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Partial continuity in prestressed concrete girder bridges with jointless decks

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PARTIAL CONTINUITY IN PRESTRESSED CONCRETE GIRDER BRIDGES WITH JOINTLESS DECKS

A Thesis
Submitted to the Graduate Faculty of the Louisiana State University and Agricultural and Mechanical College in partial fulfillment of the requirements for the degree of Master of Science in Civil Engineering in The Department of Civil and Environmental Engineering

By
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ABSTRACT

Jointless multi-simple span deck-girder (composite concrete deck with steel or prestressed concrete girders) bridge construction has been accepted as an alternative to jointed construction. Bridge deck joints add to the construction and maintenance costs of the bridge. Bridge support bearing pads deterioration is a common problem in bridges with jointed bridge decks. One of the methods used for building a jointless bridge is to provide a link slab that connects the decks of the adjacent spans. This study mainly focuses on the behavior of the link slab and its effect on the behavior of the bridge system as a whole. The scope of the study is to develop FE models to analyze the variation of forces, stresses and moments in the link slab as well as the level of continuity generated in the girder system. The analysis is carried out for different bridge parameters which are likely to affect the behavior of link slab; namely, bearing stiffness, skew angle, span lengths and debonding length ratio of link slab.

The present study helps in understanding the effects of the aforementioned factors on the behavior of the link slab and the system. The study also proposes development of a modified three moment equation for different parameters. The parameters which influence the three moment expression are the bearing stiffnesses, material properties and geometric information. A thorough parametric study is required to validate the expression. The results can be used for development of a design procedure for the link slab and the expression can be used for analysis of the link slab.

The results obtained show that the link slab behaves more like a tensile member rather than a bending member with the increase in bearing stiffness and debonding length ratios. This observation was consistent in all the bridge types and skew angles considered for the study.
CHAPTER 1. INTRODUCTION

1.1 General Background

Jointless multi-simple span deck-girder (composite concrete deck with steel or prestressed concrete girders) bridge construction has been accepted as an alternative to jointed construction. There are many reasons behind the popularity of the multi-simple span bridge. First, continuous span construction adds to the complexity of the system and hence the design. Second, the construction of continuous spans is also more involved and requires special details. Finally and most importantly, bridge joints relieve secondary effects such as those caused by thermal expansion, shrinkage and creep.

Bridge deck joints are nowadays becoming a major concern to bridge owners and officials. They add to the construction and maintenance costs of the bridge. Deck joints also allow water and debris to accumulate through the gaps. Water leaking through the deck joints will cause the deterioration of the bearing pads and other structural components. Debris accumulating in the deck joints will hinder the proper function of the deck joint, which may lead to structural damage if not maintained properly.

Because of the aforementioned reasons, multi-span bridges with no or minimum deck joints are receiving more attention from bridge owners and officials. The installation and maintenance costs would be greatly reduced if the number of deck joints in a multi-span bridge is minimized. An approach to minimize the deck joints is to construct a jointless bridge. Integral bridges are one of the jointless bridges alternatives. The term “integral bridge” generally refers to continuous jointless bridges with single and multiple spans that are integrally cast with capped-pile stub-type abutments.

A jointless bridge is constructed by making the girders and the deck slab continuous. The cost of connecting precast prestressed concrete or steel girders to convert the simply supported
system into a continuous system can be eliminated by constructing continuous deck slabs while keeping the girders simply supported. The latter system is often referred to as a “jointless deck” bridge. It has been used in the construction of new bridges and may be used in retrofitting existing bridges during a deck replacement project.

Simplified procedures for the design of bridges after the removal of expansion joints using partially debonded, continuous decks has been proposed by Richardson (1989). The section of the deck connecting the two adjacent spans is termed “link slab” (El-Safty 1994). Okeil and El-Safty (2005) showed that the forces in the link slab and moments in girders are influenced by the support conditions. A schematic of a conventional concrete link slab can be seen in Figure 1.1 for a steel girder bridge.

![Figure 1.1 Conventional Concrete Link Slab](image)

Idealized bridge models generally rely on two types of support conditions; namely hinged (H) and roller (R). For a two-span bridge, the possible combinations of support conditions are HRRH, RHHR, RRRR, HHHH, and RHRH.
Usually, the precast prestressed concrete girders are supported on elastomeric bearing pads, which are widely used because they are economical, efficient and maintenance free (Roeder et al 1987). A hinge or roller idealization does not truly represent the properties of bearing pads, whose restraining effects depend on its stiffness.

This research mainly aims at developing an analytical model for a two-span prestressed concrete girder bridge to be used in studying the behavior of bridges with jointless decks. The analytical model is developed in a general purpose finite element (FE) software package (ANSYS 5.6). The support condition is modeled to produce the same effect like a realistic support and not as simplified idealization. The bridge models are analyzed for the load case that produces the maximum negative moment and hence the maximum tension force in the link slab. The applied load follow the static portion of HL-93 loading model (AASHTO 2004), and does not cover the effects of other loads (e.g. dead loads, wind loads, and long term effects).

Many researchers proposed different methods for the design of link slabs. Most of these methods rely on ideal support conditions (e.g. hinged, roller). In this study, the behavior of jointless bridges with realistic support conditions is investigated for skewed and non-skewed bridge layouts. The behavior of the link slab is further investigated for the debonding length ratio of the links slab and also for three different girder types with same girder spacing. Understanding the behavior is further used to develop closed-form design expressions.

1.2 Objectives

The main objectives of this research are (1) to understand the behavior of skewed and non-skewed bridges with jointless decks, and (2) to develop simplified analysis tools based on closed-form derivations.
1.3 Scope of Study

The study investigates the behavior of bridges with jointless decks for different support conditions (bearing pad stiffnesses) and bridge skew angles. It mainly focuses on the behavior of the link slab and partial continuity generated in the girder system that is generated by linking the decks from adjacent spans.

1.4 Organization

This thesis is organized in six chapters.

Chapter 1 is a general introduction of the concepts of integral bridges, jointless bridges, link slabs and bearing pads. The scope and organization of the research project is also given.

Chapter 2 presents a thorough literature review in the fields of integral bridges, jointless bridges and link slabs. The material properties used for the construction of the link slab, as well as its advantages and disadvantages are also given. An overview of past studies of the bearing pads and properties of materials used in a bearing pad are briefly discussed.

The details of the finite element model for a jointless two-span bridge with link slab and variable support stiffnesses are presented in the Chapter 3. Also in this chapter models with different skew angles are presented. Materials properties, attributes assigned to different element types, and mesh generation are described. The procedure followed to model the bearing pads is also given. Furthermore, load application on the bridge is also described. Finally, the analytical verification of the developed model using experimental data from the literature is presented.

In Chapter 4, the analytical results obtained from different FE models are presented. The results presented include bridges with different support stiffnesses, skewed and non-skewed layouts. A detailed discussion of the results is presented.
The classical three-moment equation is discussed along with the modified three-moment equation for partial continuity induced in the system due to the introduction of link slabs is given in Chapter 5.

In Chapter 6, conclusions drawn from the research and recommendations for future research are presented.
CHAPTER 2. LITERATURE REVIEW

2.1 Introduction

Jointless deck construction is now days adopted more often than before as it reduces the construction cost and subsequent maintenance costs associated with bridge joints. Jointed deck construction was often preferred due to the lack of understanding of the jointless system, which requires advanced computational resources for its analysis and design. Furthermore, engineers design the jointed deck system to relieve the secondary forces that develop due to thermal effects, creep, shrinkage etc. The difficulties in having a jointed deck system are explained by many researchers in the literature. Consequently, the jointless deck system started gaining more attention. In this chapter, a review of published literature is presented. It first covers problems associated with the use of bridge joints, an overview of integral bridges, jointless bridges, link slab and bearing pads is also presented. Finally, standard DOT connections details from several states including Louisiana are presented.

2.2 Problems Associated with Expansion Joints

The drawback of having a bridge deck joint has been extensively studied. Investigations on the movements, performance and dynamic behavior of the bridge deck joints were carried out. Pentas et al (1995) studied the longitudinal movements in composite bridges. They carried out extensive investigation on the east approach of US-190 over the Atchafalaya River at Krotz Springs, Louisiana. They described the experimental procedures of instrumentation and monitoring and discussed the general behavioral characteristics of the bridge with respect to long-term movements and also bridge joint movements. They concluded that the primary causes of movements in the bridge decks obtained during the monitoring periods were due to the thermal effects. The movements of concrete-to-concrete girder joints were twice that of the expansion joints at steel-to-concrete girder locations. The results of the experimental study
revealed the presence of restraining effect at the expansion joint support. They also observed that the stresses developed at the neoprene bearing pads as a result of thermal expansion was suddenly relieved when a certain stress level was reached or when an external force was applied. The unsymmetrical joint movements experienced by the bridge sections were due to the restraints associated with the neoprene bearing pads. They also noted that the bridge underwent nonreversible joint movements. The bridge sections showed no signs of rigid body translation.

Chang et al (2002) discussed the deterioration indicators and causes of failure of bridge deck expansion joints. Furthermore, the common joint defects and factors influencing the joint performance were also discussed. The study was to investigate the performance of several types of joints through questionnaire surveys, analysis of INDOT (Indiana Department of Transportation) roadway management data and expert interviews. They concluded that most joint problems result from cracks in the seal and failure of bonding agents. The common joint problems like holes in seals, hardened, cracked, loose, or torn seals, and spalled cracked concrete are caused due to traffic loading, snowplow damage, weather, poor installation, inferior materials, and incorrect selection of the joint type.

Ancich et al (2006) studied the dynamic anomalies in a modular bridge expansion joint (MBEJ). The investigation identified modal vibration frequencies in the MBEJ coupling with acoustic resonances in the chamber cast into the bridge abutment below the MBEJ. The paper introduces dynamic range factor (DRF), which is a ratio of peak-to-peak dynamic response and quasi-static response (i.e., a measure of the reinforced response due to multiple wheel/beam impacts). They concluded that the joint is very lightly damped and the lowest frequency mode excited was a quasi-rigid body mode at 71 Hz. They observed that the support bars and center beams were acting dynamically as if simply supported and also the shape analysis studies (i.e. a
measurement and analysis technique that enables the dynamic response or deflection shape at particular frequencies of interest to be defined) showed good agreement between the experimental modal analysis (measurement and definition of the natural frequencies and mode shapes of a structure).

2.3 Jointless Bridge Alternatives

Several researchers proposed solutions for eliminating bridge joints. They may be categorized under two main types, namely Integral Bridges and Jointless Deck Bridges. The following is a review of information available in literature about these two alternatives. Different types of continuous bridge systems are shown in the Figure 2.1.

2.3.1 Integral Bridges

A wide range of studies were published about integral bridges. Detailed discussion about the attributes and limitations of the integral bridges is presented by Burke (1993a). The author states that the integral bridges have numerous attributes with few limitations. The attributes are simplicity in design, jointless construction, pressure resistance, rapid construction, ease of embankment construction, no or few cofferdams, small excavations, single horizontal arrangement of vertical piles, few joints, simple forms, simple beam seats, elimination of anchor bars, broad-span ratios, earthquake resistance, simplified widening and replacement and improvement in live load distribution.

The limitations are high abutment-pile stresses, because of continuous spans most of the integral bridges are subjected to uplift when bridge is submerged, additional buoyancy construction procedures must be followed and deck slab placement on integral bridges with short end spans must be controlled to eliminate uplift of beams during concrete placement in deck slab construction. Design provisions can be made to account for some of these limitations.
Burke (1993b) briefly discussed the design of integral bridges. The discussion focuses mainly on secondary effects, simplifying assumptions, design comments and suggestions. The design comments, while designing the super structure for usual primary and secondary stresses in
the design of continuity connections, provision must be made to protect the structure from effects of buoyancy and snowplows, and also while for the substructure design.

Figure 2.2 Details of Integral Bridge

Further, Burke (1994) discussed the movements and forces in semi-integral bridges. The superstructure restraints like the longitudinal, lateral and vertical restraints were also discussed along with the design aspects and construction aspects. The applicability of the semi-integral bridges concept becomes more important for those applications where the rigid abutments are necessary.

