The Development of a Composite Concrete Bridge System for Short-to-Medium-Span Bridges

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ABSTRACT

The inverted T-beam bridge system provides an accelerated bridge construction alternative for short-to-medium-span bridges. The system consists of adjacent precast inverted T-beams finished with a cast-in-place concrete topping. The system offers enhanced performance against reflective cracking, and reduces the likelihood of cracking due to time dependent effects. The effects of transverse bending due to concentrated wheel loads are investigated with respect to reflective cracking. Transverse bending moment are quantified and compared to transverse moment capacities provided by a combination of various cross-sectional shapes and transverse connections. A design methodology for transverse bending is suggested. Tensile stresses created due to time dependent and temperature effects are quantified at the cross-sectional and structure level and strategies for how to alleviate these tensile stresses are proposed. Because differential shrinkage is believed to be one of the causes of deck cracking in composite bridges, a study on shrinkage and creep properties of seven deck mixes is presented with the goal of identifying a mix whose long terms properties reduce the likelihood of deck cracking. The effects of differential shrinkage at a cross-sectional level are numerically demonstrated for a variety of composite bridge systems and the resistance of the inverted T-beam system against time dependent effects is highlighted. End stresses in the end zones of such a uniquely shaped precast element are investigated analytically in the vertical and horizontal planes. Existing design methods are evaluated and strut-and-tie models, calibrated to match the results of 3-D finite element analyses, are proposed as alternatives to existing methods to aid designers in sizing reinforcing in the end zones. Composite action between the precast beam and the cast-in-place topping is examined via a full scale test and the necessity of extended stirrups is explored. It is concluded that because of the large contact surface between the precast and cast-in-place elements, cohesion alone appears to provide the necessary horizontal shear strength to ensure full composite action. Live load distribution factors are quantified analytically and by performing four live loads tests. It is concluded that AASHTO’s method for cast-in-place slab span bridges can be conservatively used in design.
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Dedication

To Monika
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Chapter 1: Introduction
Introduction

1.1 Prefabricated Bridge Construction

Concrete has proven to be a durable, versatile material since the Romans first used it to build their aqueducts and so many other long-standing and iconic structures. Prestressed concrete has been used in European buildings and structures since the early 1900’s, but it wasn’t until the 1950s that prestressing and, later, precast concrete techniques became a significant influence in the American construction industry. The first true U.S. project to incorporate prestressed concrete components was the Walnut Lane Memorial Bridge in Philadelphia, Pa., which was built in 1950 with prestressed concrete girders. The concept was the brainchild of Professor Gustave Magnel of Belgium who developed the concept of prestressed concrete in the 1940s while at the University of Ghent. Ever since, bridge construction with prefabricated concrete components gained momentum and has become the norm for most modern highway bridges.

Prefabricated bridge construction typically consists of fabricating individual elements off-site and delivering them to the project site ready to be erected. This allows the concurrent production of the individual elements as opposed to cast-in-place concrete construction, in which the casting of a certain component can be done only if the supporting element is in place. The fabrication of elements off-site also eliminates the need to construct and remove formwork at the bridge site, work in close proximity to traffic, or operate in areas that are over water. The accelerated bridge construction offered by precast elements has been embraced by engineers and is perhaps one of the few methods that can go hand in hand with the demands of a fast paced modern life.

Similar to structural steel building and bridge construction, the fabrication of a concrete bridge structure in individual pieces raises the question of how these components with be connected. In prefabricated bridge construction, it is typically these connections that deteriorate over time and create the need for bridge rehabilitation or replacement. It is in this area that cast-in-place concrete construction has an advantage over prefabricated construction because it reduces the number of joints, which are the problematic areas, and it offers a higher degree of redundancy, which in some cases is desirable. The challenge that engineers face today is, how to design structures that consist of prefabricated elements but yet emulate monolithic construction. This has been the subject of many research programs and has fostered engineers worldwide to collaborate and share their experiences.

1.2 Scanning Tour

The Federal Highway Administration (FHWA) and the American Association of State Highway and Transportation Officials (AASHTO) initiated a scanning tour in April of 2004 to explore state-of-the-art technologies for rapid construction already being implemented in other industrialized countries. A team of eleven members (three representatives from FHWA, four representatives from state departments of transportation, one representative from county...
engineers, one university representative, and two representatives from industry) visited Japan, the Netherlands, Belgium, Germany, and France with the objective to identify international uses of prefabricated bridge elements and systems and to identify decision processes, design methodologies, construction techniques, costs, and maintenance and inspection issues associated with use of the technology. The team was interested in all aspects of design, construction, and maintenance of bridge systems composed of multiple elements that are fabricated and assembled off-site.

One of the systems identified in the scanning tour for implementation in US was the Poutre-Dalle system (Figure 1.1). This system was observed in France and poutre-dalle in French means beam-slab. As the name implies, the system consists of a series of adjacent precast inverted T-beams that serve as formwork for the cast-in-place topping. After the cast-in-place topping is placed the system behaves as a composite slab. It eliminates the need for installing formwork on site and provides a connection between the precast and cast-in-place components through the transverse hooked bars protruding from the webs of the precast inverted T-beam. The Poutre-Dalle system is intended for short-to-medium-span bridges with spans ranging from 20-65 feet. The motivation for the adoption of such a system is related to reflective cracking problems associated with traditional systems used for short-to-medium-span bridges. These traditional systems typically feature composite bridges constructed with adjacent precast voided slabs and adjacent box girders (Figure 1.2).

One of the causes that can lead to reflective cracking is the transverse bending of the bridge when subject to concentrated loads such a vehicular loads (Figure 1.3). The only resisting mechanism against interface bond failure if transverse post-tensioning is not applied in the adjacent box or voided slab system is primarily the tensile bond strength between the precast beams and the grout in the shear keys. The Poutre-Dalle system offers two improvements with respect to resistance against reflective cracking caused by transverse bending. First, it provides a thicker cast-in-place concrete topping over the longitudinal joints, and second, it offers a horizontal interface in addition to the vertical interface between the precast and cast-in-place components. The combination of these two interfaces emulates monolithic construction while preserving the benefits of prefabricated elements. In addition, the transverse hooked bars help arrest any potential cracks over the longitudinal joint or at the precast web cast-in-place topping interface.

Figure 1.1 Poutre-Dalle System

3
Inspired by the Poutre-Dalle systems observed in France, engineers in Minnesota developed a similar system, which featured the same precast inverted T-beam shape and the extended transverse bars. The 180° hook at the ends of the transverse bars was changed to a 90° hook as shown in Figure 1.4 (a). This was done to allow the placement of a “drop-in” reinforcing cage over the trough area to serve as additional reinforcing in the region above the longitudinal joint (Figure 1.4 (b)). This system was targeted for implementation in the state of Minnesota for bridges with spans ranging from 20 ft – 65 ft. The first two bridges built with this system are located in Center City, MN and Waskish Township, MN.

Over the course of seven years (2005-2012) researchers at the University of Minnesota investigated a variety of issues related to the design and construction of this new system. These issues included studies on reflective cracking, crack control reinforcing, composite action, transverse live load distribution, restraint moments, skew effects and end stresses at the end zones. This research was presented in a series of technical reports. Most of these reports were prepared for Minnesota Department of Transportation (MnDOT) \(^3,4,5,6,7\) and one of them for the National Cooperative Highway Research Program (NCHRP) \(^6\).
The inverted T-beam system developed in Minnesota was implemented on twelve bridges between 2005 and 2011. During this time the original concept underwent a number of modifications to improve performance in different design generations. To determine the effect of these design modifications on performance, a series of field inspections was done for five existing inverted T-beam bridges. Field inspections were conducted using two separate but related procedures: crack mapping and core examinations. Figure 1.5 shows a crack map and the locations were the cores were extracted for Bridge No. 33008 near Mora, MN. Cores 1 and 2 revealed a full depth reflective crack and a ½ in. deep from surface shrinkage crack, respectively. Cores 3 and 4 revealed a 5 ¼ in. deep reflective crack from joint and a 3 ½ in. deep shrinkage crack from surface. As can be seen the extracted cores could provide information on the depth of the cracks only at the extracted locations. Figure 1.5 suggests that the extent of longitudinal and transverse surface cracking is extensive. Although the inverted T-beam system showed promise with respect to addressing reflective cracking concerns compared to the traditional voided slab system, the fabrication challenges presented by the extended transverse bars and the surface cracking observed in Minnesota’s bridges prompted the need for additional research.
1.4 Objective and Scope of this Study

Being aware of reflective cracking problems present in short-to-medium-span bridges built with adjacent voided slabs and adjacent box girder systems, the Virginia Department of Transportation expressed interest in implementing the precast inverted T-beam system for the first time in Virginia. The application was a bridge replacement project near Richmond, VA on US 360 and featured four bridges (Figure 1.6). Three of these bridges were targeted to be replaced with the traditional adjacent voided slab system and one of them with the new inverted T-beam system. In addition, the bridge that was targeted for replacement using the inverted T-beam system (B607) was identical in terms of number of spans, span lengths, bridge width, traffic volume and environmental conditions with one of the neighboring bridges, which was scheduled to be replaced using the traditional adjacent voided slab system (B608). Both of these bridges were two span continuous bridges with span lengths approximately 43 ft. This provided an opportunity to observe the relative performances of these two bridges over time. The objective of this study is presented in the following frameworks:
1) Transverse Bending and Reflective Cracking:

The most pressing issue of interest to the Virginia Department of Transportation (VDOT) was that related to reflective cracking. The objective was to build on the Minnesota’s experience and investigate modifications to the inverted T-beam system that would lead to more durable, crack resistant and economical bridges. The scope of work to achieve this objective included performing a 3-D finite element analysis of the US 360 bridge to quantify transverse bending demands and testing a combination of various cross-sectional shapes and transverse connections that can provide adequate performance with respect to transverse bending and reflective cracking. The alternative cross-sectional shapes and transverse connections were developed with the purpose of emulating monolithic construction but without the need for transverse bars, which provide a challenge for the precaster during form installation and removal.

![Image of bridge replacements](image)

**Figure 1.6** Aerial view of the site featuring four bridge replacements.

2) Time Dependent and Temperature Effects

Other causes for longitudinal and transverse cracking are differential shrinkage between the cast-in-place topping and the precast beam, and temperature effects. As a result, the purpose of this study is to quantify time dependent and temperature effects in terms of stresses, restraint moment and restraint axial forces, and make recommendations for how to alleviate these effects at the cross-sectional and structure level.
Figure 1.7 Preliminary plan and elevation of the U.S.360 bridge over the Chickahominy River
The study on time dependent effects includes a numerical examination of the effects of differential shrinkage and shrinkage induced creep at the cross-sectional and structural level. Shrinkage induced creep is accounted for by using the age adjusted effective modulus method. The effects of uniform temperature changes were of interest to determine the magnitude of the axial restraint forces at the abutments when a two-span continuous bridge (such as the US 360 Bridge) is subject to a uniform decrease in temperature and axial contraction is not allowed. The study of temperature gradient effects includes the determination of self-equilibrating stresses at the cross-sectional level as well as restraint axial forces and restraint moments.

