better development length which led to better load transfer between the two adjacent deck segments. The test specimen S-2-4-2 exhibited the least displacement

before the occurrence of bond failure. For lap splice increments from 2 to 8 in., No. 6 bar was insufficient for tension development length, and bond failure occurred in these specimens. However, for lap splice length of 10-in., the failure mode was flexural. Figure 3-49-b shows that the specimens with No. 6 bars with insufficient lap splice length failed before the bar yielding occurred.

The lateral distance between the reinforcement (Gap Space) for specimens in Group B with No. 4 and No. 6 bars was increased from 2 in. to 4 in. The results show that increasing this gap space requires an increase of lap splice length, which is shown in Figure 3-50 (a) and Figure 3-50(b). The 4-in. gap space for specimens with No. 6 bars was insufficient for all splice lengths and failed in bond. Figures 3-49 and 3-50 show that reducing the gap spacing and increasing the bar diameter can improve the ultimate load capacity of the specimens due to an increase in reinforcement ratio.

Groups C and D have test parameters similar to Groups A and B but include transverse reinforcement in closure joint region. The test results of Groups C and D are presented in Figures 3-51 and 3-52. General design practice requires the use of these transverse bars for thermal and shrinkage requirements.

In Group C, the deflection specimen with No. 4 bar increased with an increase of lap spliced length, while the maximum load carrying capacity of specimens remained the same which is due to better development length which led to better load transfer between the two adjacent deck segments. The test specimens ST-2-4-2 and ST-2-4-4 exhibited the least displacement before the occurrence of bond failure. For lap splice increments from 2 to 6 in., No. 6 bar was insufficient for tension development length, and bond failure occurred in these specimens. However, for lap splice lengths of 8-in. and 10-in., the failure

mode was flexural. Figure 3-51-b shows that the specimens with No. 6 bars with insufficient lap splice length failed before the bar yielding occurred.

The lateral distance between the reinforcement (Gap Space) for specimens in Group D with No. 4 and No. 6 bars was increased from 2 in. to 4 in. The results show that increasing this gap space requires an increase of lap splice length, which is shown in Figure 3-52(a) and Figure 3-52-b. The 4-in. gap space for specimens with No. 6 bars was insufficient for all splice lengths and failed in bond. Figures 3-51 and 3-52 show that reducing the gap spacing and increasing the bar diameter can improve the ultimate load capacity of the specimens due to an increase in reinforcement ratio.

Adding transverse reinforcement in the closure joints acts as ties for spliced bars (due to their short length) to reduce cracks initiation and propagation. However, specimens in groups C and D failed in bond since the overall width of specimen was small, but in the case of large scale closure joints, this would not happen.

Figure 3-49 Group A, Experimental load-displacement result a) Specimens with No. 4 bars and 2-in. gap, b) Specimens with No. 6 bars and 2-in. gap.

Figure 3-50 Group B, Experimental load-displacement result a) Specimens with No. 4 bars and 4-in. gap, b) Specimens with No. 6 bars and 4-in. gap.

Figure 3-51 Group C, Experimental load-displacement result a) Specimens with No. 4 bars and 2-in. gap, b) Specimens with No. 6 bars and 2-in. gap.

Figure 3-52 Group D, Experimental load-displacement result a) Specimens with No. 4 bars and 4-in. gap, b) Specimens with No. 6 bars and 4-in. gap.

3.7.5.3 Measured Ductility

Since the behavior of the concrete beams is not perfectly elastic-plastic, the ductility parameters (Δ_y and Δ_{max}) were obtained from the idealized load-displacement curve. The

idealized curve consists of two regions: linear elastic and plastic. The yield displacement (Δ_y) was calculated based on initial stiffness, and maximum load applied, as shown in Figure 3-53.

Displacement ductility capacity (μ) of a member can be calculated using the following equation:

$$\mu = \frac{\Delta_{max}}{\Delta_y} \quad (3-9)$$

Where;

Δ_y : Idealized elastic displacement of the tested beam

Δ_{max} : the maximum displacement of the tested beam

Considering the described equation above, the ductility of the specimens related to lap splice length is plotted in Figures 3-54 and 3-55. The presence of transverse reinforcement in closure joint improves displacement ductility and the tension development is possible with shorter splice length. Based on the test results, it is concluded that a certain level of displacement ductility and load carrying capacity is needed for bar development length. These limits control the design of lap splice length for hooked bar and failure by either flexure or bond is of secondary importance.