Siros et al (1995) compared the two major forms of highway bridges, integral (jointless) versus the jointed. They investigated the creep stresses developed in an integral (jointless) and jointed bridges. They observed that creep causes a small increase in positive and negative stresses at the bottom of the steel stringers and a reduction in tensile and compressive stresses at the top of the concrete deck.

Alampalli et al (1998) inspected 84 integral bridges and 105 jointless bridges in New York. They concluded that integral and jointless bridges have been functioning as designed and showed superior performance compared with conventional bridges.
2.3.2 Jointless Bridges

Loveall (1985) presented an overview of jointless bridge decks over continuous girders in Tennessee DOT. Tennessee policy reads, the bridges constructed must be continuous from end to end with no intermediate joints except for cold joints required for construction and applies to both longitudinal and transverse joints.

Pierce (1991) in his case study asserts that ridding an existing bridge of expansion joint does not require sophisticated analysis and design techniques. The installation sequence for A.L. Blades and Sons of Hornell, N.Y. is presented.

Russell et al (1994) discussed the knowns and unknowns of the jointless bridges. They describe the real behavior of jointless bridge is extremely complex because of development of secondary forces caused by temperature effects, creep and shrinkage. Despite these complexities, jointless bridges perform satisfactorily.

Thippeswamy et al (1995) proposed state-of-the-art methods to analyze the jointless bridges for primary and secondary loads. They considered five in-service jointless bridges for their analysis.

In another research effort, Thippeswamy et al (2002) proposed design alternations in a jointless bridge based on conclusions drawn from the experimental and analytical results.

Wasserman (2005) briefly discuss about the simplified continuity details for short and medium-span composite steel girder bridges in the state of Tennessee. He also presents the link slab details, continuity for composite load details and continuity for dead load slab and composite load details.
2.4 Link Slab Details

2.4.1 Introduction

The section of the deck slab connecting the two adjacent simple-span girders is after referred to as the “link slab”. El-Safty (1994) developed a detailed analytical model to analyze a jointless deck on two-span steel girders. The model was based on the finite element method and accounted for forces and moments in link slab.

2.4.2 Previous Studies on Link Slabs

Caner et al (1998) conducted tests on specimens to investigate the behavior of link slabs connecting two adjacent simple-span girders, and proposed a simple design method for designing the link slab. They also illustrated three design examples in the paper. They concluded that the link slab was in bending and behaved like a beam rather than a tension member. While testing a steel bridge, they observed that when the load reached 45kips (200.2 kN), crushing of concrete was observed at the bottom portion of the link slab, indicating the failure of link slab. In the case of concrete bridge they tested it for three support configurations namely HRRH, RHRH and RHHR. The tensile strains developed in the #6 epoxy coated bars in the link slab were again similar for the three test cases. They also observed that with a load of 37.5 kips (166.8), the reinforcement in the link slab reached its yield strength. They concluded that within the elastic range, the measured deflections, the strains in the girders, the strains in the link slab reinforcement were not affected by the variations of support conditions at exterior and interior supports.

Caner et al (2002) investigated the use of link slabs as a seismic retrofit method for existing multi-simple-span highway overpasses. They carried out a case study on a 90-m long bridge with four simply supported spans with two traffic lanes. The bridge girders were AASHTO Type III that was supported on identical elastomeric bearing pads at all ends. They
concluded that the connection provided by the link slabs can eliminate the span separation problem and potential damage due to unseating. The proposed design method can be used for the preliminary design of link slabs.

Kim et al (2004) studied the performance of bridge deck link slabs designed with ductile engineered cementitious composite. Engineered cementitious composite (ECC) is a high performance, fiber-reinforced cementitious composite designed to resist tensile and shear force while retaining compatibility with normal concrete in almost all respects. They recommend the debonding of link slab over the girder joint for a length equal to 5% of each girder span, which produces reduction of stiffness in the link slab.

Okeil and ElSafty (2005) in their paper proposed a simplified method for the analysis of bridges with jointless decks. In the paper they derived an expression to calculate the tension force in the link slab. A parametric study is conducted for the case of a two-span bridge. They concluded that the tension force that develops in the link slab is affected by the systems’ idealized support configuration. They also derived expressions for the tension force and continuity moment. They concluded the hinged supports at the considered joint to lead a higher continuity moment and tension force in link slab.

Wing and kowalsky (2005) studied the performance of first jointless link slab bridge in North Carolina through the use of remote instrumentation and a variety of testing and analysis methods. They concluded that the link slab within a bridge are subjected to low rotations due to traffic loads, while thermal effects induced greater rotations. They also observed that the size of the crack in link slab exceeded the design criteria. Finally, they proposed a limit- state design approach where reinforcement is sized based on a rotational demand and crack width.
2.4.3 Properties of Materials

Generally, deck slabs are constructed using reinforced cement concrete (RCC). Link slabs are also cast using RCC, but as link slab carries huge tensile stresses compared to other structural elements, studies relating to the materials that can used for link slab have been carried out.

Kim et al (2004) proposed Engineered Cementitious Composite (ECC) materials which would help reduce the crack width and to increase the ductile nature of the link slab which are the most important requirements in terms of performance. A minimum concrete compressive strength of 35 MPa was considered. As per the AASHTO the maximum allowable crack width is 330μm in RCC bridge deck in severe exposure condition. The ECC material considered for the tests contained 2% by volume of PVA fibers, ordinary Portland cement, fine aggregates, Type F fly ash, water, and a water reducing admixture. At 28 days material properties obtained for the specimen first crack strength of 4.0 MPa at 0.02% strain, ultimate tensile strength of 6.0 MPa at 3.7% strain and the average crack width was observed to be 40μm.

The conclusions drawn from this test is that the ECC link slabs are more durable because of the improved cracking characteristics as compared to conventional RC link slabs. The amount reinforcing steel required decreased with the use of ECC material thereby reducing the stiffness of the link slab.

2.4.4 Advantages and Disadvantages

The advantages of providing link slab are numerous. They include reduction of costs associated with the construction and maintenance of bridge deck joints, Elimination of structural damage caused by improper maintenance of the bridge deck joints, Corrosion damage caused to support bearing pads and other structural elements due to water leaking through the deck joints is reduced, and Reduction in vibrations and noise produced when vehicles pass over the bridge
deck joints, thus enhancing the riding quality (Billing 1983). The aesthetics and riding quality of the bridge is also enhanced. Due to continuity of the bridge spans, the girders live load mid-span moment is reduced, thus increases the load carrying capacity of the girders.

The disadvantages associate by providing link slabs are continuity achieved due to jointless bridge deck induces secondary stresses (thermal stresses, creep, shrinkage etc.) can cause structural damages. High stresses produced due to cyclic load will lead to structural cracks and fractures (Burke 2004), if bridge is not designed properly. Proper design and construction practices must be followed if the bridge deck joints are made continuous after removing the existing joints; otherwise due to the post-tensioning the joints may give up.

2.5 Bearing Pads

2.5.1 Introduction

As can be seen from the literature, the support conditions play an important role while analyzing a link slab for tension force and moment. Most common types of supports used are the elastomeric bearing pads. The different types of bearing pads and properties associated are presented in this section.

2.5.2 Types of Bearing Pads

AASHTO specifies different types of bearing pads, namely, plain elastomer type; fiber reinforced laminated type and steel-reinforced laminated type. Plain Elastomer bearing pads manufactured by (1) molding elastomeric compound to desired size or (2) cutting from previously molded elastomeric compound, or (3) extruding the pad for a specific length, with smooth surfaces and cut edges. Fabric-Reinforced laminated type are laminated pads consisting of alternate layers of elastomeric compound and glass fabric reinforcement bonded together, with top and bottom layers of reinforcement uniformly covered with 1/8 inch of elastomer. Steel-reinforced laminated bearing pads are laminated pads consisting of alternating steel laminates
and internal elastomer laminates bonded together, with top and bottom layers of steel reinforcement uniformly covered with \( \frac{1}{4} \) inch of elastomer. Exposed sides shall be covered with 1/8 inch of elastomer (AASHTO M251).

2.5.3 Properties of Bearing Pads

The Shear Modulus, \( G \), is the most important design parameter of the bearing pads as suggested by the AASHTO. The other property is the hardness of the elastomer used for designing bearing pads. Depending on the hardness values AASHTO provides range of corresponding shear modulus values. Elastomers used in the bearing pads shall have a shear modulus of 0.655 – 1.379 MPa and a nominal hardness grade of 50 and 60 on Shore A Scale at 23\(^{\circ}\) C (AASHTO 1996a). Elastomers used in bearing pads as per FDOT specifications shall have a hardness grade 50-durometer hardness with a shear modulus range of 0.655-0.896 MPa. Reoder et al. (1987) showed that decrease in temperature would increase the hardness and shear modulus of the elastomer. The shape factor and shear modulus effect the strength of the bearing pads. AASHTO specifies an equation to calculate the shape factor for the bearing pads.

Table 2.1 Physical Parameters for FDOT Bearing Pads

<table>
<thead>
<tr>
<th>Parameter</th>
<th>II/II</th>
<th>III/III</th>
<th>IV/IV</th>
<th>V/V, VI and Florida bulb Tee</th>
</tr>
</thead>
<tbody>
<tr>
<td>Length (mm)</td>
<td>204</td>
<td>178</td>
<td>230</td>
<td>254</td>
</tr>
<tr>
<td>Width (mm)</td>
<td>356</td>
<td>458</td>
<td>458</td>
<td>610</td>
</tr>
<tr>
<td>Area (mm(^2))</td>
<td>72625</td>
<td>81524</td>
<td>105350</td>
<td>154940</td>
</tr>
<tr>
<td>Elastomer thickness (mm)</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Inner layers (2)</td>
<td>8.75</td>
<td>7.75</td>
<td>10.75</td>
<td>12.75</td>
</tr>
<tr>
<td>Outer layers (2)</td>
<td>6.00</td>
<td>6.00</td>
<td>6.00</td>
<td>6.00</td>
</tr>
<tr>
<td>Total</td>
<td>29.50</td>
<td>27.50</td>
<td>33.50</td>
<td>37.50</td>
</tr>
<tr>
<td>Shape Factor</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Inner layers</td>
<td>7.4</td>
<td>8.3</td>
<td>7.1</td>
<td>7.0</td>
</tr>
<tr>
<td>Outer layers</td>
<td>10.8</td>
<td>10.7</td>
<td>12.8</td>
<td>14.9</td>
</tr>
<tr>
<td>Weighted shape factor for pad</td>
<td>8.8</td>
<td>9.3</td>
<td>9.1</td>
<td>9.6</td>
</tr>
</tbody>
</table>
Physical properties of bearing pads are tabulated in Table 2.1. The table illustrates the physical dimensions like length, width, and shape factors for different types of FDOT bearing pads. AASHTO (1996a) specifies a simplified expression for effective compressive modulus of elasticity. Effective compressive modulus of elasticity is influenced by the shear modulus and shape factor of the bearing pad.

2.5.4 Review of Past Studies

Many studies on different aspects of elastomeric bearings can be found in the literature. The performance of elastomeric bearings was studied by Roeder et al (1987). This study also addresses the effect of low temperatures on the behavior and acceptance criteria for elastomeric bridge bearings.

Sen et al (1994) investigated the restraining effect of bearings in a study sponsored by the Florida department of transportation. They concluded that the load distribution characteristics of the test bridge were accurately represented by AASHTO. They also confirmed the restraining effect at the bearings experimentally.

2.6 Standard DOT Bridge Joint Details

The Department of Transportation (DOT) in every state of United States formulated own joint connection details for the highway bridges. Diaphragms are transverse members that connect longitudinal connections for bridge girders. There are different types of diaphragms based on their location in the bridge, namely, end diaphragms, intermediate diaphragms and continuity diaphragms.

The present study deals only with the continuity of the multi-simple span bridges, and hence will only be briefly reviewing of the continuity diaphragm details and design procedures followed by several state DOT. Continuity diaphragms details for different states are presented based on the similarity in the design and construction techniques into four different categories.
i.e. I, II, III & IV and only seven states details are presented here. Each category is unique for its details and design followed by the state DOTs.