3) Reducing the likelihood of deck cracking by controlling shrinkage and creep properties

Because differential shrinkage is believed to be one of the causes of deck cracking in composite bridges, a study on shrinkage and creep properties of seven different deck mixes was carried out with the goal of identifying a mix whose long terms properties would reduce the likelihood of deck cracking. In addition, the objective of this study is to demonstrate the effects of differential shrinkage via a numeric example and compare the resistance of the inverted T-beam bridge system against these effects compared to other bridge systems used for short-to-medium-span bridges.

4) Stresses in the pre-tensioned anchorage zone

The shape of the precast inverted T-beam used in the state of Minnesota and the one proposed in this study presents a unique case when it comes to determining stresses created in the pre-tensioned anchorage zones. The objective of this study is to investigate end zone stresses in the vertical and horizontal planes by performing a series of 3-D finite element analyses and to evaluate the validity of existing design methods.

5) Composite Action

The unique interface between the precast inverted T-beams and the cast-in-place topping prompted the need to investigate the capability of the composite bridge to develop full composite action by relying solely on the cohesion between the two interfaces. As a result, the purpose of this study is to explore the necessity of extended stirrups by performing a full scale test and subjecting a typical composite bridge cross-section to service and strength level design forces. In addition, the goal is to explore whether the proposed composite bridge system can develop its nominal moment capacity without incurring any slip at the interface.

6) Live Load Distribution Factors

Because the inverted T-beam system is a new bridge system, the selection of appropriate live load distribution factors during design is of interest. The purpose of this study is to quantify live load distribution factors for moments and shear and to compare them with those calculated
based on AASHTO’s methods. The scope of work to achieve this objective includes performing a total of four live load tests and measuring longitudinal strains at various locations at each precast inverted T-beam and complementing the field work with additional finite element analyses.

1.4 Organization

Chapter 2 provides a literature review on bridge systems with precast components featuring transverse and longitudinal joints. A summary of the key findings presented in the research reports by University of Minnesota over the course of seven years (2005-2012) is provided.

Chapter 3 describes the investigation of the effects of transverse bending with respect to reflective cracking. The capability of various cross-sectional shapes and transverse connections to provide adequate performance with respect to the transverse bending demands in the US 360 Bridge is explored.

Chapter 4 presents the study on time dependent and temperature effects on composite bridges built with precast inverted T-beams by taking the U.S. 360 Bridge as an example. In this chapter the effects of differential shrinkage and shrinkage induced creep are quantified and recommendations for how to alleviate the negative effects are made. Also methods for quantifying the effects due to uniform temperature changes and temperature gradients are presented.

Chapter 5 describes an experimental study on seven deck mixes with the goal of identifying a mix whose shrinkage and creep properties help reduce the likelihood of excessive cracking in composite concrete bridge systems. A comparison of shrinkage and creep properties of the seven mixes is presented. Numeric examples are provided to illustrate the effects that shrinkage and creep properties of the deck have on long term stresses developed in composite bridge systems. The relative resistance against time dependent effects of three bridge system that can be used for short-to-medium span bridges is examined.

Chapter 6 presents a study on quantifying tensile stresses in the pre-tensioned anchorage zones of precast inverted T-beams. The effects of various methods for modeling the prestressing force are investigated. In addition, the effect of the presence of notches in the flanges of the precast inverted T-beam at the end zone is studied. Tensile stresses in the end zone are investigated in the vertical and horizontal planes. Strut and tie models are proposed as alternatives to existing methods for quantifying the tensile forces in the end zones.

Chapter 7 presents an experimental study on a full-scale composite beam with the purpose of investigating how a typical composite section in the US 360 Bridge will perform under service and strength level loads, and with the goal of determining whether a typical composite section can develop its nominal moment capacity without incurring any slip at the interface. The necessity of extended stirrups to ensure full composite action is explored.
Chapter 8 describes an investigation focused on quantifying appropriate live load distribution factors for the inverted T-beam system, which consists of four live load tests and finite element analyses conducted on Phase I of the US 360 Bridge.

Chapter 9 presents conclusions and recommendations.

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200 W. Adams St. #2100, Chicago, IL 60606
Chapter 2: Literature Review
Literature Review

2.1 Introduction

Prefabricated construction has become an essential part of modern lifestyle. In almost any metropolitan city, building and bridge construction almost always includes some type of prefabrication to compress the time required to erect the structure. Increases in standards of living have resulted in higher number of automobile purchases and consequently increased traffic. This coupled with increases in trade and higher competition has imposed higher traffic volumes on existing bridges. These trends have created the need for and expansion of the existing infrastructure including the construction of new bridges and rehabilitation of existing ones. All these demands are expected to be fulfilled quickly and without significantly interfering with the motoring public. As a result, prefabricated bridge construction has become almost indispensable. The primary challenge associated with prefabricated construction is selecting a system that can be built quickly, is safe, economical, durable and resistant to mechanical loads and environmental effects. While most prefabricated elements have not presented any significant problems at the component level, they suffer from problems associated with the deterioration of the joints. Significant research has been done and continues with the goal of coming up with joint and connection details between prefabricated elements that are durable and enhance the resistance of bridges against traffic and environmental effects.

Because the inverted T-beam system investigated in this study consists of longitudinal joints between adjacent precast members, a literature review of existing bridge systems built with prefabricated components and featuring transverse and longitudinal joints is presented. Even though each prefabricated system is unique, connection details used in one system are quite often applicable to a variety of other systems. Because the inverted T-beam system is intended to be used for single span as well as multi-span bridges, some traditional methods for providing continuity at the intermediate supports are presented. Finally, the development of the inverted T-beam system with tapered webs was inspired from the Poutre-Dalle system observed in France during the 2004 scanning tour. This system was implemented for the first time in the United States in the state of Minnesota on several bridges. As a result, a review of the work performed by the researchers at the University of Minnesota is presented and the motivation behind this study is articulated.

2.2 Precast Systems with Transverse Joints

An example of a bridge system built with prefabricated components is shown in Figure 2.1. This system consists of precast I-girders and full depth precast deck panels. Joints between the precast deck panels run in the transverse direction and feature a female-female detail which is grouted on site. Longitudinal post-tensioning can be applied to keep the joints intact prior to placing grout in the shear pockets to achieve composite action.
Another similar system that features joints in the transverse direction is shown in Figure 2.2 (a). In this system the precast concrete I-girders are replaced with steel I-girders. The shape of the precast deck has a corrugated profile to reduce the weight of each panel. The transverse joints feature a female-female detail and are filled with rapid-set non-shrink grout (Figure 2.2 (b)). An elevation of the deck panels featuring the application of longitudinal post-tensioning is shown in Figure 2.2 (c). Both systems illustrated in Figures 2.1 and 2.2 consist of full depth precast panels which may or may not receive an overlay. The success of these systems is related to the performance of grouted joints. In the system described in Figure 2.1 the grouted joints that would be exposed to traffic if an overlay is omitted include the transverse joint between the precast panels as well as the grouted shear pockets. Whereas in the system described in Figure 2.2 only the transverse joints would be exposed to traffic in cases excluding an overlay. Typically, in these systems the operations performed on site include the grouting of the transverse joints, application of longitudinal post-tensioning, grouting of the shear pockets, casting of a continuity and end diaphragm (when applicable), connection to the substructure (when applicable) and casting of vehicle barriers.

To reduce the number of joints exposed to traffic other systems that use partial depth precast panels as stay-in-place forms for the cast in place deck have been proposed (Figure 2.3). In this system the cast-in-place deck provides a smooth riding surface, offers redundancy and emulates monolithic construction. The onsite activities are slightly prolonged because of the larger amount of cast-in-place concrete and the relatively longer time required to cure it (compared to rapid-set non-shrink grouts).
Figure 2.2 System with transverse joints (a) Full Depth Precast Prestressed Concrete Bridge Deck system, (b) Transverse joint, (c) Elevation view of longitudinal post-tensioning in a Full-Depth Precast Prestressed Bridge Deck System (dimensions are in mm)
Other systems with prefabricated components include those featuring joints in the longitudinal direction. One such system is the decked bulbed T system shown in Figure 2.4. In this system the girder and the corresponding portion of the deck are cast monolithically at the precast yard. These composite assemblies are then placed adjacent to one another on site. The connection between adjacent decked bulb T’s can feature a female-female grouted key and welded connections at some spacing (Figure 2.5). The welded connection consists of a steel plate welded to steel angles which are located in recessed pockets in the precast components. Each steel angle is welded to reinforcing bars embedded in the flange of the precast component in the transverse direction. The recessed pockets are then grouted for protection against corrosion. This type of detail provides a mechanical connection in the transverse direction helps in the transverse distribution of live loads. In cases where differential camber exists between adjacent members adjustments need to be made to perform the transverse connection. In cases where the placement of an overlay is omitted, the longitudinal joints will be exposed to traffic.

Figure 2.4 Decked Bulbed T concrete bridge (DBT) being constructed
Another system that is similar to the decked bulb T system is shown in Figure 2.6. This system was observed in Germany during the scanning tour performed in 2004. The prefabricated composite assembly in this case consists of steel girders and partial depth precast panels. When the composite assembly is erected on site the partial depth precast panels abut against each other and serve as stay in place forms for the cast-in-place deck. Composite action between the partial depth precast panels and the cast-in-place deck is achieved by using reinforcing bars that extend from the top surface of the precast panels. Welded studs on the top flange of the beam provide composite action between the steel girders the precast panel and the cast-in-place deck.

**Figure 2.5** A typical DBT bridge connected by longitudinal joints with welded steel connectors

**Figure 2.6** System with longitudinal joints - Partial Depth Concrete Decks Prefabricated on Steel or Concrete Beams
Figure 2.7 shows another system which features adjacent box girders post-tensioned in the transverse direction. Adjacent box girders are connected in the transverse direction by grouted shear keys. Figure 2.8 illustrates the typical geometry of a grouted shear key in such a system. Typically, a 2 in. thick wearing surface is used to provide a smooth riding surface, however, in some cases; a 5 in. to 6 in. structurally composite concrete overlay is used\textsuperscript{6}. Adjacent box beams have been widely used in the United States for the construction of new bridges as well as for the replacement of old bridges\textsuperscript{8}. The National Bridge Inventory shows that box beams represented about one-third of all prestressed concrete bridges constructed in the United States between 1979 and 1989\textsuperscript{9}. Box beams can span up to 100 ft while maintaining a high span-to-depth ratio\textsuperscript{10}. Bridges built with adjacent box beams have been reported to experience longitudinal reflective cracks in the topping directly over the shear keys joints\textsuperscript{8}. It is believed that these longitudinal reflective cracks are usually caused by the large torsional stiffness of the adjacent boxes and by the lack of an adequate transverse connection to account for the significant forces needed to be transferred between the boxes\textsuperscript{8}.