Figure 3-53 Measuring ductility approach.

Figure 3-54 Ductility against different lap splice length a) Specimens with No. 4 bars and 2-in. gap, b) Specimens with No. 6 bars and 2-in. gap.

Figure 3-55 Ductility against different lap splice length a) Specimens with No. 4 bars and 4-in. gap, b) Specimens with No. 6 bars and 4-in. gap

3.8 Design Recommendations

The performance of a tension splice is considered similar to that of an identical component in which the reinforcing bars are continuous. Thus, to comply with general ACI design philosophy due to the lack of the same requirement in AASHTO-LRFD, the members with tension splice should exhibit some level of ductility. The ACI requirement specifying $1.25f_y$ (section 12.14) provides insufficient knowledge of the level of ductility (curvature of displacement) in a member with tension splice. Therefore, in order to develop design criteria, strength and ductility criteria should be incorporated. The displacement ductility ratio, described in the previous section, was used to express the ductility criteria. A displacement ductility ratio greater than one signifies: firstly, those longitudinal bars are capable of developing at least their actual yield stress, and secondly, specimens are capable of reaching deformation levels corresponding to limits beyond the first yield displacement [89].

Each data point in Figures 3-54 and 3-55 represents the displacement ductility ratio achieved by each of 48 test specimens. Based on the structural performance of the specimens compared to the calculated ductility ratio, it might be concluded that the

specimens achieving a displacement ductility ratio of greater than 3 shall be considered satisfactory. Graybeal et al. [25] conducted several tests for UHPC closure joints and the similar corresponding ductility was used by the author. It was estimated that the UHPC closure joint specimens reached a ductility ratio of 3 to 4. Since the UHPC closure joints have been implemented in practice in many bridges with same ductility range The performance of these UHPC joints has been satisfactory. Therefore, it can be concluded that performance of closure joints with 90° degree hooked reinforcement is satisfactory since it reaches the same ductility range as UHPC closure joints with straight bars.

Results of the experimental tests are summarized in Table 3-9. Following is a suggested design recommendation for the closure joint detail recommended in this research. It should be noted that until further tests are performed, this is a conservative recommendation.

For specimens with No. 4 and No. 6 reinforcement, using the 90° hook detail, as shown in Figure 3-56, lap splice length should be at least 12 times diameter of the reinforcement. Further, at least three No. 3 reinforcement, acting as confining reinforcement and running parallel to the closure joints (as shown in Figure 3-56) should be provided over the spliced hooked reinforcement. In case of the absence of the transverse bars, lap splice length should be at least 14 times diameter of the reinforcement. The maximum gap spacing should be limited to 4 in. until further research is conducted to study its effect on joint performance.

Table 3-9 Design Recommendations

Bar Size	Gap Space (in.)	Without Transverse Bar	With Transverse Bar
#4	2	8db (4 in.)	8db (4 in.)
#6	2	14db (10 in.)	8db (6 in.)
#4	4	12db (6 in.)	8db (4 in.)
#6	4	14db (10 in.)	11db (8 in.)

Figure 3-56 Design recommendation details.

CHAPTER 4 SUMMARY AND CONCLUSION

4.1 Conclusions

Many details are used in closure joints in Accelerated Bridge Construction, however, none of these details have comprehensively solved design and construction issues, for example in the case of using headed reinforced bars, an issue of reduced concrete cover is present. In the case of using straight reinforcing bars, a very wide closure joint is present which can cause shrinkage cracking. In case of the use of UHPC, high cost of material may result bidding and contracting issues. In case of using post-tensioning tendons, corrosion and the need of re-jacking the tendons are present.

An ideal closure joint detail should address all design and construction issues while satisfying the structural performance such as load-carrying capacity and ductility. Design criteria should be established to facilitate the use of the proposed closure joint.