Category I, the diaphragm under category is constructed over pier to resist live load and superimposed dead load. The positive moment in these bridges are resisted by the dowels or strands extending into the diaphragm. Continuity diaphragm design details for the states of Idaho, Illinois and Pennsylvania fall under this category. These details are presented in Appendix A.

Category II, the continuity diaphragms under this category are constructed without positive moment reinforcement shown. The dowels are sheathed with a rigid sleeve with a compressible cap above top of the dowel to allow girders to deflect on their bearing pads under future loads. Continuity diaphragm design details for the states of Louisiana and Virginia categorized under this category. These details are presented in Appendix A.

Category III, the continuity details of the deck slab are provided in this category. It can be seen from the continuity diaphragm is split into two diaphragms that are not continuous over piers. The section details for the diaphragm are presented. Georgia state DOT details for diaphragm are categorized in this category. The details are presented in Appendix A.

Category IV, the design details are different in that the diaphragm is used to develop an interlocking mechanism in the transverse direction with the risers. The state of Massachusetts adopts such a detail as can be seen in Appendix A (see Fig A4.1) continuity reinforcement is not provided in that detail. It should be noted that the detail in Figures A.1.3 and A.1.4 also show a similar mechanism.
CHAPTER 3. ANALYTICAL STUDY

3.1 Introduction

In this chapter, finite element (FE) models used for investigating the behavior of bridges with link slabs are presented. The FE models are developed using software package (ANSYS 5.6). Developing three-dimensional (3-D) models instead of 2-D models was deemed necessary in order to be able to capture the behavioral aspects targeted in this study. The model was first verified by comparing the results to experimental data in the literature. It was then used in a parametric study where various bridge characteristics were changed to investigate their effects on the behavior. The parametric study included one hundred and sixty nine models to study the behavior.

3.2 Development of Bridge Model

AASHTO II, III and IV girder types are considered for modeling. The span lengths for each type of girder were determined based on acceptable design practices according to LA-DOTD Bridge Design Manual (AASHTO 1996a) for assumed girder spacing. The gap between two adjacent spans was taken as 200 mm. All bridge models were for the same roadway width and had the girder spacing same. FE models for two-span skewed (30°, 45°) and non-skewed (0°) for each girder type with link slab are generated. Three link slab debonding lengths were considered in the study. They represented a debonded length at the central support equal to 0.0%, 2.5% and 5.0% on each side of the support. Each bridge model is designated as “Bridge Girder Type– Skew angle-Debonding Ratio”, i.e. “Bridge II-S30-D50”, implies that is it is a bridge with Type II AASHTO girder, 30° skew angle, and 5.0% debonding length ratio for the model. Table 3.1 summerizes the variation of parameters considered for bridge models.
Table 3.1 Physical Parameters influencing Bridge Models

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Girder Type</th>
<th>II</th>
<th>III</th>
<th>III</th>
</tr>
</thead>
<tbody>
<tr>
<td>Skew Angle</td>
<td>0°</td>
<td>30°</td>
<td>45°</td>
<td></td>
</tr>
<tr>
<td>Debonding Ratio</td>
<td>0.0%</td>
<td>2.5%</td>
<td>5.0%</td>
<td></td>
</tr>
</tbody>
</table>

The bridge deck and girders are meshed using SOLID65 in ANSYS 5.6. Girder and deck material properties (Modulus of Elasticity and Poisson Ratio) were assigned to their respective elements in the mesh. The supports were modeled as spring element using COMBIN14 element for which properties of stiffness can be set to represent an actual bearing pad. The bridge is restrained at supports for different stiffnesses. A standard truck load (HS-20) was applied to the bridge to produce maximum tensile force in link slab and maximum negative moment in girders. The applied loads are proportionally distributed to the corresponding nodes of the elements where the load actually acts. In this section, a brief introduction to ANSYS software package, elements used for modeling, support configurations, modeling of bearing pads and link slab, verification of the model and overview of the bridge model etc are presented.

The analyses were carried out to study the tension force and moment in link slab, moments and reactions in girders in the bridge. The study is to observe the change in the moments in girders and tensile forces produced in the link slab with change in the support configuration. Seven types of configurations are considered for the analytical study.

### 3.2.1 Bridge Categories

In the present study, the bridge types are categorized into three main categories. The categorization is based on the AASHTO girder types and span lengths. The girders vary in geometric properties, and hence in their load carrying capacity. Therefore, the span lengths were determined based on each girder type following LA-DOTD recommendations (AASHTO 1996a). The details of each bridge category are presented in Table 3.1.
3.2.2 Elements Used in this Study

The two types of elements used for the development of bridge models for the present study are SOLID65 and COMBIN14 from ANSYS element library. The SOLID65 element was used to model concrete portions of the bridge along with the steel reinforcement present in the concrete, while COMBIN14 element was used to model the supports and they represent the properties of bearing pads. An overview of the elements is presented next.

<table>
<thead>
<tr>
<th>Type II</th>
<th>Type III</th>
<th>Type IV</th>
</tr>
</thead>
<tbody>
<tr>
<td>Girder Type (AASHTO)</td>
<td>II</td>
<td>III</td>
</tr>
<tr>
<td>Span Length (m)</td>
<td>16.05</td>
<td>20.10</td>
</tr>
<tr>
<td>Width (m)</td>
<td>13.03</td>
<td>13.03</td>
</tr>
<tr>
<td>Spacing of Girders (mm)</td>
<td>2640</td>
<td>2640</td>
</tr>
<tr>
<td>Deck thickness (mm)</td>
<td>178</td>
<td>178</td>
</tr>
</tbody>
</table>

SOLID65 is a 3-D solid element which allows for the presence of four materials within each element. In this study, concrete material properties were assigned to the element and only two of the three allowed embedded reinforcements were used; namely transverse and longitudinal.

The element is defined by eight nodes having three degrees of freedom at each node with translations in the nodal x, y, and z directions. The geometry of the SOLID65 element can be seen in Figure 3.1. Rebars in the element has tension and compression capability. Rebars are modeled using a “smeared” technique. The sum of the volume ratios for all rebar must not be greater than 1.0 in all three directions.

The input data required to model a solid element are the modulus of elasticity and Poisson’s ratio. The reinforcement data is entered as real constants: modulus of elasticity, Poisson’s ratio, volume ratio and orientation angles. The volume ratio is defined as the rebar
volume divided by the total element volume. The orientation is defined by two angles (in degrees) from the element coordinate system.

Figure 3.1 SOLID65 Geometry

The output data from the analysis of SOLID65 element that are of interest for this study are nodal deformations, forces and also element stresses and strains.

COMBIN14 element in ANSYS is a spring damping element that has longitudinal and torsional capabilities for 1-D, 2-D and 3-D applications. The longitudinal spring damper has the capability to take uniaxial tension or compression and upto three degrees of freedom at each node. The spring damper has no mass associated with it. The torsional capabilities of the element are not described as it is irrelevant for the present study. Similarly, the damping aspect of the element was not utilized as this study was limited to static loading cases.

The geometry of the COMBIN14 element for both longitudinal and torsional cases can be seen in Figure 3.2. The element is defined by two nodes and, a stiffness coefficient (k) and damping coefficients (cv1) & (cv2). In the present study COMBIN14 element is considered as spring element with two nodes and stiffness (k).
The outputs produced after analysis for this element are the nodal forces, displacements in all the three directions depending on the degrees of freedom.

![Figure 3.2 CONBIN14 Geometry](image)

### 3.2.3 Material Properties

The material properties needed for the 3-D bridge models are: (1) Concrete (Poisson’s ratio and Modulus of elasticity) for both girders and decks, and (2) steel reinforcement (Poisson’s ratio and Modulus of elasticity). Table 3.3 lists the values used in this study.

The Shear Modulus, $G$, is the most important property for the bearing pad design as specified by AASHTO (1996a). Hardness is also another property used by the engineers to specify bearing pads. AASHTO specifies that at $23^\circ$C the elastomer used in the bearing pads shall have a shear modulus of 0.655 to 1.379 MPa and a nominal hardness grade between 50 and 60. The stress strain characteristics of the elastomers are controlled by the shear modulus and shape factor. The shape factor for one elastomer is given by AASHTO (1996a).

$$S = \frac{LW}{2h_s(L+W)}$$  \hspace{1cm} (3.1)
### Table 3.3 Material Properties used for Bridge Model (Yazdani et al 2000)

<table>
<thead>
<tr>
<th>Properties</th>
<th>AASHTO</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Type II</td>
</tr>
<tr>
<td><strong>Concrete</strong></td>
<td></td>
</tr>
<tr>
<td>Poisson’s ratio</td>
<td>0.18</td>
</tr>
<tr>
<td>Modulus of Elasticity $E$ (MPa)</td>
<td></td>
</tr>
<tr>
<td>Girder</td>
<td>$2.78 \times 10^4$</td>
</tr>
<tr>
<td>Deck</td>
<td>$2.23 \times 10^4$</td>
</tr>
<tr>
<td><strong>Steel</strong></td>
<td></td>
</tr>
<tr>
<td>Poisson’s ratio</td>
<td>0.29</td>
</tr>
<tr>
<td>Modulus of Elasticity $E$ (MPa)</td>
<td></td>
</tr>
<tr>
<td>Flexural reinforcement</td>
<td>$1.90 \times 10^5$</td>
</tr>
<tr>
<td>Slab rebars</td>
<td>$2.00 \times 10^5$</td>
</tr>
</tbody>
</table>

Where $S$ is the shape factor from a layer of an elastomeric bearing; $L$ is the dimension of the bearing parallel to the longitudinal axis; $W$ is the dimension of the bearing pad normal to the beam axis; and $h_i$ is the thickness of single elastomer layer. The physical properties along with shape factors are summarized in Table 2.1. AASHTO specifies equations for calculating the effective compressive modulus of elasticity and also included the restraints of bulging; the expressions are given by the following equations.

$$E_c = 3G(1 + kS^2)$$  \hspace{1cm} (3.2)

$$E_c = 6GS^2$$  \hspace{1cm} (3.3)

Yazdani et al (2000) has validated the AASHTO moduli for the bearing pad types experimentally. An equivalent bearing stiffness was calculated for each of the developed models. The stiffnesses obtained were compared with the AASHTO predicted values. The summary of the stiffnesses are presented in Table 3.3. The neoprene shear modulus of FDOT Type II bearing pads was increased 50 times, designated as II* in Table 3.4, and it is observed that the increase in the shear stiffness is about 41 times of Type II model. The inference drawn from the observation is that the shear stiffness of the bearing pads has linear relationship to the shear modulus of neoprene, whereas the vertical and bending stiffness did not show linear relationship.
Table 3.4 Stiffness of FDOT Bearing Pads Based on FE Modeling (Yazdani et al 2000)

<table>
<thead>
<tr>
<th>Stiffness (kN/mm)</th>
<th>FDOT Bearing Type</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>II</td>
</tr>
<tr>
<td>$k_x$</td>
<td>622</td>
</tr>
<tr>
<td>$k_y$</td>
<td>1.87</td>
</tr>
<tr>
<td>$k_z$</td>
<td>1.87</td>
</tr>
<tr>
<td>$k_{Rx}$</td>
<td>40.7</td>
</tr>
<tr>
<td>$k_{Ry}$</td>
<td>2.16 x 10^6</td>
</tr>
<tr>
<td>$k_{Rz}$</td>
<td>6.58 x 10^6</td>
</tr>
</tbody>
</table>

Note: For type II*, shear modulus $G^* = 50G$

Table 3.5 Stiffness of FDOT Bearing Pads (Based on AASHTO 1996) Standard Specifications

<table>
<thead>
<tr>
<th>Stiffness and other parameters</th>
<th>Bearing Type</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>II</td>
</tr>
<tr>
<td>$I_y$ (mm$^4$)</td>
<td>2.52 x 10^8</td>
</tr>
<tr>
<td>$G$ (MPa)</td>
<td>0.76</td>
</tr>
<tr>
<td>$E_c$ (MPa)</td>
<td>267</td>
</tr>
<tr>
<td>$k_x$ (kN/mm)</td>
<td>656</td>
</tr>
<tr>
<td>$k_z$ (kN/mm)</td>
<td>1.87</td>
</tr>
<tr>
<td>$k_{Ry}$ (kN/mm)</td>
<td>2.28 x 10^6</td>
</tr>
</tbody>
</table>

A summary of FDOT bearing pad Stiffnesses as specified by AASHTO are given in the Table 3.5. The effective modulus is calculated based on the Equation 3.2. AASHTO provisions allow determination of only the three stiffnesses shown in Table 3.5. It is observed that the vertical and bending stiffnesses based on AASHTO specifications are higher than that of FE predicted values. This can be attributed to the non linear behavior of the neoprene materials considered in the AASHTO formulas (AASHTO 1996a).