El-Remaly et al.\textsuperscript{6} have proposed a diaphragm and transverse post-tensioning system to account for these forces, however, the relatively large amount of transverse post-tensioning required to restrain the torsionally stiff boxes may significantly increase the cost of the system\textsuperscript{8}. This was the primary motivation that led to the adoption and development of the precast inverted T-beam, which is described later in this chapter.

**Figure 2.7** System with Longitudinal Joints - Transverse cross-section of adjacent box girder bridge with transverse post-tensioning\textsuperscript{6}
Kamel and Tadros\textsuperscript{11} proposed a system that consists of adjacent precast inverted T-beams covered with a cast-in-place topping for spans up to 100 ft (Figure 2.9 (a)). The system has a span-to-depth ratio of up to 35 and emerged after an extensive literature search and national survey of producers and bridge designers. It is suggested that the system is ideal for applications in which clearance is critical such as bridges crossing waterways and railroads. It was found that similar cross-sections were used in England, Germany, the Netherlands, Israel and other countries and that a similar bridge sections was used in Ohio in the 1950’s. The precast inverted T-beam section was made possible by a change in the sixteenth edition of AASHTO Standard Specifications\textsuperscript{12}. Figure 2.9 (b) shows reinforcing details for the precast inverted T-beam section. It has been shown that the compressive concrete stress at service load at mid-span top fibers and member deflection are the two criteria that often control the design of this system\textsuperscript{11}. Forms for the cast-in-place deck are left in place and can consist of either expanded polystyrene blocks (EPS) (as shown in Figure 2.9 (a)) or ½ in cement boards.
Badie et al.\textsuperscript{8} proposed a trapezoidal box beam system that can be used for spans up to 100 ft, has a span-to-depth ratio, ranging between 30 and 40, which makes it suitable for low clearance sites. The beam was presented in two different shapes: a closed totally precast concrete shape (Figure 2.10) and an open-top shape requiring a cast-in-place concrete topping (Figure 2.11)\textsuperscript{8}. 

\textbf{Figure 2.9} System with Longitudinal Joints - The shallow inverted T–beam system for rural areas\textsuperscript{11}, (a) Transverse Cross-section, (b) Reinforcing Details

![Diagram of system with longitudinal joints](image-url)
To provide continuity in the transverse direction two types connection mechanisms are used. The first is a continuous shear key between adjacent girders, and the second consists of discrete blockouts in the precast flanges spaced at 23.4 in. on center. Details of the shear keys are provided in Figure 2.10 detail B and Figure 2.11 detail A. Figure 2.12 illustrates three types of discrete connections between adjacent precast flanges. Connection Type I is similar to the one used for the decked bulb T system shown in Figure 2.4. Because of the smaller thickness of the
precast flange in the open top system, Connection Type I and Connection Type II can only be used with the closed box system, whereas Connection Type III can be used with either system.

Figure 2.12 Transverse connection details

Although the Poutre-Dalle system was identified as a new system during the scanning tour in 2004, there already were existing systems in use in the United States that resembled the Poutre-Dalle system. One such system is the Full-Span Form Panel Bridge System. The concept is identical. Flat precast adjacent panels span from support to support and serve as stay-in-place formwork for the cast-in-place topping. Figure 2.13 (a) shows an elevation of a two-span two-beam laboratory specimen used in a study performed by Hay Jr. et al. Figure 2.13 (b) shows a
transverse cross-section of this laboratory specimen featuring flat precast panels. Figure 2.13 (c) shows an alternative transverse cross-section, which is almost identical in concept with the Poutre-Dalle system. The primary difference is that the detail shown in Figure 2.13 (c) does not have any transverse bars protruding from the webs of the precast inverted T-beams, which was the case in the Poutre-Dalle system. As will be later shown in Chapter 3, this detail is very similar to one of the details investigated during the course of this study. Figure 2.13 (d) shows the recommended cross-section, which features the precast inverted T-beams, and the fact that the center portion of the precast can be either flat or ribbed to reduce the weight of the panel. Advantages associated with ribbing the center portion of the precast panel besides reducing its weight include the creation of a shear key mechanism that helps in the development of composite action. Disadvantages associated with creating these ribs include the reduction in the shear and flexural capacity of the precast section when the cast-in-place topping is placed.

The goal of the study performed by Hays Jr. et al.\textsuperscript{13} was to develop details that would reduce the development of reflective cracking that were almost always observed within the cast in place topping above the longitudinal joints of existing full-span form panel bridges at the time. The researchers hoped to accomplish this through a research program that consisted of four phases: a field survey on the condition of existing bridges, analytical modeling, laboratory testing, and field testing on an existing bridge\textsuperscript{14}.

The field survey revealed that all the nine bridges visited exhibited extensive reflective cracking in the cast-in-place topping in the region above the joint, however, the intensity and frequency of these cracks varied from bridge to bridge. Interestingly, one of the visited bridges, the Sampson River Bridge, exhibited several major longitudinal cracks prior to being open to traffic. This provided evidence that shrinkage played a role in the development of these cracks. Other bridges built with more conventional techniques such as cast-in-place (CIP) concrete deck slabs, and girders with either a CIP deck or precast panels oriented in the transverse direction were also visited to draw a comparison with the full-span form panel system. It was observed that while all these bridges exhibited cracking, the extent of cracking appeared to less extensive and the pattern much more random.

Three laboratory specimens were constructed and tested. The width of the specimens consisted of two adjacent precast panels. Loads were applied at mid-span of one of the panels on each span. It was concluded that no loss of bond between the precast panels and the cast-in-place topping was observed. However, a minimum amount of shear reinforcing was recommended to provide a factor of safety. Field and finite element studies revealed that stresses developed due to the shrinkage of the cast-in-place topping can be high enough to cause cracking.
Figure 2.13 System with Longitudinal Joints - Study performed by Hays, Jr. et al.\textsuperscript{13}, (a) Elevation, (b) Standard cross-section 1, (c) Alternative cross-section 2, (d) Recommended cross-section
As a result, Hays Jr. et al.\textsuperscript{13} recommended that transverse reinforcing consisting of No. 4 at 12 in. on center and a minimum concrete topping thickness of 4.5 in. be used. It was believed that this detail will help alleviate some of the shrinkage cracking observed in the field and improve the load transfer between adjacent panels. In addition, the alternative detail shown in Figures 2.13 (c) and (d) was favored over the flat panel detail with regards to longitudinal cracking and load transfer. It was also recommended that positive moment reinforcement be provided over the piers.

Shortly after the study performed by Hays Jr. et al.\textsuperscript{13} Buckner and Turner\textsuperscript{15} investigated the effects of repetitive loading on the serviceability and strength of full-span form panel bridge system. Six single-span simply supported bridge decks specimens were loaded repetitively with 2,000,000 cycles of design load followed by a test to failure. Each specimen was constructed of three precast panels as shown in Figure 2.14 (a) and (b). The width of each panel was 3 ft – 5.5 in., which resulted in overall specimen width of 10 ft -5 in. The overall thickness if each specimen was 13 in. The performance of each tested specimen was evaluated primarily based on flexural rigidity, differential deflection between panels, and the strength and ductility of the composite system. The specimens were also inspected for visible cracks, prestressing strand slip, and strains in the transverse steel. The objective of the study was to develop recommendations for minimum cast-in-place concrete topping thickness, a minimum quantity of transverse reinforcement, and a preferred type of joint (flat or beveled) between the panels. Figures 2.14 (a) and (b) show transverse cross-sections of the tested specimens with flat panels and beveled edges, respectively.

The study concluded that the composite section created by the precast panels and the cast-in-place topping could withstand 2,000,000 cycles of design load without any significant loss in serviceability and strength. Simply roughening the top surface of the precast panels was adequate to achieve composite action. In contrast to the conclusions drawn Hays Jr. et al., the researchers concluded that adequate serviceability and strength could be obtained using flat precast panels rather than the more expensive beveled-edge panels. In addition, the researchers report that there was no indication that the relative thickness of the cast-in-place topping to the total thickness of the composite section had any effect on the fatigue strength of the section. The researchers recommended the use of a 5 in. topping and No.4 at 12 in. on center to provide adequate shear transfer strength.
Figure 2.14 Study performed by Buckner and Turner\textsuperscript{15}, (a) Transverse cross-section with flat panel, (b) Transverse cross-section with beveled edge, (c) Loading Apparatus

In the mid 90’s Peterman and Ramirez\textsuperscript{16,17} performed another study on this system to better understand its behavior and to investigate its strength. The advent of high performance concrete, better construction practices and increased quality control made this type of construction more attractive and provided the motivation for this study. Two full-scale specimens were tested and subjected to 5,000,000 cycles of service loading and then tested to
failure. The objectives of this study were to investigate the effects of repetitive loading on the continuity over the piers and to evaluate the ultimate strength of multi-span composite bridges built with this system. Each laboratory specimen consisted of two span and two panels. Figure 2.15 shows a transverse cross-section of one of the test specimens investigated in this study. Precast panels were 21 ft long, 4 ft wide, and 6 in. thick and were topped with a 6 in. thick cast-in-place concrete topping. Boundary conditions consisted of rollers at the exterior supports and a pin connection at the interior support. Based on the recommendation by Buckner and Turner\textsuperscript{15} the top of the precast panels was raked to attain composite action.

The study concluded that the continuity between adjacent spans was not affected by 5,000,000 cycles of repeated service loading in which the calculated reinforcement stress range at the pier exceeded the AASHTO allowable design stress range by 20%. The experimentally determined positive and negative moments in both laboratory specimens at failure exceeded the AASHTO design nominal moment capacities for the composite sections by factors of 1.7 and 1.6 for Specimens No.1 and No.2, respectively. This was primarily due to strain hardening of the mild steel reinforcement. In addition, it was concluded that full composite action could be obtained by applying a raked finish to the surface of the precast panels and that the ultimate load carrying capacity of the test bridges was not affected by time-dependent effects.

\begin{figure}[h]
\centering
\includegraphics[width=\textwidth]{figure2_15}
\caption{Cross-section of precast panels used in the study performed by Peterman and Ramirez\textsuperscript{16,17} (Panels are 4 ft wide and 21 ft long)}
\end{figure}
2.4 Continuity over the Piers

Many multi-span bridges constructed with prefabricated elements are made continuous to make the structure more economical and to eliminate joints, which are a costly maintenance item. Traditional methods for creating continuity include placing a cast-in-place concrete deck (Figure 2.16 (a)) or by applying full-length longitudinal post-tensioning (Figure 2.16 (b)). The former is the most frequent method for establishing continuity in precast concrete girder bridges and works by placing conventional reinforcement in the cast-in-place deck at negative moment regions. In this system the precast girder acts as a simply supported beam and resists its self-weight, the weight of the deck and construction loads, which comprise the majority of the loads in the bridge system. The system is only continuous under the effects of superimposed dead and live loads. This method is the simplest and, perhaps, the least costly of existing methods because it requires no specialty contractor.