The objective of this research was to focus on evaluating a hooked detail in the closure joints through experimentation and numerical work. To this extent, 48 beam specimens of 1 ft. strip width of bridge deck were tested under flexural loading scheme to examine the effect of reinforcing bar size, lap splice length, the lateral distance between reinforcing bars, and the use of transverse reinforcement. Four groups of specimens were tested. The first group included 12 specimens with bar size 4 and 6 with 2 in. lateral distance between the spliced reinforcing bars and lap splice length ranges from 2 in. to 10 in. The second group differs from the first group by changing the lateral distance to 4 in. while the remaining parameters stay the same. The third and fourth groups replicated the first and second groups, respectively, with one exception of adding transverse

reinforcement. After the completion of test, the load-displacement curves were plotted, and mode of failure was identified for each specimen. The specimen either failed in flexural or in bond. For better understanding of the proposed joint detail performance, the ductility ratio of each specimen was estimated. By comparing these ductility ratios with current state-of-the-art research, the ductility ratio was more than three, which was considered satisfactory.

In addition, numerical analysis was carried out to better comprehend the bond mechanism, which is otherwise difficult to observe. A calibrated FE model was then used to carry out study on effect of two parameters: 1) lap splice, and 2) material strength. Results of numerical analysis show that lap splice is dependent on material properties and should be decreased with the use of higher compressive strength.

Based on the conducted research, the following specific conclusions are made:

1. For the 90 degree hook detail, AASHTO-LRFD Specification (similar for ACI 318-11) requires 17 times the diameter of the reinforcement as lap splice length, as compared to 12 to 14 times the diameter of the reinforcement recommended in this study. The main reason for this observed behavior is that AASHTO-LRFD and ACI design recommendations were developed based on test specimens simulating 90 degree hook detail in beam-column connections. In the case of deck slab, significant confinement for the 90 degree hook detail is provided by concrete in the prefabricated decks adjacent to the closure joint. This additional confinement helps to reduce the required lap splice length.
2. The design recommendations in this research are based on providing sufficient ductility by, providing sufficient tension lap splice length, decreasing the lateral

distance between spliced reinforcement, and including transverse reinforcement. The recommended design provisions ensure achieving acceptable levels of ductility before failure, which could be by either flexure or bond failures. Achieving certain level of ductility ensures the joint performs adequately, and in such cases, the mode of failure can be considered of secondary importance.

3. Performance of closure joints with 90 degree hook reinforcement is considered satisfactory as it reaches a ductility ratio between 3 and 4. This ductility ratio range is similar to the one obtained from UHPC closure joints with straight bars which have performed well in the field.
4. The finite element model for 90 degree hook bar in pull out test was able to predict the response of the structure with the same crack pattern and stress distribution from experimental results in the literature.
5. The numerical study of the effect of the concrete compressive strength showed that using UHPC instead of NSC result in an increase in closure joint capacity.
6. The finite element model was presented for lap splice, which demonstrated the brittle failure in the splice region when insufficient length of lap splice was provided. The numerical results showed that increasing the overlap length from 4 in. to 18 in. in normal strength concrete could result in 50% increase in load capacity of the investigated beam.
7. Moreover, the numerical results showed that increasing the overlap length from 4 in. to 18 in. for different concrete strengths could result in an increase in displacement ductility of the investigated specimens.

4.2 Future Study

This dissertation provides a comprehensive experimental and numerical study of new closure joint detail used for connecting prefabricated deck elements. New closure joint is detailed and designed using 90° hooked bars to provide satisfactory structural performance while is durable. The finite element analysis generally gives a very good estimate of structural behavior of the whole system.

The experimental phase of this research was limited to two bar sizes i.e. No. 4 and No.6 which represents the upper and lower bounds of reinforcing steel typically used in the bridge decks. Supplement testing is required for different bar sizes as bond mechanism of different bars may not be linearly extrapolated. In addition, cyclic testing of closure joints needs to be experimentally evaluated.

Another area of interest which requires further experimental testing is finding minimum lap splice length for the epoxy coated bars. This task may focus on several parameters such as bar size and type, clear cover etc. There is a need to test the new closure joint at full scale to understand the field implementation, constructability and monitoring for structural performance and durability under service conditions.

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