The vertical (compressive) and shear (horizontal) stiffness are considered for the finite modeling of the bearing pads. The other stiffnesses are considered to be negligible or ineffective for the present study.
3.2.4 Idealized Support Conditions

The support conditions depend on the properties of the bearing pad material used. An overview of the properties of bearing pads is presented in the previous sub section. These are HRRH, RHRH, RHHR, HHHH and RRRR, where H stands for hinged (pinned) support and R stands for roller supports. RRRR configuration is not possible because of structural instability due to lack of longitudinal restraints. Different possible support configurations are shown in Figure 3.3.

It should be noted that these idealized conditions imply that the stiffness of the bearing pads is infinite in the vertical direction. Laterally, an H-support implies infinite stiffness while an R-support implies zero stiffness.

Based on the previous section, it is clear that the actual stiffnesses are different then those idealizations. Therefore, detailed modeling of the pads was necessary for investigating the behavior of the system as described in the next section.

3.2.5 Modeling of Bearing Pads

Modeling of the bearing pads in a way that reflects its actual stiffness was deemed necessary for this study. Following the work by Yazdani et al (2000), bearing pads were modeled as 20 spring elements with calculated fractional stiffness coefficients. The spring element considered for modeling is COMBIN14. Out of the 20 spring elements considered for modeling a bearing pad, 15 elements are modeled as vertical spring elements and 5 elements are modeled as horizontal spring elements. The vertical spring elements present the compressive stiffness and the horizontal spring elements represent the shear stiffness of the bearing pads.

The difference between the width of bottom flange of the girders and bearing pads for each type of girder is approximately 100 mm. For simplicity, the difference between the widths of the bottom flange and the bearing pads is neglected and the springs are modeled to cover the
full width of the bottom flange. The girders are meshed in such a way that the nodes generated for the elements would match exactly with dimensions required for original bearing pads in the longitudinal direction.

![Diagram](image)

**HRRH**

![Diagram](image)

**RHRH**

![Diagram](image)

**RHHR**

![Diagram](image)

**HHHH**

Figure 3.3 Configurations of Idealized Support Conditions

The mesh of the girder end zone spring elements representing the bearing pads can be seen in Figure 3.4. At each support 15 additional nodes were generated by mapping the corresponding nodes on the girder to a distance of 200mm below the girder soffit. Another 5 nodes were mapped in the horizontal direction. One end of the spring element is connected to the nodes of meshed girder elements and the other end is restrained in all degrees of freedom. The
general setup of the nodes and spring elements with end restraints are shown in the Figure 3.5. Similar procedure is followed for other cases also (skew 30, 45). It is assumed for the study that the bridge girders will displace only in the longitudinal direction and therefore the girders are restrained in the transverse direction at the supports so that any transverse movement caused by the applied load can be restricted.

The sum of the individual stiffness values of each of the spring elements adds upto the actual total stiffness of the bearing pad. The spring elements are designated as corner, edge and center for the vertical elements and outer and inner for the horizontal elements. The stiffness values of for each spring element modeled for different shear modulus of type II, III & IV bearing pads is presented in Table 3.6, 3.7 & 3.8 respectively. The torsional and bending stiffness were indirectly accounted for due to the symmetric placement of vertical spring elements along the y and z-axis of the bearing pad model.

Figure 3.4 Modeling of Bearing Pad with Spring Elements
Figure 3.5 Spring Elements (COMBIN14) used to Model Bearing Pads

Table 3.6 Spring Stiffnesses for Type II Bearing Pads

<table>
<thead>
<tr>
<th>Type of Element</th>
<th>No. of Elements</th>
<th>Fraction of Total Stiffness</th>
<th>Spring Stiffness (kN/mm)</th>
<th>G = 0.655 MPa</th>
<th>G = 0.896 MPa</th>
<th>G = 1.379 MPa</th>
<th>G = 6.89 MPa</th>
<th>G = 34.47 MPa</th>
</tr>
</thead>
<tbody>
<tr>
<td>Horizontal Elements</td>
<td>5</td>
<td>Full</td>
<td>1.55</td>
<td>2.15</td>
<td>3.30</td>
<td>16.30</td>
<td>82.71</td>
<td></td>
</tr>
<tr>
<td>Outer Springs</td>
<td>2</td>
<td>1/8</td>
<td>0.19</td>
<td>0.27</td>
<td>0.41</td>
<td>2.04</td>
<td>10</td>
<td></td>
</tr>
<tr>
<td>Inner Springs</td>
<td>3</td>
<td>1/4</td>
<td>0.39</td>
<td>0.54</td>
<td>0.83</td>
<td>4.08</td>
<td>21</td>
<td></td>
</tr>
<tr>
<td><strong>Vertical Elements</strong></td>
<td>15</td>
<td>Full</td>
<td><strong>495</strong></td>
<td><strong>650</strong></td>
<td><strong>1050</strong></td>
<td><strong>5207</strong></td>
<td><strong>25006</strong></td>
<td></td>
</tr>
<tr>
<td>Corner Springs</td>
<td>4</td>
<td>1/32</td>
<td>15</td>
<td>20</td>
<td>33</td>
<td>163</td>
<td>781</td>
<td></td>
</tr>
<tr>
<td>Edge Springs</td>
<td>8</td>
<td>1/16</td>
<td>31</td>
<td>41</td>
<td>66</td>
<td>325</td>
<td>1563</td>
<td></td>
</tr>
<tr>
<td>Center Springs</td>
<td>3</td>
<td>1/8</td>
<td>62</td>
<td>81</td>
<td>131</td>
<td>651</td>
<td>3126</td>
<td></td>
</tr>
</tbody>
</table>

Table 3.7 Spring Stiffnesses for Type III Bearing Pads (Yazdani et al 2000)

<table>
<thead>
<tr>
<th>Type of Element</th>
<th>No. of Elements</th>
<th>Fraction of Total Stiffness</th>
<th>Spring Stiffness (kN/mm)</th>
<th>G = 0.655 MPa</th>
<th>G = 0.896 MPa</th>
<th>G = 1.379 MPa</th>
<th>G = 6.89 MPa</th>
<th>G = 34.47 MPa</th>
</tr>
</thead>
<tbody>
<tr>
<td>Horizontal Elements</td>
<td>5</td>
<td>Full</td>
<td>1.93</td>
<td>2.63</td>
<td>4.05</td>
<td>20.20</td>
<td>101.56</td>
<td></td>
</tr>
<tr>
<td>Outer Springs</td>
<td>2</td>
<td>1/8</td>
<td>0.24</td>
<td>0.33</td>
<td>0.51</td>
<td>2.52</td>
<td>13</td>
<td></td>
</tr>
<tr>
<td>Inner Springs</td>
<td>3</td>
<td>1/4</td>
<td>0.48</td>
<td>0.66</td>
<td>1.01</td>
<td>5.05</td>
<td>25</td>
<td></td>
</tr>
<tr>
<td><strong>Vertical Elements</strong></td>
<td>15</td>
<td>Full</td>
<td><strong>736</strong></td>
<td><strong>1016</strong></td>
<td><strong>1576</strong></td>
<td><strong>7742</strong></td>
<td><strong>38971</strong></td>
<td></td>
</tr>
<tr>
<td>Corner Springs</td>
<td>4</td>
<td>1/32</td>
<td>23</td>
<td>32</td>
<td>44</td>
<td>242</td>
<td>1231</td>
<td></td>
</tr>
<tr>
<td>Edge Springs</td>
<td>8</td>
<td>1/16</td>
<td>46</td>
<td>63</td>
<td>98</td>
<td>484</td>
<td>2424</td>
<td></td>
</tr>
<tr>
<td>Center Springs</td>
<td>3</td>
<td>1/8</td>
<td>92</td>
<td>127</td>
<td>197</td>
<td>968</td>
<td>4886</td>
<td></td>
</tr>
</tbody>
</table>
Table 3.8 Spring Stiffnesses for Type IV Bearing Pads

<table>
<thead>
<tr>
<th>Type of Element</th>
<th>No. of Elements</th>
<th>Fraction of Total Stiffness</th>
<th>Spring Stiffness (kN/mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td>G = 0.655 MPa</td>
</tr>
<tr>
<td>Horizontal Elements</td>
<td>5</td>
<td>Full</td>
<td>2.00</td>
</tr>
<tr>
<td>Outer Springs</td>
<td>2</td>
<td>1/8</td>
<td>0.25</td>
</tr>
<tr>
<td>Inner Springs</td>
<td>3</td>
<td>1/4</td>
<td>0.50</td>
</tr>
<tr>
<td>Vertical Elements</td>
<td>15</td>
<td>Full</td>
<td>725</td>
</tr>
<tr>
<td>Corner Springs</td>
<td>4</td>
<td>1/32</td>
<td>23</td>
</tr>
<tr>
<td>Edge Springs</td>
<td>8</td>
<td>1/16</td>
<td>45</td>
</tr>
<tr>
<td>Center Springs</td>
<td>3</td>
<td>1/8</td>
<td>91</td>
</tr>
</tbody>
</table>

Modeling of the bearing pads with the spring elements is in good conformation with the actual bearing pads performance (Yazdani et al. 2000). The restraint effects of the bearing pads were found to be useful as the outward movement of the bearing pads was restrained by horizontal spring elements and the restraining effect can be well seen for the bearing pads with larger shear modulus.

3.2.6 Modeling of Link Slabs

The link slab is modeled using SOLID65 elements with reinforcement in the longitudinal direction. The width of the link slab is the same as the width of the bridge and the length depends on the span lengths of the bridge and level of debonding. In the present study, link slab lengths considered for the analysis are actual distance between the two spans plus 0%, 2.5% and 5% of the span lengths on each side of the central support. These lengths represent the portion of bridge deck that is not bonded to the girders; i.e. debonded. The link slab lengths are summarized in Table 3.9 for different span lengths of investigated bridges.
Table 3.9 Link slab lengths

<table>
<thead>
<tr>
<th>% Debonding</th>
<th>Type II</th>
<th>Type III</th>
<th>Type IV</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.0% (mm)</td>
<td>200</td>
<td>200</td>
<td>200</td>
</tr>
<tr>
<td>2.5% (mm)</td>
<td>802.5</td>
<td>1005</td>
<td>1435</td>
</tr>
<tr>
<td>5.0% (mm)</td>
<td>1605</td>
<td>2010</td>
<td>2870</td>
</tr>
</tbody>
</table>

The link slab nodes are only connected at the end nodes to the deck nodes along the bridge width. All other link slab nodes are not bonded (debonded) to the main bridge model. The link slab models and debonding are shown in the Figures 3.6 to 3.11 for different debonding lengths.

Figure 3.6 Bridge model showing link slab
Figure 3.7 Link Slab for 0% debonding

Figure 3.8 Debonding in 2.5% Link Slab
Figure 3.9 2.5% Link Slab for Type II Model

Figure 3.10 Debonding in 5% Link Slab
3.2.7 Application of Loads

The loads were applied to produce maximum negative moments in the link slab. A total of four HS20 trucks were placed on two spans based on AASHTO specifications. The loads were placed around the middle girder (Girder 3) to produce maximum continuity moment at central support and tensile force in the link slab.
3.3 Analytical Verification

U.S 27 Bridge (Yazdani et al. 2000) was used to verify the modeling of the bridge and the bearing pads. FDOT field tests were conducted on this simply supported bridge (L=20.10m) using two tractor trailers. The trailers loads were placed on the bridge in such a way to produce maximum positive moment in the girders. The field test results and FE modeling results were compared with respect to the average maximum deflections and the tensile strains obtained at the bottom of each beam flange at mid span. Six cases of support conditions were considered for analytical study that are compared with the field test results. Figure 3.13 and 3.14 show the deflection and strains obtained from the analysis.