Full-length longitudinal post-tensioning requires the placement of a continuity diaphragm before longitudinal post-tensioning is applied and the rest of the deck is placed. In this case the precast girders act as simply supported beams under their self-weight and the system is made continuous for the effects of deck weight and superimposed dead and live loads. The likelihood of cracking in the deck can be reduced by installing post-tensioning tendons that are stressed after the deck is placed and cured. The number of pre-tensioned strands is reduced because they are only required to control stresses due to the self-weight of the girder. This reduced pre-tensioning results in less camber and less demand for high strength concrete at release. This continuity method provides greater resistance to stresses and allows for longer spans for a given girder size than the conventional deck reinforcement continuity method. A disadvantage of this method is that it requires full length ducts and necessitates the widening of the girder webs in addition to providing end blocks to resist stresses at the anchorage zones. End blocks are required by code to be as wide as one of the two flanges and as long as ¾ of the girder depth. They not only add to the girder weight, but also require expensive alterations of the girder formwork and a specialty contractor to perform the post-tensioning and grouting.

Saleh et al. developed two new methods for creating continuity in bridges with precast concrete girders. These make the system continuous prior to the placement of the cast-in-place deck. One of these methods uses high-strength threaded rods in the top of the girder at negative moment regions and the other uses pre-tensioned top strands. The former is illustrated in Figure 2.17 and the latter in Figure 2.18. The method with pre-tensioned top strands requires the top strands to be long enough to allow for field cutting and staggered splicing. The top strands are mechanically spliced using contiguous hardware that allows for slack recovery. Examples of strand splicing with slack recovery hardware are provided in Figure 2.19.

Any of the methods described in this section is a plausible method for the inverted T-beam system. The method with conventional deck reinforcement and the one high-strength threaded rods are the simplest methods with the latter providing continuity for not only superimposed dead and live loads but also for deck weight and construction loads. The simplicity of these methods comes at the cost of not providing a pre-compression force in the negative
moment regions. The pre-tensioned strand continuity methods provides all the advantages of full-length post-tensioning but costs less due to the elimination of the high costs associated with full-length post-tensioning\textsuperscript{18}. This method results in truly continuous pre-tensioned girders and eliminates the need for end blocks and web widening required in full-length post-tensioning\textsuperscript{18}.

![Diagram](image1.png)

**Figure 2.16** Traditional methods for creating continuity, (a) Mild steel in diaphragm and deck, (b) Longitudinal post-tensioning\textsuperscript{18}
Figure 2.17 High strength threaded bar splicing (Note that other reinforcement and the deck slab are not shown for clarity)\textsuperscript{18}
Figure 2.18 Pre-tensioned strand splicing (Note that other reinforcing and deck concrete are not shown for clarity)
2.5 Precast Inverted T-Beam System

2.5.1 Introduction

Recognizing the need for accelerated bridge construction techniques, FHWA and AASHTO funded a scanning tour in 2004 in several countries in Europe and Japan with the purpose of identifying innovative bridge systems that utilize prefabricated elements. One of the systems identified during this scanning tour was the Poutre-Dalle system observed in France (Figure 2.20). The state of Minnesota was the first state in the United States that implemented this concept. The objective was to develop a durable bridge superstructure alternative to the traditional CIP slab-span bridge system that takes advantage of the benefits associated with prefabricated construction. The traditional CIP slab-span system is used for short-to-medium-span bridges and requires shoring during the placement of slab concrete, which increases construction time and often has a negative effect on the environment around the bridge. The advantage of the inverted T precast slab system is that it eliminates the need for shoring by utilizing the precast inverted T-beams as stay-in-place forms.
Inspired by the original Poutre-Dalle concept the system implemented in the state of Minnesota was developed in a collaborative process between MnDOT engineers, the University of Minnesota, and local fabricators for spans ranging between 20 and 65 ft$^{14}$.

**Figure 2.20** Poutre-Dalle System$^5$

Figure 2.21 shows the cross-section of a typical interior precast inverted T-beam. Bell et al.$^{14}$ report that the width of the section was chosen to be 6 ft, which is the maximum width that local fabricators could readily accommodate. This maximum width was chosen to minimize the number of sections required to span the width of the bridge. Using wider precast sections results in fewer longitudinal joints in the finished structure, which require special reinforcement and time to construct$^{14}$.

**Figure 2.21** Cross-section of interior precast Inverted T-beam$^{14}$
The exterior precast section features a flange on one side and a flat web on the rest of the section with extended vertical reinforcing to accommodate a barrier (Figure 2.23). The fact that the exterior sections did not have a standard width, allows the system to be used for bridges of any desired width. Because the exterior precast section is un-symmetric attention should be paid to properly account for the potential of having biaxial moments as a result of the prestressing force. An alternative detail is to replace the un-symmetric exterior precast section with a rectangular section of the required width. This was the detail implemented in the design of the US 360 Bridge.

![Diagram of Transverse Reinforcement for CIP Concrete Rail](image)

**Figure 2.22** Cross-section of exterior precast Inverted T-beam

Transverse reinforcing protruding from the webs of the precast sections was used to connect adjacent precast beams and to ensure monolithic action with the cast-in-place topping. Unlike the Poutre-Dalle system, the shape of these transverse hooks featured a 90° hook rather than an 180° hook (Figure 2.23). This was done to allow the installation of a drop-in reinforcing cage on site prior to the placement of the cast-in-place topping. This drop-in cage was intended to provide additional resistance against potential cracks over the longitudinal joint (Figure 2.24).

![Photograph of the precast bars in the precast inverted T-beam](image)

**Figure 2.23** Photograph of the precast bars in the precast inverted T-beam
Because this system is similar to slab-span bridges the overall depth of the composite section was approximated using AASHTO\textsuperscript{20} Table 2.5.2.6.3-1, which for slabs-pan bridges with continuous spans provides the following equation\textsuperscript{14}, where $S$ is the clear length (ft) of the longest span and the total superstructure depth is obtained in feet.

$$Total\ Structure\ Depth = \frac{[S + 10]}{30} \quad Eq. \ (1)$$

Bell et al.\textsuperscript{14} report that the thickness of the cast-in-place topping over the precast web was set to 6 in. to provide 3 in. of cover above the longitudinal deck reinforcement, therefore the height of the precast section would be 6 in. less than the value obtained from Eq. 1. Precast sections were designed as simply supported for dead load and continuous over the piers for live loads. The maximum span for which this bridge system could be economically used was estimated to be 65 ft.

Bell et al.\textsuperscript{14} state that precast flanges were designed with a 1:24 slope to increase the constructability of the section. In addition, it was felt that eliminating the flat surface on top of the flanges would simplify the removal of the formwork, as well as simplify the casting of both precast and CIP concrete by facilitating the flow of concrete\textsuperscript{14}. It should be noted that the presence of extended transverse reinforcing creates a double challenge during the fabrication of the precast inverted T-beams. Firstly, the side forms need to accommodate the protruding bars, and secondly, the protruding bars make the removal of flange forms difficult as noted by Bell et al.\textsuperscript{14}. Addressing this difficulty was one of the motivations behind the study presented in this investigation (See Chapter 3). The thickness of the precast flange used in the first inverted T-beam bridges in the state of Minnesota was 5.24 in. This was the minimum flange thickness that allowed for a No.4 bar with an 180° hook around the longitudinal reinforcement and 1.5 in. of cover at the bottom surface of the precast section (Figure 2.21). There was no concrete cover...
requirement at the top of the precast flange because the flanges were covered with CIP concrete in the field, however a minimal amount cover was required to ensure proper bound between precast concrete and steel\textsuperscript{14}. Minimizing the thickness of the precast flange was desired because a thinner flange increases the thickness of the cast-in-place topping and lowers the location of transverse reinforcing, both of which help better distribute loads in the transverse direction. The web and flange corners had a $\frac{3}{4}$ in. chamfer (Figure 2.21), which was intended to reduce stress concentrations at these locations and lessen the probability of the development of reflective cracking\textsuperscript{14}.

All precast surfaces in contact with the CIP topping were intentionally roughened to enhance the bond and help achieve composite action. These surfaces were roughened to a $\frac{1}{4}$ in. amplitude and the roughness at the side of the webs and top of the flanges was created by using a form liner as shown in Figure 2.25. A rake was used to roughen the top of the web. During the casting process of the inverted T-sections, styrofoam blocks were used to block out the final 10 in. of each flange at both ends of the member (Figure 2.26)\textsuperscript{14}. After the precast sections were in place, these block outs were filled with CIP concrete, which served, along with reinforcement, to tie the precast sections to the pier caps\textsuperscript{14}.

![Photograph of the formwork with indented inner surface](image)

**Figure 2.25** Photograph of the formwork with indented inner surface\textsuperscript{14}

Figure 2.27 shows the reinforcing in the trough over the longitudinal joint. This reinforcing consists of transverse bars protruding from the precast webs, which are lapped when adjacent precast sections are installed and a drop-in cage, which is installed in the field (Figure 2.27). The purpose of the reinforcing in the trough area was twofold. Firstly, it was intended to help distribute loads in the transverse direction and emulate monolithic action, and secondly, it
was intended to arrest the development of reflective cracking in the area above the longitudinal joint.

Continuity for live loads was created by adding reinforcement at the top of the CIP deck above the pier caps. This method of achieving continuity was much more practical than the use of longitudinal post-tensioning.\textsuperscript{14} In addition, the CIP topping that filled the blocked out areas of the precast sections as well as at the gap between the ends of the precast sections acted similar to a concrete diaphragm. This diaphragm would resist compressive forces developed as a result of negative moments over the piers and the longitudinal reinforcing at the top of the deck would resist the corresponding tensile forces. Furthermore, time dependent effects, such as shrinkage and creep can lead to the development of negative and positive restraint moments over the piers. As a result a positive moment connection was provided by adding longitudinal bars in the bottom of the trough area over the piers (Figure 2.24).

\textbf{Figure 2.26} Plan view of precast section layout showing blocked-out portions of the precast section flanges\textsuperscript{14}

\textbf{Figure 2.27} Photograph of the reinforcement cage installed above the precast longitudinal Joint\textsuperscript{14}
One of the first bridges built with the precast inverted T-beam section in Minnesota was the bridge near Center City, MN. Figure 2.28 and Figure 2.29 show the connection detail used in this bridge between the cast-in-place portion of the deck, the abutment and the approach slab, and the connection detail between the pier caps to the cast-in-place portion of the deck, respectively. The reinforcing dowels were only located within the center 50% of the bridge width to allow the outer portions of the bridge to expand and contract laterally under thermal loading.\textsuperscript{14}

**Figure 2.28** Connection detail at abutment, bridge deck, and approach panel\textsuperscript{14}

**Figure 2.29** Connection detail at pier cap\textsuperscript{14}
2.5.2 Transverse Bending and Reflective Cracking

Many bridge systems that consist of precast and cast-in-place components promoting faster construction schedules, suffer from problems associated with the performance of joints between adjacent members. One of the causes of deterioration of such composite systems is reflective cracking, which occurs at the interface of adjacent voided slabs or box girders. In both of these systems (adjacent box girders and voided slabs), typically a cast-in-place topping or overlay is used to provide a continuous riding surface. Grouted shear keys are typically used at the joint between the precast members and are intended to prevent the adjacent components from moving relative to one another. However, often these grouted shear keys fail and monolithic action in the transverse direction is lost, which gives rise to reflective cracking.

Reflective cracking provides an avenue for water and deicing salts to penetrate the topping and corrode the reinforcing steel. In addition, reflective cracking reduces redundancy and increases live load distribution factors. Post-tensioning the voided slabs or adjacent box girders in the transverse direction helps resist this behavior but it also introduces an operation that needs to be carried out in the field, which reduces the advantages associated with accelerating bridge construction. Using a thicker topping or overlay also helps but it is not the most economical solution to address reflective cracking.

The precast inverted T-beam system addresses the issue of reflective cracking by providing a thicker cast-in-place topping over the joints and by creating a horizontal interface in addition to the vertical interface between the precast and cast-in-place components. Figure 2.30 shows potential locations for the development of reflective cracking in bridges with precast inverted T-beams.

![Diagram of reflective cracking in MnDOT Precast Concrete Slab Span System (PSSS)](image)

**Figure 2.30** Anticipated locations of reflective cracking in MnDOT Precast Concrete Slab Span System (PSSS)

Because the bridge at Center City was one of the first bridges built with precast Inverted T-beams, researchers at University of Minnesota were contracted by MnDOT to instrument the
bridge to validate some of the assumptions used for the design of the system and to monitor the performance of the completed structure under traffic. One of the objectives of monitoring was to check for reflective cracking in the region between adjacent inverted T-beams. To accomplish this, Bell et al.\textsuperscript{14} installed vibrating wire gages over the longitudinal joints and above the web corners of the precast inverted T-beams to capture any indication of reflective cracking. Data collected during the period of September 2005 and June 2006 indicated that no cracking had developed within the cast-in-place topping.

Monitoring of the Center City Bridge continued and Smith et al.\textsuperscript{21,22} reported that longitudinal cracking had initiated in the cast-in-place concrete at mid-span where some of the transverse gages were located. This cracking was believed to be a result of restrained shrinkage and environmental effects rather than due to vehicular loads. In addition to the monitoring of the Center City Bridge, Smith et al.\textsuperscript{21,22} constructed and load tested a two-span laboratory specimen to investigate the effects of various parameters including variations in flange thickness and roughness on the behavior of the system including the potential for the development of reflective cracking. The Center City bridge was built with a flange thickness of 5 ¼ in. to fit confinement steel with an 180° hook around the flange. The same flange thickness was used in Span 2 of the laboratory specimen. Test results indicated that the reduction in flange thickness to 3 in. improved transverse load distribution and that there was no indication that the reduced flange was not durable enough for transportation and construction. Therefore it was recommended to use the 3 in. flange in future implementations of the MnDOT PCSSS\textsuperscript{21,22}.

Because the results from initial field implementation projects were relatively favorable, six more bridges were planned and constructed in Minnesota with the intent to improve the system\textsuperscript{23}. Dimaculangan and Lesch\textsuperscript{23} report that three “2nd generation bridges” were designed and built in 2007. In 2009, three additional “3rd generation bridges” were designed and constructed that included a large number of modifications with the hopes of reducing cracking. To phase in the design changes only a few of the proposed changes were incorporated in each generation of bridges\textsuperscript{23}. The most significant change was to make the bottom flange thinner based on the recommendation by Smith et al.\textsuperscript{21,22}. Other notable modifications reported by Dimaculangan and Lesch\textsuperscript{23} that are related to reflective cracking and transverse bending include:

- Increasing the chamfer sizes on the edges of the beams
- Increasing the transverse deck reinforcement bars using closer spacing
- Assuring the direct placement of the drop-in reinforcement cage such that it is staggered with the bars protruding from the beams
- Moistening the precast beams before placement of the CIP deck
- Adding welded wire reinforcement to the longitudinal joint between the beams

French et al.\textsuperscript{4} continued the research on the inverted T-beam systems as part of an NCHRP study titled “Cast-in-Place Concrete Connections for Precast Deck Systems”. The NCHRP study included large-scale laboratory tests on two-span and simple-span inverted T-
beam bridges (Figure 2.31), as well as seven sub-assemblage tests (Figure 2.32). The two-span and simple-span large scale bridge specimens provided an opportunity to investigate different cross-sectional details and associated aspects of bridge behavior including the effect of flange thickness and type and quantity of transverse joint reinforcement on reflective crack control\(^4\). Both bridge specimens were subject to cyclic loading to observe their fatigue performance in their un-cracked and cracked states. The tire patch loading used to simulate tire traffic created strains that were much lower than the ones observed in the Center City Bridge. As a result reflective cracking was intentionally induced to monitor the behavior of the cracked system. Both laboratory bridge specimens performed adequately in controlling cracking in the longitudinal joint region throughout loading that simulated traffic and environmental effects\(^4\). Spans with thinner flanges exhibited less degradation as a result of repeated loading compared to spans with thicker flanges.

**Figure 2.31** Load placement during transverse load distribution tests for Concept 1 and Concept 2 laboratory specimens\(^4\).
In addition, the seven sub-assemblages provided an opportunity to investigate variations in crack control reinforcement across the longitudinal joint between the precast flanges. Figure 2.32 shows the initial test setup for the sub-assemblage specimens. The NCHRP report states that: “It was found during testing of the first specimen, that the stiff flanges of the precast section rotated and caused delamination between the precast flange and CIP concrete, resulting in propagation of cracks at the precast-CIP concrete interface. The test setup was modified to clamp the precast flanges to the CIP concrete a distance of approximately 1.25 in. from the longitudinal joint in both directions. The test setup with the clamping system was believed to more realistically emulate the field conditions because in a bridge system, the pier supports are normal to the longitudinal joint and would constrain the relative rotation between the precast flange tips and the CIP at the end of the span.” The modified test setup is shown in Figure 2.33.

The depth of precast members was 12 in. to match that used in the bridge specimens, whereas the depth of the deck was minimized to reduce the transverse cracking moment of the specimens so that yielding of the transverse reinforcement immediately at cracking could be avoided. Details for sub-assemblage specimens are provided in Table 2.1.

According to the report, each of the sub-assemblage specimens performed adequately throughout the range of loading, though variations in the extent of cracking indicated some relative differences. The specimens with the largest reinforcement ratios performed better than the rest of the specimens by exhibiting smaller crack widths. The presence of a smooth flange surface was studied with the expectation that it would distribute the transverse tensile stresses throughout the bottom of the cast-in-place concrete, thereby reducing the potential stress concentration immediately above the longitudinal joint. However, even with the presence of a smooth flange it was determined that a single crack was present and that the transverse tensile stress was not distributed adequately well. In addition, it was concluded that a smooth flange would not be desirable as it would likely promote delamination of the horizontal precast flange cast-in-place concrete interface. While a 9 in. maximum transverse spacing appeared to be sufficient to control cracking in the sub-assemblage study it did not correlate to an improvement in crack control relative to the 12 in. transverse spacing in the simply-supported laboratory bridge specimen.

The NCHRP study culminated in recommended design and construction specifications for the inverted T-beam system. The design recommendations related to reflective cracking included equations for determining the amount and spacing of transverse load distribution reinforcing and reflective crack control reinforcing. Transverse load distribution reinforcing was considered to include the transverse hooked bars that extend through the precast webs and lap with companion hooked bars protruding from the adjacent precast webs. It is recommended that only one of the hooked bars in the lap be used in transverse load distribution reinforcement calculation because the lapped bars must transfer load to each other between the adjacent panels.
Figure 2.32 Elevation view of sub-assemblage specimens

Figure 2.33 Clamping system developed to simulate restraint near joint region on subassemblage specimens
Table 2.1 Sub-assemblage specimen design details

<table>
<thead>
<tr>
<th>Specimen Identification</th>
<th>Width [in]</th>
<th>Depth [in]</th>
<th>Transverse Bars (Load Trans)</th>
<th>Cage (Crack Control)</th>
<th>Max Spacing</th>
<th>R/T Ratio</th>
</tr>
</thead>
<tbody>
<tr>
<td>SSMBLGI-Control1</td>
<td>62.75</td>
<td>24</td>
<td>#4 18 in. OC 4 1/2 in.</td>
<td>Cage #3</td>
<td>18 in. Oc 4</td>
<td>9</td>
</tr>
<tr>
<td>SSMBLGI2-NoCage</td>
<td>67.25</td>
<td>34</td>
<td>#4 18 in. OC 4 1/2 in.</td>
<td>No Cage</td>
<td>18 in. Oc 4</td>
<td>18</td>
</tr>
<tr>
<td>SSMBLGI5-HighBars</td>
<td>62.75</td>
<td>34</td>
<td>#4 18 in. OC 4 1/2 in.</td>
<td>Cage #3</td>
<td>18 in. Oc 4</td>
<td>9</td>
</tr>
<tr>
<td>SSMBLGI6-Deep</td>
<td>67.25</td>
<td>34</td>
<td>#4 18 in. OC 4 1/2 in.</td>
<td>Cage #3</td>
<td>18 in. Oc 4</td>
<td>9</td>
</tr>
<tr>
<td>SSMBLGI5-No.6Bar</td>
<td>62.75</td>
<td>34</td>
<td>#4 18 in. OC 4 1/2 in.</td>
<td>Cage #3</td>
<td>18 in. Oc 4</td>
<td>9</td>
</tr>
<tr>
<td>SSMBLGI5-Fisch</td>
<td>54</td>
<td>34</td>
<td>#4 18 in. OC 4 1/2 in.</td>
<td>Cage #3</td>
<td>4.5 in. Oc 4.5</td>
<td>4.5</td>
</tr>
<tr>
<td>SSMBLGI7-Control2</td>
<td>62.75</td>
<td>34</td>
<td>#4 18 in. OC 4 1/2 in.</td>
<td>Cage #3</td>
<td>18 in. Oc 4</td>
<td>9</td>
</tr>
</tbody>
</table>

*The depth of the transverse reinforcement was taken from the bottom of the precast section to the center of the reinforcement.