![Figure 3.13 Maximum Deflections in U.S. 27 Bridge](image-url)
3.4 Conclusion

In this chapter, the procedure for modeling of bridge for different cases and the conditions and properties of the supports, modeling of link slab and bearings is discussed. The method followed to generate the model was verified using published experimental results from fields test.
CHAPTER 4. ANALYTICAL RESULTS AND DISCUSSIONS

4.1 Introduction

The results obtained from the finite element analyses described in the previous chapter are discussed. The continuity moment in left and right spans, the positive mid-span moments, and the link slab moments, maximum top fiber stress and average stress are extracted from the results and studied to establish trends with respect to bridge characteristics (bearing stiffnesses and debonding length). A discussion based on the results obtained from the plots drawn between bearing type cases, percentage of debonding in link slab is also presented.

4.2 Analysis by Finite Element Method

The ANSYS software package was utilized to perform static analyses of the FE models described earlier. The results presented in this chapter are obtained for the case of applied vehicular loads. Other load cases such as dead load are not considered since they have little effect on the continuity. The vehicular loads are placed on the deck slab model to produce maximum continuity (negative) moment in the system and maximum tensile force in the link slab. This location was determined by analyzing simple line models of two-span beams. Each FE model produced various results about the behavior of the system including the data for the link slab, horizontal and vertical reactions at supports and positive and negative moments in the bridge girders. The data extracted from the link slab results were maximum top fiber stress, top and bottom fiber forces, and the total net force. This data was extracted for total width of the link slab and as well as for the middle girder (G3), around which the loads were applied. The data extracted from FE results and the calculated quantities are tabulated and presented in Appendix B. The load positions of the axles on bridge models are shown in Figure 4.1 and 4.2.
Figure 4.1 Load Positions of axles on Bridge Models

a) Load position for Bridge II-S0-D0

b) Load position for Bridge III-S0-D0

c) Load position for Bridge IV-S0-D0

Figure 4.2: Transverse Load Positions of the Truck Wheels
4.3 General Behavior of Link Slab

Before presenting the results on which this study focuses Figure 4.3 shows the displacements of the link slab, it is useful to discuss bridge displacements at the joint under consideration for all the seven cases considered. As can be seen from the plot, both the top and bottom displacement of the link slab is in expansion. However, the bottom displacement is smaller than the top displacement. This implies that development of local flexure in the link slab in addition to the axial force. In the case of RHHR, it can be clearly observed that the top portion of the link slab is in tension and the bottom portion is in compression. Results from the HRRH case are similar to the other cases considered for the study. It is also observed from the plots that the displacement is more in the link slab at girder locations. The girder support displacement at central support is shown in the Figure 4.4, which clearly shows that the displacement in the middle girder support is more compared to the other girders displacement because of its proximity to the applied loads. The displacement for the HRRH case is more than that for Case 5 (stiffest bearing pad considered), which is expected as more resistance to girder movements is generated by the bearing pads as compared to an idealized roller support; i.e. k=0.

The displacement values for the Figure 4.4 are also listed in Table 4.1. Figures 4.3 and 4.4 are based on the analysis of Type IV bridge with zero skew angle and zero debonding length ratio (Bridge IV-D0-S0).

Figure 4.5 shows longitudinal normal stress distribution in the link slab for 0%, 2.5% and 5.0% debonding length ratios for Bridge IV-S0. It can be seen that longer the link slab (i.e. higher debonding length ratios) lead to smaller stresses at the extreme fibers which implies a reduction in the moment that develop locally in the link slab.
Figure 4.3 Link Slab Top and Bottom Fiber Displacements for Bridge IV-D0-S0
Figure 4.4 Average Displacement at Girder Supports (Bridge IV-D0-S0)

Table 4.1 Girder Support Displacement Details

<table>
<thead>
<tr>
<th>Bearing Stiffness Case</th>
<th>Left Displacement (mm)</th>
<th>Right Displacement (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Girder Number</td>
<td></td>
</tr>
<tr>
<td></td>
<td>1</td>
<td>2</td>
</tr>
<tr>
<td>RHHR</td>
<td>NA</td>
<td>NA</td>
</tr>
<tr>
<td>Case 1</td>
<td>-0.72</td>
<td>-1.11</td>
</tr>
<tr>
<td>Case 2</td>
<td>-0.71</td>
<td>-1.11</td>
</tr>
<tr>
<td>Case 3</td>
<td>-0.70</td>
<td>-1.10</td>
</tr>
<tr>
<td>Case 4</td>
<td>-0.63</td>
<td>-1.02</td>
</tr>
<tr>
<td>Case 5</td>
<td>-0.49</td>
<td>-0.84</td>
</tr>
<tr>
<td>HRRH</td>
<td>-0.58</td>
<td>-0.87</td>
</tr>
</tbody>
</table>
Figure 4.5 Stress Variation in Links Slabs with change in Debonding Length Ratio

(a) Stress Variation in Link Slab with 0% Debonding Length

(b) Stress Variation in Link Slab with 2.5% Debonding Length

(c) Stress Variation in Link Slab with 5.0% Debonding Length
4.4 Results of Analytical Models

Figures 4.6 to 4.32 are plots drawn for all the bridges considered in this study. Two groups of figures are presented. The first group presents the results for girder moments, positive midspan and negative at link slab end. The second group of figures shows the effects of studied parameters on link slab behavior.

4.4.1 Girder Moments

The effect of bearing stiffness at support on the continuity moment, $M_{CL}$, of the bridge can be observed in Figure 4.6 to 4.14. The results are presented for the entire bridge width and also for the middle girder G3 that is closest to applied loads. A general trend is observed for the entire bridge width and for the G3 in which higher support bearing stiffness results in higher continuity moments for Case 1 through 5. This increase in the continuity moment may be attributed to higher rigidity of the supports which attract larger forces because of the stiffer constraints they provide. It can be seen from the plots for the Case HRRH, a negligible moment develops while a much larger negative moment develops for the RHHR case as compared to other cases. It is also observed that debonding has small or negligible effect on the continuity moments. The positive mid span moments observed decreased with the increase in the stiffness of the bearing pads. The trend is similar for all span lengths and skew angles.

It can be inferred from the figures that the mid-span positive moments, $M_{Pl}$ and $M_{Pr}$, are influenced by bearing stiffness and percentage of link slab debonding. An increase in the bearing stiffness causes a drop in the mid-span positive moment for the same debonding length. Furthermore, it can be stated that the decrease is proportional to the debonding length ratio, $(L_{LS} / L)$. This is due to the fact that stiffer bearings attract larger horizontal reactions which in turn develop a larger continuity moment causing a reduction in the mid span positive moments to
satisfy the equilibrium. A similar observation can be stated for the debonding length ratio. Increasing the debonding length ratio reduces the stiffness of the link slab which leads to smaller continuity moments and hence larger positive moments. A similar trend is observed in all the cases considered.

Figure 4.6 Effect of bearing stiffness on positive and negative moment (Bridge II-S0)
Figure 4.7 Effect of bearing stiffness on positive and negative moment (Bridge II-S30)
Figure 4.8 Effect of bearing stiffness on positive and negative moment (Bridge II-S45)
Figure 4.9 Effect of bearing stiffness on positive and negative moment (Bridge III-S0)
Figure 4.10 Effect of bearing stiffness on positive and negative moment (Bridge III-S30)
Figure 4.11 Effect of bearing stiffness on positive and negative moment (Bridge III-S45)
Figure 4.12 Effect of bearing stiffness on positive and negative moment (Bridge IV-S0)
Full Width

(a) Continuity Moment, $M_{CL}$ vs. Bearing Stiffness Types

Girder G3

(b) Mid Span positive Moment vs. Bearing Stiffness

Figure 4.13 Effect of bearing stiffness on positive and negative moment (Bridge IV-S30)
Figure 4.14 Effect of bearing stiffness on positive and negative moment (Bridge IV-S45)
4.4.2 Link Slab

The local bending moment in the link slab, $M_{LS}$, was computed for all analyzed cases. It was observed that $M_{LS}$ decreases with the increase in bearing stiffness. The rigidity at central supports affects the link slab forces and moments as it can be observed from the figure 4.15 to 4.23. The variation is similar for total width of link slab and for the portion of the link slab associated with middle girder (G3). It is also observed that, for the RHHR case the link slab moment is found to be the least although it has been found that this case produces the largest continuity moment. This indicates that larger bearing stiffnesses subject the link slab to larger axial forces and smaller local bending moments. This finding is of importance as it may affect the cracking pattern that will develop in the link slab. Similar trends can be seen for all bridges considered in this study.

A decrease in link slab moment is observed with the increase in the debonding length ratio. The reduction of link slab moment is due to the fact that longer link slabs are less stiff, and hence, they generate smaller continuity moments in the girders and local bending moments in the link slab. This trend can be viewed in the Figures 4.24 to 4.32.

As stated earlier, the average stress in the link slab, $f_{LS,ave}$, varies with bearing stiffness. A gradual increase is observed in average stress in the link slab with increase in bearing stiffness because of the larger continuity moment caused by these supports. However, the increased stiffness has a reducing effect on the extreme fiber stresses as it tends to generate smaller local bending moments in the link slab.
Figure 4.15 Effect of bearing stiffness on Link Slab Moment and Stresses (Bridge II-S0)
Figure 4.16 Effect of bearing stiffness on Link Slab Moment and Stresses (Bridge II-S30)
Figure 4.17 Effect of bearing stiffness on Link Slab Moment and Stresses (Bridge II-S45)
Figure 4.18 Effect of bearing stiffness on Link Slab Moment and Stresses (Bridge III-S0)
Figure 4.19 Effect of bearing stiffness on Link Slab Moment and Stresses (Bridge III-S30)
Figure 4.20 Effect of bearing stiffness on Link Slab Moment and Stresses (Bridge III-S45)
Figure 4.21 Effect of bearing stiffness on Link Slab Moment and Stresses (Bridge IV-S0)
Figure 4.22 Effect of bearing stiffness on Link Slab Moment and Stresses (Bridge IV-S30)
Figure 4.23 Effect of bearing stiffness on Link Slab Moment and Stresses (Bridge IV-S45)
Figure 4.24 Effect of Debonding Length Ratio on Link Slab Moment and Stresses (Bridge II-S0)
Figure 4.25 Effect of Debonding Length Ratio on Link Slab Moment and Stresses
(Bridge II-S30)
Figure 4.26 Effect of Debonding Length Ratio on Link Slab Moment and Stresses (Bridge II-S45)
Figure 4.27 Effect of Debonding Length Ratio on Link Slab Moment and Stresses (Bridge III-S0)
Figure 4.28 Effect of Debonding Length Ratio on Link Slab Moment and Stress (Bridge III-S30)
Figure 4.29 Effect of Debonding Length Ratio on Link Slab Moment and Stresses (Bridge III-S45)
Figure 4.30 Effect of Debonding Length Ratio on Link Slab Moment and Stresses (Bridge IV-S0)
Figure 4.31 Effect of Debonding Length Ratio on Link Slab Moment and Stresses (Bridge IV-S30)
Figure 4.32 Effect of Debonding Length Ratio on Link Slab Moment and Stresses (Bridge IV-S45)
4.5 Conclusion

The conclusion drawn from above discussion is that the continuity moments in girders and link slab local moments are effected by the bearing pad stiffness. Furthermore, link slab moments are also effected by the debonding length of the link slab.
CHAPTER 5. MODIFIED THREE MOMENT EQUATION

5.1 Introduction

In this chapter, a brief discussion of the classical three moment equation for continuous spans is presented. Two continuous spans are considered for the discussion. A modified three moment equation (Okeil and El-Safty 2005) is derived for the partial continuity induced in bridges with jointless decks. The expressions developed by Okeil and El-Safty (2005) are for the idealized support conditions cases (HRRH & RHHR). An attempt to further modify the expressions for the general case of any support condition is presented. An analytical verification was conducted by comparing the results for the idealized cases to what is reported by Okeil and El-Safty (2005).