*The maximum spacing was the maximum nominal distance between reinforcement traversing the longitudinal joint, regardless of type (i.e., transverse hooked bars or cage).

*The reinforcement ratio shown is that corresponding to crack control, see above and Section 5.1.

The NCHRP report\(^4\) recommends that Equation 2 be used to calculate the area of transverse load distribution reinforcing. This equation combines the percentages of longitudinal mild and prestress flexural reinforcement that need to be provided in the transverse direction based on AASHTO LRFD\(^20\) Equations 5.14.4.1-1 and 5.14.4.1-2, respectively. Because the AASHTO equations were developed for cast-in-place slab systems in which the transverse reinforcement is located immediately above the longitudinal reinforcement, an adjustment is made to account for the fact that the depths to the center of gravities for the longitudinal prestress and transverse load distribution reinforcement could be significantly different\(^4\). This adjustment is made by multiplying the second part of Equation 1 with \(\alpha\), which is defined below.

\[ A_{tld} = k_{mild}A_{l-mild} + \alpha k_{ps}A_{l-ps} \]  \(\text{(2)}\)

\[ k_{mild} = \frac{100}{\sqrt{L}} \leq 50\% \]  \(\text{(3)}\) \(- (LRFD 5.14.4.1 - 1)\)

\[ k_{ps} = \frac{100}{\sqrt{L}} \frac{f_{pe}}{60} \leq 50\% \]  \(\text{(4)}\) \(- (LRFD 5.14.4.1 - 2)\)
where:

\begin{align*}
A_{\text{thd}} &= \text{area required for transverse load distribution reinforcement (in}^2) \\
A_{l,\text{mild}} &= \text{area of longitudinal mild flexural reinforcement (in}^2) \\
A_{l,\text{ps}} &= \text{area of longitudinal prestressed flexural reinforcement (in}^2) \\
k_{\text{mild}} &= \text{percentage of longitudinal mild flexural reinforcement} \\
k_{\text{ps}} &= \text{percentage of longitudinal prestressed flexural reinforcement} \\
\alpha &= \frac{d_{\text{cgs}}}{d_{\text{trans}}} \geq 1.0 \\
d_{\text{cgs}} &= \text{depth to center of gravity of prestressed reinforcement (in.)} \\
d_{\text{trans}} &= \text{depth to center of gravity of transverse reinforcement (Figure 2.34) (in.)} \\
L &= \text{span length (ft)} \\
f_{\text{pe}} &= \text{effective stress in prestressing strand (ksi)}
\end{align*}

**Figure 2.34** Reinforcement and depth of concrete considered in the calculation of the reinforcement ratio for transverse load transfer (highlighted in yellow)\(^4\)

On the other hand, reflective crack control reinforcing is considered to include both lapped transverse hooked bars plus the lower leg of the cage hoops. It is noted that the drop-in-cage is only effective in controlling cracks that develop in the vicinity over the longitudinal joint and that cracks that might form at the interface between the cast-in-place topping and precast web would only be restrained by the reinforcement provided for transverse load distribution\(^4\).

The recommended spacing for crack control reinforcing included an evaluation of existing provisions in AASHTO LRFD Bridge Design Specifications\(^{20}\), ACI 318-08\(^{24}\) and recommendation by Frosch et al.\(^{25}\) related to crack control in bridge decks. These recommendations are summarized in
Table 2.2. Based on the test results of large scale laboratory bridges and sub-assemblage specimens French et al.\textsuperscript{4} concluded that the amount of reflective crack control reinforcing should be based on the equation recommended by Frosch et al.\textsuperscript{25} and that maximum spacing could be based on AASHTO’s\textsuperscript{20} equation bounded to a maximum spacing of 12 in.

### Table 2.2 \textsuperscript{1} Spacing and reinforcement ratio limits for flexural and crack control reinforcement\textsuperscript{4}

<table>
<thead>
<tr>
<th>Reinforcement Type</th>
<th>Reinforcement Design Limits</th>
<th>Source</th>
<th>Article in Spec.</th>
</tr>
</thead>
<tbody>
<tr>
<td>Crack control and shrinkage and temperature</td>
<td>[ s &lt; \frac{700\gamma_e}{\beta_s \cdot f_{ss}} - 2 \cdot d_c \quad \text{where} \quad \beta_s = 1 + \frac{d_c}{0.7 \cdot (h - d_t)} ]</td>
<td>AASHTO (2010)</td>
<td>5.7.3.4</td>
</tr>
<tr>
<td></td>
<td>[ s \leq \min (1.5h, 18 \text{ in.}) ]</td>
<td></td>
<td>5.10.3.2</td>
</tr>
<tr>
<td>Shrinkage and temperature</td>
<td>[ A_{\text{sAASHTO}} \geq \frac{1.30 \cdot bh}{2(b + h)f_y}; \quad 0.11 \leq A_{\text{sAASHTO}} \leq 0.60 ]</td>
<td></td>
<td>5.10.8</td>
</tr>
<tr>
<td>Crack control and shrinkage and temperature</td>
<td>[ s &lt; 15 \cdot \frac{40000}{f_s} - 2.5 \cdot c_c ]</td>
<td>ACI 318-08</td>
<td>10.6.4</td>
</tr>
<tr>
<td>Shrinkage and temperature</td>
<td>[ A_{\text{sACI}} = 0.0018bh \quad \text{(when using Gr. 60 bars)} ]</td>
<td></td>
<td>7.12.2.1</td>
</tr>
<tr>
<td></td>
<td>[ s \leq \min (5h, 18 \text{ in.}) ]</td>
<td></td>
<td>7.12.2.2</td>
</tr>
<tr>
<td>Crack control</td>
<td>[ s &lt; 9 \cdot \left(2.5 - \frac{c_c}{2}\right) \leq 9\text{in} ]</td>
<td>Frosch et al. (2006)</td>
<td></td>
</tr>
<tr>
<td></td>
<td>[ \rho_{\text{Frosch}} = \frac{r}{f_y} ]</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

\textsuperscript{1}Variables in Table 1.1.1 are defined as follows:
\[ \gamma_e = \text{1.0 for Class 1 (No corrosion concerns); 0.75 for Class 2 (Corrosion concerns)} \]
\[ f_{ss} = \text{Stress in reinforcement at service [ksi]} \]
\[ d_c = \text{Depth of concrete cover measured from tension fiber of concrete to center of reinforcement defined by AASHTO (2010) Section 5.7.3.4 [in.]} \]
\[ c_c = \text{Depth of concrete cover measured from tension fiber of concrete to face of reinforcement defined by ACI 318 \text{ } 08 \text{ Section 10.6.4 [in.]} \]
\[ h = \text{Total section depth [in.]} \]
\[ A_{\text{sAASHTO}} = \text{Area of reinforcement in each direction and each face [in.}^2/{\text{ft.}} \]
\[ A_{\text{sACI}} = \text{Area of shrinkage and temperature reinforcement [in.}^2 \]
\[ b = \text{Least width of component section [in.]} \]
\[ f_s = \text{Stress in reinforcement at service [psi]} \]
\[ f_c = \text{28 - day concrete compressive strength [psi]} \]
\[ f_y = \text{Reinforcement yield strength [psi]} \]
\[ \rho_{\text{Frosch}} = \text{Reinforcement ratio defined by Frosch et al. (2006). Equivalent to: area of reinforcement / gross area of section [dim]} \]

According to the design guide in the NCHRP report\textsuperscript{4}, it is recommended that when using the equation recommended by Frosch et al.\textsuperscript{25}, the gross area of section be taken as the distance between the top of the precast flange and the top of the precast web times the width under
consideration (Figure 2.35). This approach assumes that the entire cast-in-place topping over the joint, from the top of the precast flange to the top of the precast web, will be cracked under a tensile stress of \(6\sqrt{f'c}\) (where \(f'c\) is in psi) and that enough steel should be provided to prevent yielding. In addition, \(d_c\) in AASHTO’s\(^{20}\) equation, should be taken equal to the distance between the top of the joint to the center of transverse steel.

\[ \text{Figure 2.35 Reinforcement and depth of concrete considered in the calculation of the reinforcement ratio for reflective crack control (highlighted in yellow)}^4 \]

Halverson et al.\(^{26}\) evaluated the performance of existing bridges built with precast inverted T-beams and reviewed a number of design details. Field inspections were conducted for five existing inverted T-beam bridges, consisting of crack mapping surveys and core sample examinations to evaluate design changes related to reflective crack control\(^{26}\). Reflective cracking was observed on both Bridge No. 13004 near Center City, MN and Bridge No.33008 near Mora, MN. The latter exhibited long surface cracks over a number of longitudinal joints and two reflective cracks determined from the observation of core specimens. Chamfering the corners of the precast web and flange was considered favorable because it reduces the severity of the discontinuity by increasing the surface area over which stresses in the cast-in-place topping develop at these points. Halverson et al.\(^{26}\) made the following recommendations within the framework of transverse bending and reflective cracking:

- Transverse deck reinforcement should be detailed to provide a \(\rho_g\) of at least 0.0063 with spacing no greater than 9 in. to control shrinkage cracking at the level of the deck reinforcement. These recommendations are based on those of Frosch\(^{25}\). For PCSSS design, \(\rho_g\) was calculated using the 6 in. of CIP concrete above the precast webs. One way to implement these recommendations would be to increase the current detailing requirements of No. 4 bars at 6 in. spacing to No. 5 bars at 6 in. spacing \((\rho_g = 0.0086)\). Alternatively, No. 5 bars at 8 in. spacing could be used \((\rho_g = 0.0065)\).
• Transverse reinforcement in the trough region consisting of the combination of trough hooks and the stirrups of the “drop-in” cage should also be detailed to satisfy Frosch\textsuperscript{25} recommendations. The trough hooks from adjacent precast inverted tees should be located adjacent to each other to facilitate transverse load transfer in the bridge. The drop-in cage can be staggered relative to the trough hooks to meet the Frosch\textsuperscript{25} 9 in. maximum spacing requirement.

• Smooth flange surfaces should continue to be used. The shallow voids resulting from fabrication need not be patched. Subsequent to this study, MnDOT has employed the use of a bond breaker such as mastic over the longitudinal joint between adjacent inverted tees. The purpose of the mastic is to soften a potential crack initiation site. Evaluation of the effectiveness of the bond breaker was not within the scope of this report. Pre-wetting of the flange precast surface prior to casting the CIP should help to reduce the shrinkage of the concrete at the CIP-precast interface and has an added benefit of removing laitance from the smooth flange surface, reducing the likelihood of delamination. Water should not be allowed to pool in the voids, and if mastic is used in construction, measures should be taken to enable the water to drain prior to casting the CIP.