5.2 Classical Three Moment Equation

The classical three moment equation for two continuous spans can be derived using the moment area method. The three moment equation was first derived by Emile Clapeyron in 1857 using differential equations of beam bending. Alternatively, the same equation may be obtained by studying two continuous spans. The principle of superposition is utilized for this approach. The moments generated by the applied loading at the continuity support can be separated into two separate problems. Firstly, the two adjacent spans are treated as simply supported spans with the applied loading. Secondly, the spans are assumed to be acted upon by the unknown internal continuity moments produced by the applied load.

Bending moment diagrams are drawn for both cases; the areas under the moment diagram are evaluated and multiplied with the centroidal distance of the corresponding areas from the supports. The rotation at the central support establishes the compatibility condition between both spans at the joint. Using second-moment area method, the rotations in both spans are determined.
in terms of unknown internal moments and flexural rigidity. All the expressions are solved to obtain three moment equation for continuous spans.

5.3 Derivation of Modified Three Moment Equation

In this section, the details of a closed-form derivation are presented. The model is based on the work published Okeil and El-Safty (2005) in which two idealized support conditions were considered. The idealized cases are namely, HRRH for hinged-roller-roller-hinged supports and RHHR for roller-hinged-hinged-roller supports. Although these idealized cases are considered upper and lower bounds for the behavior of partially continuous systems, most bridges are actually supported on bearing pads and fall in between those two bounds.

5.3.1 Basic Assumptions for Current Study

In this study, a more general case is considered for the analysis of the continuity of girders with different support stiffnesses. Figure 5.1 shows basic structural model used in the derivation where bearing pads are modeled as horizontal springs. The other restraints provided by the bearing pad are ignored as their contribution to the focus of this study (continuity) is deemed negligible. Here only two continuous girders are shown. The derivation focuses on the central support to develop an expression that links continuity moments to system characteristics such as dimensions and stiffnesses. The approach is similar to the one used in developing the “three-moment equation” which is used for the analysis of continuous multi-span beam systems.

In the case of fully continuous systems, the three moment equation is given as:

\[
M_0 \frac{L_L}{(EI)_L} + 2M_1 \left[ \frac{L_L}{(EI)_L} + \frac{L_R}{(EI)_R} \right] + M_2 \frac{L_R}{(EI)_R} = -6 \left[ \frac{r_{1L}}{(EI)_L} + \frac{r_{1R}}{(EI)_R} \right]
\]

where, by considering two adjacent spans for a generic loading as seen in Fig. 5.1, \( M_i \)
Figure 5.1 Model used for closed-form derivation showing springs representing bearing stiffness at girders ends are the continuity moments at the supports; \( r_{1L} \) and \( r_{1R} \) are the central support reactions for each span independently due to the elastic load, \( M/EI \), and \( (EI) \) and \( L \) are the flexural rigidity and span length respectively. Figure 5.2 Shows the end rotations used in the derivation of Equation 5.1. In the figure, the spans are separated and studied individually. They are then reconnected by imposing the appropriate compatibility condition at the interface between them; i.e. central support.

It should be noted that, unlike in normal continuity formulations, the continuity moment at the central support, \( M_1 \), is here assumed to be unequal, on both sides of the support. For the case of partial continuity, continuity moments at the central support may be expressed in terms of the forces generated there. By designating the moments as \( M_{1R} \) and \( M_{1L} \) for right and left spans respectively, one can express these moments as

\[
-M_{1L} = T_{1L} h_i + P_{k1L} h_b
\]  
(5.2)

\[
-M_{1R} = T_{1R} h_i + P_{k1R} h_b
\]  
(5.3)

These forces are: the tensile force in the link slab, \( T \); the forces in the bearing pads at supports \( P_{k1L} \) and \( P_{k1R} \) at the \( i^{th} \) joint for the left and right girder, respectively. \( h_i \) is the distance from the center of the link slab to the centroid of the slab and girder system and \( h_b \) is the distance from the support to the centroid of the slab and girder system.
5.3.2 Compatibility Condition at Central Support

Rotations due to the applied loads and due to continuity moments will be used to establish the compatibility condition. The following equations can be found from the end rotations due to applied loads and continuity moments (see Figure 5.2). End rotations due to applied loads are represented as $\phi_{1L}$ and $\phi_{1R}$, and end rotations due to continuity moments are labeled as $\theta_{1L}$ and $\theta_{1R}$ for the left and right side respectively. The modulus of elasticity of girder, $E_g$, and moment of inertia of girders on both sides are different and are represented as $I_{gL}$ and $I_{gR}$ respectively for left and right side.

$$\phi_{1L} = -\frac{r_{1L}}{E_g I_{gL}}$$ (5.4)
\[
\phi_{1R} = -\frac{r_{1R}}{E_g I_{gR}}
\] (5.5)

\[
\theta_{1L} = -\frac{1}{E_g I_{gL}} \left[ \frac{M_0 L_L}{6} + \frac{M_{1L} L_L}{3} \right]
\] (5.6)

\[
\theta_{1R} = -\frac{1}{E_g I_{gR}} \left[ \frac{M_2 L_R}{6} + \frac{M_{1R} L_R}{3} \right]
\] (5.7)

Figure 5.3 shows the deformations of girder ends at the central support due to an arbitrary load. It should be noted that the centroidal points will also translate longitudinally due to: (1) girder elongation/shortening as a result of the net axial force acting on them, and (2) flexural rotations which are not necessarily pivoted about the centroidal point.

Due to the complexity of this general deformed shape, it is logical to consider each movement (translation and rotation) separately and then superimpose them to study the total effect. The main compatibility condition used in this derivation is between the girder tops at the central support, where the total movement due to girder elongation/shortening and girder rotation must be equal to the elongation of the link slab. By assuming that the link slab will crack due to the applied tension force, only steel reinforcement will contribute to the stiffness of the link slab,
$K_{\text{link}}$, which will be assumed to behave as an axial member. As such, the following compatibility equation is obtained

$$\left(-\alpha_{1L} h_t - \delta_{1L}\right) + \left(\alpha_{1R} h_t - \delta_{1R}\right) = \frac{T_{1L} + T_{1R}}{2K_{\text{link}}}$$  \hspace{1cm} (5.8)

Here tension forces in the link slab are considered as $T_{1L}$ and $T_{1R}$ on left and right side respectively. It may be noted that forces in the right and left supports vary, but the net tensile force in the link slab on both ends of the link slab is equal.

In equation 5.8, the total rotation of the girder due to applied load and continuity of the girders. The total rotations at the end can be written as

$$\alpha_{1L} = \phi_{1L} + \theta_{1L}$$  \hspace{1cm} (5.9)

$$\alpha_{1R} = \phi_{1R} + \theta_{1R}$$  \hspace{1cm} (5.10)

In Equation 5.8, $\delta_{1L}$ and $\delta_{1R}$ are the elongation at the centroid of the cross section of the girders. For the general case when the girders are resting on bearing pads stiffness, $K_{1L}$ and $K_{1R}$, the elongations are given as:

$$\delta_{1L} = \frac{\Delta F_{1L}}{K_{el}}$$  \hspace{1cm} (5.11)

$$\delta_{1R} = \frac{\Delta F_{1R}}{K_{er}}$$  \hspace{1cm} (5.12)

where $K_{el}$ and $K_{er}$ are the equivalent stiffness for the spring system shown in Figure which represents the stiffness of the girder, $K_{br}$, and bearing pads’ stiffness at the girders ends, $K_{2R}$ and $K_{1R}$.

The equivalent stiffness can be derived from basic principles as

$$K_{er} = \frac{K_{br} K_{2R} + K_{br} K_{1R} + K_{1R} K_{2R}}{K_{br} + K_{2R}}$$  \hspace{1cm} (5.13)
5.3.3 Equilibrium Condition at Central Support

In addition to the compatibility requirements, equilibrium also has to be satisfied. At the central support, the tensile force in the link slab, $T$, can be expressed in terms as in equation 5.15 and 5.16.

\[
\Delta F_L = T_{1L} - P_{k1L} \tag{5.15}
\]

\[
\Delta F_R = T_{1R} - P_{k1R} \tag{5.16}
\]

\[
P_{k1L} = K_{1L} \alpha_{1L} h_b \tag{5.17}
\]

\[
P_{k1R} = K_{1R} \alpha_{1R} h_b \tag{5.18}
\]

In the previous equations, the effect of girder elongation on $P_{k1L}$ and $P_{k1R}$ is ignored as an approximation.

5.3.4 Solution

The closed-form solution is obtained by substituting for $M_{1L}$ and $M_{1R}$ in Equations 5.9 and 5.10 by the expressions in Equations 5.2, 5.3 and 5.4. Further reduction in variables is achieved by substituting for $\delta_{1L}$ and $\delta_{1R}$ from Equations 5.11 and 5.12 and $\alpha_{1L}$ and $\alpha_{1R}$ from Equations 5.9 and 5.10 into the compatibility equations (5.9).
Simplifying and rearranging yields the following general solution for the problem of partial continuity

\[
M_0 \left[ \frac{L_L H_L}{E_g I_L} \right] + 2M_1L \left[ \frac{L_L H_L}{E_g I_L} + 3 \left( \frac{1}{K_{eL}} + \frac{1}{2.2K_{link}} \right) \right] + \cdots \\
2M_1R \left[ \frac{L_R H_R}{E_g I_R} + 3 \left( \frac{1}{K_{eR}} + \frac{1}{2.2K_{link}} \right) \right] + M_2 \left[ \frac{L_R H_R}{E_g I_R} \right] = -6 \left[ \frac{r_{1L} H_L}{E_g I_L} + \frac{r_{1R} H_R}{E_g I_R} \right] 
\]

(5.19)

where

\[
H_L = \left[ h_t^2 - \frac{K_{iL} h_b h}{K_{eL}} - \frac{K_{1L} h_b^2}{2K_{link}} \right] 
\]

(5.20)

\[
H_R = \left[ h_t^2 - \frac{K_{iR} h_b h}{K_{eR}} - \frac{K_{1R} h_b^2}{2K_{link}} \right] 
\]

(5.21)

Equation 5.19 involves four unknowns \((M_0, M_{1L}, M_{1R}, M_2)\) which requires an additional equation to be able to obtain a solution. From equations 5.2, 5.3, 5.9, 5.10, 5.17 and 5.18 another expression for moments can be derived as follows

\[
M_0 \left[ \frac{L_L K_{iL} h_b^2}{E_g I_L} \right] + 2M_1L \left[ \frac{L_L K_{iL} h_b^2}{E_g I_L} - 1 \right] + \cdots \\
2M_1R \left[ \frac{L_R K_{iR} h_b^2}{E_g I_R} + 1 \right] + M_2 \left[ \frac{L_R K_{iR} h_b^2}{E_g I_R} \right] = -6 \left[ \frac{r_{1L} K_{iL} h_b^2}{E_g I_L} + \frac{r_{1R} K_{iR} h_b^2}{E_g I_R} \right] 
\]

(5.22)

5.3.5 Verification

The derived expressions are now studied to ensure that they yield the same expressions as those reported in the literature (Okeil and El-Safty 2005) for ideal support conditions. For the hinge-roller-roller-hinge (HRRH) case the bearing stiffnesses are as follows

\[
K_{0L} = \infty \quad K_{1L} = 0 \quad K_{1R} = 0 \quad K_{2R} = \infty 
\]
Also, due to the fact that there is no variation between both sides at the internal support, the continuity moments are equal; i.e.

\[ M_{1L} = M_{1R} = M_1 \]

Furthermore, the equivalent stiffnesses, \( K_{el} \) and \( K_{er} \), reduce to

\[ K_{el} = K_{1L} + \frac{1}{\frac{1}{K_{bl}} + \frac{1}{K_{0L}}} \], and \( K_{er} = K_{1R} + \frac{1}{\frac{1}{K_{bR}} + \frac{1}{K_{2R}}} \)

which leads to

\[ K_{el} = K_{bl} = \frac{E_g A_{gl}}{L_L}, \quad K_{er} = K_{bR} = \frac{E_g A_{gR}}{L_R} \]