2.5.3 Time Dependent Effects

Most bridges composed of simple span precast girders are made continuous by means of a cast-in-place concrete diaphragm. Various methods of creating continuity over the piers were discussed in section 2.4. In such cases, the designer has the option of taking advantage of the created continuity and design the bridge as continuous for loads applied after the continuity is established or design it as a series of simple spans. Even when the latter approach is taken the bridge in many cases is still detailed as continuous to eliminate expansion joints in the deck slab. Deformations that occur after continuity is established from time dependent effects such as creep, shrinkage and temperature variation cause restraint moments\textsuperscript{20}. The cast-in-place topping in such composite bridges is cast at a later date than the concrete for the precast girder. As a result the deck will tend to shrink more than the girder. This gives rise to differential shrinkage which leads to the development of negative restraint moments at the interior supports (Figure 2.36 (a)). Data from various projects does not show the effects of differential shrinkage, therefore, it is questionable whether negative moments due to differential shrinkage form to the extent predicted by analysis\textsuperscript{20}. Because field observations of significant moment distress have not been reported, negative moments caused by differential shrinkage are often ignored in design.

Similarly, the prestressing force in the precast girder will cause the girder to creep and give rise to the formation of positive restraint moments (Figure 2.36 (b)). Even though restraints moments are computed at interior supports of continuous bridges, they affect the design moments at all locations on the bridge\textsuperscript{20}. The magnitude and direction of these restraint moments depend on girder age at the time continuity is established, properties of girder and slab concrete, and bridge and girder geometry. The data show that the later continuity is formed, the lower the
values of positive restraint moments which will form. Because positive restraint moments are not desirable, waiting as long as possible after the girders are cast to establish continuity and cast the deck appears to be beneficial.

AASHTO requires bridges to be designed for restraint moments that may develop because of time-dependent or other deformations, unless contract documents require a minimum girder age of at least 90 days when continuity is established. If the 90 day provision is satisfied then the following simplification may be applied:

- Positive restraint moments caused by girder creep and shrinkage and deck slab shrinkage may be taken to be zero
- Computation of restraint moments shall not be required
- A positive moment connection shall be provided with a factored resistance, $\phi M_n$, not less than 1.2$M_{ct}$

Smith et al. investigated the development of restraint moments in Concept 1 laboratory bridge described above prior to load testing it. The precast inverted T-beams were 7 days old when continuity was established, which was considered to represent the earliest feasible age for the precast girders when the cast-in-place deck is placed. The development of restraint moments was monitored for a period of 250 days using loads cells at each simply-supported end of the specimen. Experimental results on restraint moments were compared with two prediction methods: the PCA method, developed by Freyermuth and the P-method proposed by Peterman and Ramirez. To evaluate the accuracy of each prediction method, coefficients were fitted to the AASHTO and PCA creep and shrinkage models using strains from concrete shrinkage and creep specimens. Based on this investigation it was recommended that the P-method be used in future designs, or that the PCA method be modified such that CIP shrinkage restraint is considered and another method for creep and shrinkage prediction, such as AASHTO replaces the charts in the PCA method paper.

Halverson et al. report that when originally conceived, MnDOT designed the inverted T-beam system as a continuous system for live loads, assuming that it would be the most economical method. Provided that accounting for restraint moments is a design-intensive task and is highly variable depending on the method of analysis and assumed material properties, Halverson et al. performed a parametric study to determine how much benefit a continuous system design could provide versus simple-span design. This parametric study included bridges with two and three span configurations with spans that ranging from 20 ft to 60 ft. The results of the parametric study were used to recommend one of several design options: designing as a continuous system, as a simply-supported system, or as a simply supported system with an added continuity connection. These recommendations are summarized below:

- PCSSS beams should be designed as simply-supported for live load and dead loads. Providing a positive moment connection for live load continuity produces significant time-dependent and thermal restraint moments. These restraint moments effectively
negate the reduction in live load, making continuous system design equally or less economical than simply-supported design. The CIP deck should be designed as continuous for negative live load.

- Designing the PCSSS beams as simply-supported for live load while also providing a positive moment connection should not be done without accounting for the induced restraint moments. A simple method to account for restraint moments was developed, but it required the positive moment connection to yield in order to control the maximum restraint moment that could develop. For typical PCSSS configurations, four No. 5 bars is reasonable for reinforcement in positive moment connections. However, the restraint moment developed by these bars needs to be determined so the precast beams can be properly designed. The positive moment connection reinforcement should extend a development length on either side of the pier to develop the full yield restraint moment. Fatigue issues of the positive moment connection may need to be taken into consideration, as this reinforcement could undergo significant tensile stress ranges over its life due to thermal gradient effects.

- Deck design should be bounded by both simple span assumptions and a pseudo-continuous assumption. This bounding approach recognizes the time-dependent nature of creep and shrinkage locked into the beam and deck system. In the analysis of negative 52 moments for longitudinal deck reinforcement design, it was found that the controlling Service I load combination could be approximated by the following equation, where $DL_2$ represents the post-continuity dead load, $LL$ represents live load, and $\gamma LL_{RM}$ represents a conservative approximation of the time-dependent and thermal gradient restraint moment equal to a value of 1.2 times the $LL$ moment at the bridge pier.

$$DL_2 + LL + \gamma LL_{RM}$$

The approximate restraint moment envelope is tapered from a $\gamma$ value of 0 at the abutment to a $\gamma$ value of 1.2 times the $LL$ moment value at the pier and constant $\gamma$ value of 1.2 over the center span. This approximate analysis of negative moments was calibrated for configurations with spans around 20 ft long and becomes increasingly conservative for spans up to 60 ft. For longer span lengths, even a $\gamma$ value of 1.0 may yield unnecessarily large negative moments at the piers. In this case, a more refined restraint moment analysis may be called for to limit the amount of negative moment steel needed in the deck.
End regions of prestressed members are subject to high concentrated loads during the transfer of the prestressing force. As a result, the state of stress in these regions is complicated and cannot be predicted by the Euler-Bernoulli beam theory in which plane sections are assumed to remain plane. According to Saint Venant’s principle, the disturbance caused by the concentrated forces at the ends of the member diminishes after a distance $h$ from the end of the member, where $h$ is the overall depth of the member. The tensile stresses created within the distance $h$ from the end of the member are identified as either spalling or bursting stresses depending on their location.

Gergerly et al.\textsuperscript{30} state that the horizontal cracks that frequently forms in the end region of prestressed concrete members when the prestressing strand is released and the prestressing force is transferred to the concrete section are defined as “spalling” cracks, though often incorrectly labeled as “bursting” or “splitting” cracks. If unrestrained, these cracks can extend into the precast member and negatively impact the flexural and shear strength of the member, as well as its durability. Studies performed by Fountain\textsuperscript{31} suggest that these cracks cannot be eliminated, however vertically oriented reinforcing steel can limit crack width and propagation. Unlike spalling stresses, which develop at the end face of the member, bursting stresses occur along the line of the prestressing force beginning a few inches into the beam and extend through the
transfer length. The terminology for the stresses developed in the end zones of pre-tensioned members used in the study performed by French et al.\textsuperscript{4} is illustrated in Figure 2.37.

**Figure 2.37** Spalling and bursting stresses near the end zone of prestressed members\textsuperscript{4}

Studies by Gergerly\textsuperscript{30} have shown that the distribution of the tensile stresses in the end region depend on the eccentricity of the prestressing force in the member. For example, in a concentrically loaded member forces distribute symmetrically through the vertical member height until a uniform stress distribution is established at a distance $h$ from the end of the member (Saint Venant’s principle). In such a member, the spalling forces developed at the end face are smaller than the bursting forces that develop at a distance $h/2$ from the end of the member (Figure 2.38 (a)). Conversely, in an eccentrically loaded member the spalling force $s$ developed near the end face are higher than the bursting forces developed a certain distance away from the end of the member (Figure 2.38 (b)). Hawkins\textsuperscript{32} corroborated Gergerly’s\textsuperscript{30} findings and also found that as eccentricity increased so did the magnitude of maximum tensile stress in the spalling zone.
Eriksson\textsuperscript{33} performed an evaluation of the stresses in the end zones of precast inverted T-beams to determine the applicability of the AASHTO\textsuperscript{20} provisions on pre-tensioned anchorage zones. AASHTO\textsuperscript{20} pre-tensioned anchorage zones provisions require the placement of vertical reinforcement to resist splitting forces. This vertical reinforcing is required to be placed within a distance equal to $h/4$ from the end of the member, where $h$ is the overall depth of the member. French et al.\textsuperscript{4}, state that the original specifications, were developed to control “spalling” stresses in I-girders, and were mislabeled in the code as “bursting” stresses. In the current AASHTO\textsuperscript{20} provisions, the term “bursting” has been replaced with “splitting” although the correct terminology that should be used is “spalling”. Although not specifically addressed in AASHTO\textsuperscript{20}, the confinement requirements of AASHTO\textsuperscript{20} 5.10.10.2 should help control the bursting and splitting stresses that develop in the transfer length region\textsuperscript{4}.

The vertical reinforcing is required to resist a vertical force equal to 4\% of the total prestressing force. Because the overall depth of precast inverted T-beams is relatively shallow compared to I-girders, the requirement to place the vertical steel in the end zone within a distance equal to $h/4$ from the end of the member results in congestion problems. On the other hand, the placement of vertical steel in the end zones of wide and shallow members (solid or voided slabs) is relaxed by allowing the designer to spread this steel within a distance $h/4$ where $h$ is the width of the member rather than its depth. According to French et al.\textsuperscript{4} such a relaxation may not be appropriate when trying to control spalling stresses.

The evaluation that Eriksson\textsuperscript{33} and French et al.\textsuperscript{4} performed, included experimental and numerical studies. The experimental study was performed on the Concept 1 and Concept 2 Laboratory Bridge Specimens, which featured various configurations of end zone reinforcing (Table 2.3). The experimental results indicated that the 12 in. deep precast sections had sufficient strength to resist the tensile stresses created in the end zone even in cases where no vertical steel was present. These findings were corroborated with the results of numerical studies that showed certain inverted-T members did not require spalling reinforcement, specifically those members

![Diagram of stresses in the end zone](image-url)
with depths less than 22 in. for which the expected concrete strength was higher than the expected tensile stresses due to the development of prestress\(^4\).

In contrast, for deep inverted T-beams, it was found that larger amounts of spalling reinforcement than specified by AASHTO\(^20\) were found to be required through the numerical study. It was also concluded that the reinforcement should be placed as close to the end of the beam as possible (i.e., within h/4 of the end of the member, where h represents the depth of the member).