\( H_L = h_i^2 \) and \( H_R = h_i^2 \)

Substituting for these expressions in Equation 5.19 and arranging the terms yields the following expression which is identical to the HRRH case expression in Okeil and El-Safty (2005).

\[ M_0 \frac{L_L}{E_g I_{gL}} + 2M_1 \left[ \frac{L_L}{E_g I_{gL}} + \frac{L_R}{E_g I_{gR}} + \frac{3}{h_i^2} \left( \frac{L_L}{E_g A_{gL}} + \frac{L_R}{E_g A_{gR}} + \frac{L_{Link}}{E_s A_s} \right) \right] + \ldots \]

\[ M_2 \frac{L_R}{E_g I_{gR}} = -6 \left[ \frac{r_{1L}}{E_g I_{gL}} + \frac{r_{1R}}{E_g I_{gR}} \right] \]

Similarly, for the roller-hinge-hinge-roller case in Okeil and El-Safty (2005) is obtained by substituting for the bearing pad stiffnesses by the following values

\[ K_{0L} = 0 \quad K_{1L} = \infty \quad K_{1R} = \infty \quad K_{2R} = 0 \]

which leads to the following equivalent system stiffnesses

\[ K_{el} = K_{1L} + \frac{1}{\frac{1}{K_{bl}} + \frac{1}{K_{0L}}} \], and \( K_{er} = K_{1R} + \frac{1}{\frac{1}{K_{bR}} + \frac{1}{K_{2R}}} \)

Therefore

\[ K_{el} = \infty, K_{er} = \infty, H_L = h_i^2, \text{ and } H_R = h_i^2 \]
As before, continuity moments at the central support are also equal on both sides; i.e.

\[ M_{1L} = M_{1R} = M_1 \]

Substituting for these variables in Equation 5.19 and arranging the terms results in the desired equation.

\[
M_0 \frac{L_L}{E_g I_{gl}} + 2M_1 \left[ \frac{L_L}{E_g I_{gl}} + \frac{L_R}{E_g I_{gr}} + \frac{3}{h^2} \left( \frac{L_{link}}{E_g A_s} \right) \right] + M_2 \frac{L_R}{E_g I_{gr}} = -6 \left[ \frac{r_{1L}}{E_g I_{gl}} + \frac{r_{1R}}{E_g I_{gr}} \right]
\] (5.24)

5.4 Conclusion

The derived modified three moment expression is based on the classical three moment equation and the expressions derived by Okeil and ElSafty (2005) for jointless bridges on idealized supports. Equation 5.19 yields similar expressions to those derived for the cases with idealized support conditions; HRRH and RHHR. A thorough parametric study is required to validate the derived modified three moment expression presented here for different support conditions.

From expression in Equation 5.19, it can be inferred that the modified three moment equation is influenced by bearing stiffness influence parameters \( H_L \) and \( H_R \). The effects of bearing stiffness on forces and moments in a bridge system need to be investigated via a parametric study.
CHAPTER 6. CONCLUSIONS AND RECOMMENDATIONS

6.1 Summary

The present study investigates the behavior of bridges with link slab (which connects the decks of the adjacent spans). FE bridge models were developed to study the behavior of the partially continuous system. A single simple span bridge U.S. 27 was considered to validate the ANSYS FE model used in this study. After the model was verified, a parametric study is conducted using two-span bridge models with link slab. One hundred and sixty nine analytical bridge models were generated to study the effect of different parameters on the behavior of the system. The parameters considered in the study are the stiffness of bearing pad, girder properties, debonding length ratio, and skew angle.

Displacements, forces, stresses and moments from the FE analyses were investigated. The analytical results were used to study the effects of the bearing stiffness and the debonded link slab length ratio ($L_{LS}/L$) on the behavior of the system. Several measures of the behavior were considered; namely, top fiber stress and over all link slab force. The moment produced by the link slab and the continuity moment in the bridge girders were also studied.

A modified three moment equation was then derived for bridges with link slabs. The proposed equation can be used in the future to obtain a closed form solution for the analysis of link slab, which can be used for the development of a design tool for link slabs.

6.2 Conclusions

Based on the results from this study, the following conclusions may be drawn:-

1. Increasing the bearing pad stiffness caused the continuity moments in the spans to increase. The increase in the continuity moment for the most loaded girder (G3) was in the range of 80% for bearing pad cases as compared to Case 1. The increase was considerably larger (1150%) for HRRH case. For the RHHR case, (infinite stiffness at linked joint), the highest
continuity moments were produced. Conversely, the HRRH case produced the minimum continuity moment as a result of the minimum retraining effect it provides. It is obvious that the force in bearing pads increase with the increase in stiffness, which is the cause for more continuity moment.

2 As a result of the change in continuity moments, the positive mid-span moment also changed to satisfy equilibrium. The positive midspan moment in both spans of the bridge decreases with the increase in the stiffness of the bearing pads. The decrease in the mid-span moment ranged from 0% to 18% as compared to Case 1.

3 The study revealed that the local moments in the link slab decreased as the bearing pads stiffness increased. For the RHHR case, the link slab moment is found to be minimum compared to other cases. This could be due to the fact that increasing the bearing stiffness causes the link slab to behave more like a axial member under tension rather than a flexural member.

4 The use of stiffer bearing pads causes less displacement at the support level and more extension at the link slab level. Hence, the partial continuity develops as a result of the axial stiffness of the link slab rather than its flexural rigidity, which leads to larger average stresses, smaller extreme fiber stresses in the link slab.

5 It may be stated that the local moment in the link slab decreased with the increase in debonding length ratio. This may be due to the fact that shorter link slabs are stiffer in flexure and hence they produce more continuity in the system that leads to larger moment in the slab. A similar conclusion can be drawn for stress results. Changing the debonding length seemed to have no effect on the average stresses in the link slab. However, the maximum top fiber stress in the link slab was found to decrease with the increase in
debonding length of the link slab. Thus, it may be concluded that longer debonding length cause decrease in top fiber stress and decrease in link slab moment.

6 A general modified three moment expression was derived for the complex structural system of bridges with link slabs. Two bearing influence factors $H_L$ and $H_R$, were defined in Equation 5.20 and 5.21. They directly influence the continuity moments in the spans. The derived expressions yielded identical results to other three moment equations found in the literature for idealized cases.

6.3 Recommendations for Future Research

1. In the present study, the link slab behavior is studied for different skew angles, debonding lengths and bearing stiffnesses. The bearing stiffness at the supports was considered same for all the four supports. The behavior of the link slab can be further studied for the cases with different bearing stiffnesses at each support.

2. The behavior of the link slab was studied for equal spans on both sides of the link slab; a study of link slab behavior with different span length is needed.

3. Same girder types and depths were considered for the present study. The behavior of the link slab needs to be studied for different girder types and depths with same span length.

4. A thorough parametric study of the modified three moment equation can be conducted to establish a simplified procedure that can be used in a design environment.
REFERENCES


2. AASHTO (1996a) LRFD Bridge Design Specifications, American Association of State Highway and Transportation Officials, Washington D.C.


4. ANSYS 5.6 users manual, Ansys Inc.

5. ANSYS reference manual, Ansys Inc.


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<th>Abbreviation</th>
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APPENDIX A. COMPILATION OF CONTINUITY DIAPHRAGM DETAILS FOR FEW STATE DOT’S OF UNITED STATES

Figure A.1: Section Details between Girders from Idaho DOT

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| TABLE "B" |        |
| NUMBER OF BARS |     |
| D         |        |
| E         |        |
| F         |        |
| G         |        |

NOTE:
NO HORIZONTAL CONSTRUCTION JOINTS ALLOWED BETWEEN TOP SLAB AND END DIAPHRAGM.

ALTERNATE LOCATION OF CUT STRAND AT ADJACENT GIRDER ENDS

G4 OR G1A DOWEL BARS

CUT 4 STRAND IN BOTTOM ROW WITH 610 mm PROJECTION SHOPE BEND AS SHOWN WITH 25 mm RADIUS

GIRDER END DETAILS

90
Diaphragm Over Expansion Piers

Roofing felt shall be bonded to side of beams embedded into diaphragm. Cost Incidental.

Pour diaphragm flush with bottom of slab. Concrete in slab above this line shall be placed not less than 45 min. nor more than 90 min. after diaphragm has been poured.

Note: Details for I-Beams shown. Details for Bulb-T's similar except as shown.

*See Fig. 1.4.22 for diaphragm dimensions.

Figure A.2: Section Details between Girder from Illinois DOT

SECTION A - A
Figure A.3: Section Details between Girders from Illinois DOT
Figure A.4: Section Details between Girders from Pennsylvania DOT
Continuity Detail at Continuity Diaphragm

**Note A**
Dowels in the cap shall be galvanized No. 35 deformed reinforcing steel. Prior to pouring the continuity diaphragm, each dowel shall be sheathed by 45mm I.D rigid sleeve with a 12mm thick compressible cap above the top of the dowel to allow the girders to deflect on their bearing pads under future loads.

**Note B**
Risers at fixed bents shall be sloped to approximate tangent grade at CL bent. Elevations shown for these risers are at CL bent.

Figure A.5: Section Details between Girders from Louisiana State DOT
Figure A.6: Section Details between Girders from Virginia DOT
Figure A.7: Section Details between Girders from Georgia DOT
Figure A.8: Section Details between Girders from Massachusetts
APPENDIX B. DATA RECORDED AND CALCULATED RESULTS FROM THE ANALYSIS OF FE MODELS IN ANSYS

Table B.1: Data Extracted from Analysis of Bridge II-S0-D0

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### Table B.28: Calculations for data extracted from Analysis of Bridge II-S0-D0

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Mid Span Moments (kNm): 666.02 1646.34

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Mid Span Moments (kNm): 637.60 1567.68

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Mid Span Moments (kNm): 566.34 1400.18

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Mid Span Moments (kNm): 214.74 1102.20
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Mid Span Moments (kNm) 465.74 1324.66

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Mid Span Moments 682.15 1661.50

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Mid Span Moments 680.79 1657.58

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Mid Span Moments 677.48 1648.97

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Mid Span Moments 648.41 1582.56

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Mid Span Moments 582.03 1435.78

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Mid Span Moments 211.24 1105.21
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Table B.32: Calculations for data extracted from Analysis of Bridge II-S30-D25

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Mid Span Moments (kNm) 434.74 1249.23

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Mid Span Moments (kNm) 650.83 1575.02

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Mid Span Moments (kNm) 649.85 1569.98

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Mid Span Moments (kNm) 646.91 1559.27

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Mid Span Moments (kNm) 618.82 1492.21

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Mid Span Moments (kNm) 558.78 1363.69

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Mid Span Moments (kNm) 248.92 1057.86
Table B.33: Calculations for data extracted from Analysis of Bridge II-S30-D50

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| Case 1 | 1080.84 |

| G3 | 27.70 | -28.10 | 0.0119 | -0.0121 | 78.92 | 0.28 |
| Total | 1.44 | -2.61 | 0.0006 | -0.0011 | 230.47 | 0.00 |

| Mid Span Moments (kNm) | 587.68 |
| Case 2 | 1398.80 |

| G3 | 27.91 | -28.45 | 0.0120 | -0.0122 | 77.36 | 0.28 |
| Total | 1.98 | -3.58 | 0.0009 | -0.0015 | 225.92 | 0.00 |

| Mid Span Moments (kNm) | 586.87 |
| Case 3 | 1392.46 |

| G3 | 28.21 | -29.01 | 0.0121 | -0.0125 | 74.31 | 0.28 |
| Total | 2.96 | -5.34 | 0.0013 | -0.0023 | 216.94 | 0.00 |

| Mid Span Moments (kNm) | 584.11 |
| Case 4 | 1378.80 |

| G3 | 30.81 | -34.29 | 0.0132 | -0.0147 | 62.32 | 0.28 |
| Total | 12.71 | -22.98 | 0.0055 | -0.0099 | 181.05 | 0.00 |

| Mid Span Moments (kNm) | 557.92 |
| Case 5 | 1307.69 |

| G3 | 42.54 | -57.12 | 0.0183 | -0.0245 | 51.22 | 0.27 |
| Total | 52.65 | -94.85 | 0.0226 | -0.0407 | 146.84 | 0.01 |

| Mid Span Moments (kNm) | 507.96 |
| RHHR | 1201.61 |

| G3 | 302.54 | -382.30 | 0.1298 | -0.1641 | 12.74 | 0.78 |
| Total | 848.57 | -848.57 | 0.3642 | -0.3642 | 27.99 | 0.36 |

| Mid Span Moments (kNm) | 273.46 |

<p>| 963.37 |</p>
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<td></td>
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<td>Stress</td>
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Mid Span Moments (kNm) 404.13 1108.74

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Mid Span Moments (kNm) 612.73 1424.62

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Mid Span Moments (kNm) 611.29 1418.40

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<td>G3</td>
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<td>Total</td>
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Mid Span Moments (kNm) 607.23 1405.13

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Mid Span Moments (kNm) 576.14 1335.43

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Mid Span Moments (kNm) 520.19 1225.46

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Mid Span Moments (kNm) 266.67 970.72
Table B.36: Calculations for data extracted from Analysis of Bridge II-S45-D50

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<td>Left</td>
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Mid Span Moments (kNm) 415.89 1132.80

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Mid Span Moments 631.97 1447.24

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<th>Link Slab</th>
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<tr>
<td>G3</td>
<td>27.20</td>
<td>-27.75</td>
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Mid Span Moments 629.95 1441.50

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Mid Span Moments 625.05 1429.31

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<th>Link Slab</th>
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<tbody>
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<td>G3</td>
<td>30.97</td>
<td>-34.59</td>
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Mid Span Moments 592.24 1362.03

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Mid Span Moments 532.07 1247.58

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Mid Span Moments 261.96 976.32
Table B.37: Calculations for data extracted from Analysis of Bridge III-S0-D0

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<td></td>
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<td></td>
<td>Moment (kNm)</td>
</tr>
<tr>
<td>Left</td>
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<td>Left</td>
<td>Right</td>
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<td>G3</td>
<td>-19.29</td>
<td>19.29</td>
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<tr>
<td>Total</td>
<td>-249.44</td>
<td>249.44</td>
<td>-0.0594</td>
</tr>
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</table>

Mid Span Moments (kNm) 1698.30 1885.42

Case 1

| Left | Right | Left | Right |                      |                          |
| G3   | 60.02 | -60.16 | 0.0143 | -0.0143 | 135.52 | 0.35 | 8.90 |
| Total| 5.93  | -6.19  | 0.0014 | -0.0015 | 402.94 | 0.00 | 8.90 |

Mid Span Moments (kNm) 2041.72 2269.43

Case 2

| Left | Right | Left | Right |                      |                          |
| G3   | 60.71 | -60.90 | 0.0145 | -0.0145 | 134.95 | 0.35 | 8.89 |
| Total| 8.11  | -8.46  | 0.0019 | -0.0020 | 400.77 | 0.00 | 8.89 |

Mid Span Moments (kNm) 2038.12 2264.84

Case 3

| Left | Right | Left | Right |                      |                          |
| G3   | 61.95 | -62.24 | 0.0148 | -0.0148 | 133.98 | 0.35 | 8.86 |
| Total| 12.34 | -12.88 | 0.0029 | -0.0031 | 397.41 | 0.00 | 8.86 |

Mid Span Moments (kNm) 2031.27 2256.61

Case 4

| Left | Right | Left | Right |                      |                          |
| G3   | 74.56 | -75.90 | 0.0178 | -0.0181 | 126.89 | 0.36 | 8.43 |
| Total| 57.27 | -59.70 | 0.0136 | -0.0142 | 373.68 | 0.01 | 8.43 |

Mid Span Moments (kNm) 1965.61 2184.17

Case 5

| Left | Right | Left | Right |                      |                          |
| G3   | 127.97 | -133.72 | 0.0305 | -0.0318 | 110.04 | 0.38 | 7.30 |
| Total| 236.28 | -246.06 | 0.0563 | -0.0586 | 318.85 | 0.04 | 7.30 |

Mid Span Moments (kNm) 1800.80 2008.97

Case 6

| Left | Right | Left | Right |                      |                          |
| G3   | 725.70 | -764.71 | 0.1728 | -0.1821 | 15.67 | 1.11 | 1.60 |
| Total| 1887.79 | -1887.79 | 0.4495 | -0.4495 | 30.29 | 0.59 | 1.60 |

Mid Span Moments (kNm) 1150.36 1400.93
Table B.38: Calculations for data extracted from Analysis of Bridge III-S0-D25

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<th>Moment (kNm)</th>
<th>Average Stress (N/mm²)</th>
<th>Maximum Top Fiber Stress (N/mm²)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Left</td>
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<td></td>
<td>Left</td>
<td>Right</td>
<td></td>
<td></td>
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<tr>
<td>G3</td>
<td>-18.86 18.86 0.0045 0.0045 -18.86 -0.11 2.10</td>
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<tr>
<td>Total</td>
<td>-246.10 246.10 -0.0586 0.0586 -246.10 -0.30 2.10</td>
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<td>Mid Span Moments (kNm)</td>
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Case 1 Continuity Moment (kNm) | DF | Link Slab | Moment (kNm) | Average Stress (N/mm²) | Maximum Top Fiber Stress (N/mm²) |
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<td>Left</td>
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<tr>
<td>G3</td>
<td>60.89 -61.03 0.0145 -0.0145 76.13 0.36 2.75</td>
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Case 2 Continuity Moment (kNm) | DF | Link Slab | Moment (kNm) | Average Stress (N/mm²) | Maximum Top Fiber Stress (N/mm²) |
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<td>G3</td>
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Case 3 Continuity Moment (kNm) | DF | Link Slab | Moment (kNm) | Average Stress (N/mm²) | Maximum Top Fiber Stress (N/mm²) |
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<td>G3</td>
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Case 4 Continuity Moment (kNm) | DF | Link Slab | Moment (kNm) | Average Stress (N/mm²) | Maximum Top Fiber Stress (N/mm²) |
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<td>Right</td>
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<tr>
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Case 5 Continuity Moment (kNm) | DF | Link Slab | Moment (kNm) | Average Stress (N/mm²) | Maximum Top Fiber Stress (N/mm²) |
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<td>G3</td>
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RHHR Continuity Moment (kNm) | DF | Link Slab | Moment (kNm) | Average Stress (N/mm²) | Maximum Top Fiber Stress (N/mm²) |
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<td>Right</td>
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<tr>
<td>G3</td>
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Table B.39: Calculations for data extracted from Analysis of Bridge III-S0-D50

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<th>Link Slab</th>
<th>Moment (kNm)</th>
<th>Average Stress (N/mm²)</th>
<th>Maximum Top Fiber Stress (N/mm²)</th>
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Table B.40: Calculations for data extracted from Analysis of Bridge III-S30-D0

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Table B.41: Calculations for data extracted from Analysis of Bridge III-S30-D25

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Mid Span Moments (kNm) 1378.87 1586.44

Case 1

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Mid Span Moments (kNm) 1752.33 1969.13

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Mid Span Moments (kNm) 1742.99 1960.39

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Mid Span Moments (kNm) 1728.55 1946.60

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Mid Span Moments (kNm) 1644.49 1862.44

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Mid Span Moments (kNm) 1515.68 1724.43

RHHR

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Mid Span Moments (kNm) 1054.03 1288.01
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### Table B.45: Calculations for data extracted from Analysis of Bridge III-S45-D50

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**Mid Span Moments (kNm)**

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**Mid Span Moments**

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**Mid Span Moments**

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**Mid Span Moments**

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**Mid Span Moments**

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**Mid Span Moments**

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**Mid Span Moments**

<p>|          | 1053.75 | 1291.87 |</p>
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Mid Span Moments (kNm)

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Mid Span Moments (kNm)

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Mid Span Moments (kNm)

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Mid Span Moments (kNm)

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Mid Span Moments (kNm)

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Mid Span Moments (kNm)
Table B.48: Calculations for data extracted from Analysis of Bridge IV-S0-D50

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<th>Average Stress (N/mm²)</th>
<th>Maximum Top Fiber Stress (N/mm²)</th>
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<td>Left</td>
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Mid Span Moments (kNm) 3086.97 3146.53

Case 1 Continuity Moment (kNm) DF Link Slab

|      |                           |           |              |                        |                                 |
| G3   |                           |           |              |                        |                                 |
| Left Span | Right Span | Left | Right |              |                        |                                 |
| 68.76 | -68.74                   | 0.0101   | -0.0101 | 47.21                  | 0.30                            | 1.77                            |
| Total | 12.34                    | 0.0018   | -0.0018 | 169.64                 | 0.00                            | 1.77                            |

Mid Span Moments 3609.24 3674.91

Case 2 Continuity Moment (kNm) DF Link Slab

|      |                           |           |              |                        |                                 |
| G3   |                           |           |              |                        |                                 |
| Left Span | Right Span | Left | Right |              |                        |                                 |
| 70.07 | -70.05                   | 0.0103   | -0.0103 | 47.49                  | 0.30                            | 1.78                            |
| Total | 17.21                    | 0.0025   | -0.0026 | 169.76                 | 0.00                            | 1.78                            |

Mid Span Moments 3601.12 3666.68

Case 3 Continuity Moment (kNm) DF Link Slab

|      |                           |           |              |                        |                                 |
| G3   |                           |           |              |                        |                                 |
| Left Span | Right Span | Left | Right |              |                        |                                 |
| 72.22 | -72.19                   | 0.0106   | -0.0106 | 47.65                  | 0.30                            | 1.78                            |
| Total | 25.65                    | 0.0038   | -0.0038 | 169.40                 | 0.01                            | 1.78                            |

Mid Span Moments 3586.01 3651.37

Case 4 Continuity Moment (kNm) DF Link Slab

|      |                           |           |              |                        |                                 |
| G3   |                           |           |              |                        |                                 |
| Left Span | Right Span | Left | Right |              |                        |                                 |
| 95.52 | -95.35                   | 0.0141   | -0.0140 | 46.10                  | 0.32                            | 1.74                            |
| Total | 118.19                   | 0.0174   | -0.0176 | 160.82                 | 0.02                            | 1.74                            |

Mid Span Moments 3452.45 3515.65

Case 5 Continuity Moment (kNm) DF Link Slab

|      |                           |           |              |                        |                                 |
| G3   |                           |           |              |                        |                                 |
| Left Span | Right Span | Left | Right |              |                        |                                 |
| 192.09 | -191.19                  | 0.0283   | -0.0282 | 40.45                  | 0.38                            | 1.61                            |
| Total | 478.01                   | 0.0704   | -0.0711 | 136.57                 | 0.10                            | 1.61                            |

Mid Span Moments 3132.21 3190.08

RHHR Continuity Moment (kNm) DF Link Slab

|      |                           |           |              |                        |                                 |
| G3   |                           |           |              |                        |                                 |
| Left Span | Right Span | Left | Right |              |                        |                                 |
| 1095.04 | -1074.21                 | 0.1613   | -0.1582 | 11.37                  | 1.18                            | 1.44                            |
| Total | 3128.01                  | 0.4607   | -0.4607 | 29.58                  | 0.82                            | 1.44                            |

Mid Span Moments 2062.08 2121.74
### Table B.49: Calculations for data extracted from Analysis of Bridge IV-S30-D0

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### Table B.51: Calculations for data extracted from Analysis of Bridge IV-S30-D50

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Table B.52: Calculations for data extracted from Analysis of Bridge IV-S45-D0
Table B.53: Calculations for data extracted from Analysis of Bridge IV-S45-D25

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### Table B.54: Calculations for data extracted from Analysis of Bridge IV-S45-D50

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VITA

Ram Naren Mothe was born in the city of Hyderabad, Andhra Pradesh, India. He earned his bachelor’s degree in Civil Engineering from College of Engineering, Osmania University, Hyderabad. He joined Louisiana State University for a master’s program in January 2005 and he will be graduating with the degree of Master of Science in Civil Engineering in December 2006.