**Table 2.3 Vertical reinforcement in configurations 1-4 of the precast members utilized in experimental study**

<table>
<thead>
<tr>
<th>Configuration</th>
<th>Description of Vertical End Zone Reinforcement</th>
<th>Cross Section View of Stirrup</th>
<th>Elevation View of Reinforcement Spacing</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>#3 stirrup at 2 and 4 in. total area = 0.44 in(^2)</td>
<td><img src="image1.png" alt="Image" /></td>
<td>2 spaces @ 2 in.</td>
</tr>
<tr>
<td>2</td>
<td>#4 stirrup at 2 in. total area = 0.40 in(^2)</td>
<td><img src="image2.png" alt="Image" /></td>
<td>1 space @ 2 in.</td>
</tr>
<tr>
<td>3</td>
<td>#5 four legged stirrup at 2 and 4 in. total area = 2.5 in(^2)</td>
<td><img src="image3.png" alt="Image" /></td>
<td>2 spaces @ 2 in.</td>
</tr>
<tr>
<td>4</td>
<td>#5 stirrup at 2 and 4 in. total area = 1.2 in(^2)</td>
<td><img src="image4.png" alt="Image" /></td>
<td>2 spaces @ 2 in.</td>
</tr>
</tbody>
</table>

Some of the suggested modifications to AASHTO\(^20\) Article 5.10.10.1 that resulted from this study are presented below:

- For all sections other than rectangular slabs and shallow inverted-T sections with heights less than 22 in, the splitting spalling resistance of pretensioned anchorage zones provided by reinforcement in the ends of pretensioned beams shall be taken as:

\[
Pr = f_s A_s
\]

*where:*

- \(f_s\) = stress in steel not to exceed 20 ksi
- \(A_s\) = total area of reinforcement located within the distance h/4 from the end of the beam (in.\(^2\))
\( h \) = overall dimension of precast member in the direction in which splitting spalling resistance is being evaluated (in.)

The resistance shall not be less than four percent of the total prestressing force at transfer.

- In pretensioned anchorage zones of rectangular slabs and shallow inverted-T sections with heights less than 22 in., vertical reinforcement in the end zones is not required if:

\[
\sigma_s < f_r
\]

where:
\[
\sigma_s = \frac{P}{A} \left( 0.1206 \frac{e^2}{h d_b} - 0.0256 \right) \geq 0
\]

\[
f_r = 0.23 \sqrt{f'_{ci}}
\]

\( \sigma_s \) = maximum spalling stress on the end face (ksi)
\( f_r \) = direct tensile strength as defined by Article C5.4.2.7 (ksi)
\( P \) = prestressing force at transfer (kips)
\( A \) = gross cross-sectional area of concrete (in\(^2\))
\( e \) = strand eccentricity (in.)
\( h \) = overall depth of precast member (in.)
\( d_b \) = prestressing strand diameter (in.)
\( f'_{ci} \) = concrete compressive strength at transfer (ksi)

Where end zone vertical reinforcement is required, it shall be located within the horizontal distance \( h/4 \) from the end of the beam, and shall be determined as:

\[
A_s = \frac{P \left( 0.02 \frac{e^2}{h d_b} - 0.01 \right)}{\bar{f_s}}
\]

where:
\( A_s \) = total area of reinforcement located within the distance \( h/4 \) from the end of the beam (in.\(^2\))

The resistance shall not be less than four percent of the total prestressing force at transfer. In all cases, the reinforcement shall be as close to the end of the beam as practicable. Reinforcement used to satisfy this requirement can also be used to satisfy other design requirements.
2.5.5 Composite Action

French et al.\textsuperscript{4} investigated the effects of variations in horizontal shear reinforcement on composite action by testing Concept 1 and Concept 2 laboratory bridge specimens to near ultimate levels of load. The presence of horizontal shear reinforcing created difficulties for the fabricator when finishing the top of the precast web and during the placement of deck steel. To facilitate the placement of deck steel on the site, in initial field applications the clearance between the bottom of the hook and the top of the precast web was kept as small as possible. This condition was illustrated in Figure 2.27. However, such a small clearance may have limited the effectiveness of extended shear reinforcing, because aggregate could not flow below the returned stirrups. In addition, past research by Kovach et al.\textsuperscript{34} suggested that concrete girders loaded to induce positive moments in section without horizontal shear ties were observed to achieve sufficiently large levels of horizontal shear stress and remain composite (French et al. 2011). To increase the contribution of cohesion at the interface between the precast beam and cast-in-place topping the top of the precast web was roughened to an amplitude of approximately ¼ in by means of raking in the transverse direction (Figure 2.39).

Figure 2.39 Intentionally roughened surface, by means of raking, of the top of the precast web used for the construction of Concept 2 laboratory bridge specimen\textsuperscript{4}

Figure 2.40 shows the tri-actuator test setup used in testing the laboratory specimens for composite action up to the largest load levels available in the laboratory. Load was applied to each specimen using a displacement-controlled program, primarily to prevent the rapid collapse of the specimens, if applicable\textsuperscript{4}. For Concept 1 laboratory specimen, the location of the loading frame was moved 20 in, away from mid-span towards the center pier. This was done to avoid a situation in which the applied load was close to the longitudinally oriented strain gages used in the fatigue and environmental studies described earlier, which were located around the mid-span area. The presence of a normal force was believed to increase the horizontal shear resistance at the location of the applied load. The longitudinally oriented instrumentation for Concept 2 laboratory specimen was designed to allow the load frame to remain in the same position for the ultimate loading as was utilized for the fatigue and environmental studies, with a center to center distance of 21 in. between the instrumentation and the load beam.
Longitudinally oriented strain gages were installed in the precast and cast-in-place components to obtain a strain distribution through the depth of the composite section. A composite section should experience linearly varying strains through the full depth of the section, while loss of composite section would be evinced if distinct curvatures were observed in each component (i.e. precast and CIP components). Longitudinal strain measurements were used to obtain longitudinal curvatures. Figure 2.41 shows how the linear trend line appears to provide a good fit to the strains measured in both the precast and cast-in-place components. The measured curvature was based on the linear trend line, where the curvature was the reciprocal of the slope of the line.

**Figure 2.41** Longitudinal strains measured through the section depth at mid-span of one of the panels.

Horizontal shear reinforcing in span 2 of Concept 1 laboratory bridge specimens was selected to satisfy AASHTO requirements and matched the one used in the Center City Bridge,
The horizontal shear reinforcing used in span 1 of Concept 1 laboratory bridge specimen consisted of less steel than the code minimum\(^{35}\), whereas Concept 2 laboratory bridge specimen contained no horizontal shear reinforcing.

In all tests the sections remained composite well past service load levels. The specimens were loaded until the capacity of the loading system was met, which was in excess of the predicted nominal capacity of both laboratory bridge specimens. The horizontal shear stress was calculated by dividing the total compression force in the composite section corresponding to the ultimate load achieved during testing by half the span times the total width of the bridge. This stress was determined to be 135 psi. It was concluded that AASHTO LRFD Bridge Design Specifications\(^{35}\) should allow for the design of precast slab span structures without horizontal shear ties, and allow for the development of a maximum factored shear stress of 135 psi in sections with intentionally roughened surfaces (i.e., \(\frac{1}{4}\) in. rake) unreinforced for horizontal shear.

### 2.5.6 Skew Effects

French et al.\(^{4}\) performed a parametric study to investigate skew effects in bridges built with precast inverted T-beams. According to the PCI Bridge Design Manual\(^{10}\), in solid slab-span bridge systems with skew supports, the load tends to take a “short-cut” between the obtuse corners of the span, while the load in skewed bridges supported by longitudinal I-girders tends to flow along the length of the supporting members\(^{10}\). The behavior of the inverted T-beam system was expected to be bounded by these behaviors, and would subsequently tend to exhibit the characteristics of longitudinal stringers bridges as the precast joint is degraded due to reflective cracking\(^{4}\). Figure 2.42 illustrates the placement of precast slab span panels at a skewed support.

Another motivation behind the parametric study of skew effects in inverted T-beam bridges was the quantification of the horizontal shear stress above the precast joint. Several Finite Element Models (FEM) were developed to investigate the relationship between the skew angle and the resulting magnitude of this stress\(^{4}\). A total of eight FEM were developed, with skew angles ranging from 0\(^{\circ}\) to 45\(^{\circ}\). Each model represented a 30 ft simple-span bridge structure. For each of the skew angles selected, two corresponding finite element models were created, one with the presence of a 3 in. deep precast joint and an un-bonded surface between the top of the precast flanges and the CIP concrete and a second with a monolithic thickness and the absence of the precast joint\(^{4}\).

Figure 2.43 illustrates the load cases considered in this parametric study. Each load case consisted of a 35 kip concentrated load applied at quarter points and at mid-span of the exterior panel over a 12 in. by 12 in. patch. For each load case the largest horizontal shear stress above the precast joint was recorded and these values were used to obtain a horizontal shear stress envelope for the jointed and monolithic models. It was determined that the magnitude of the horizontal shear stress envelope above the precast joint remained relatively constant through the range of skew angles considered for both the jointed and monolithic models. Figure 2.44 shows the variation of the horizontal shear stress above the joint for both the monolithic and jointed models. With increasing skew angle the horizontal shear stress increased by 15 % for the
monolithic models and by less than 10% for the jointed models. Based on this observation it was concluded that the presence of a joint between the precast panels was not expected to significantly affect the performance of the inverted T-beam system in skewed applications, and that the design of skewed bridges with precast inverted T-beams could be completed assuming a monolithic slab span system.

Figure 2.42 Placement of precast slab span panels at a skewed support
2.5.7 Live Load Distribution Factors

According to AASHTO LRFD Bridge Design Specifications\textsuperscript{20}, the determination of live load distribution factors for slab-span type bridges is related to the concept of determining an effective lane width. The same approach can be applied to composite bridges built with precast...
inverted T-beams. The live load effect expected to be resisted by each panel can be determined by multiplying the live load effects (moments and shears) for each lane by the live load distribution factors. The live load distribution factors can be calculated by dividing the width of the precast panel by the effective lane width. AASHTO LRFD Bridge Design Specifications provide equations for determining this effective lane width for slab-span bridges. The specification provides further guidance in the determinations of transverse reinforcement for slab span bridges as a simple proportion of the total longitudinal tension reinforcement based on the span of the structure, however the validity of this relationship when applied to composite bridges with precast inverted T-beams was unknown.

To determine whether the effective lane width approach for slab span bridges can be applied to the inverted T-beam system for the purpose of calculating live load distribution factors French et al. performed a parametric study to investigate the effects of longitudinal discontinuity between precast members on the longitudinal and transverse load distribution.

A total of nine finite element models were created which featured bridges built with monolithic slab-spans and those built with precast inverted T-beams with longitudinal joints. Most of the bridge models featuring precast inverted T-beams incorporated the presence of a 3 in. longitudinal joint with the exception of one model which featured a 15 in. longitudinal crack.

Modeled bridges included simple span as well as three-span continuous bridges. Figure 2.45 illustrates panel and joint numbering used in the placement of tandem loading for the center span of the continuous models and the simple-span models. A tandem load was utilized for all models, with a total load of 12.5 kips distributed over a 10 in. by 20 in. patch. Five variations in the applied tandem loading were considered for each run, as described below:

- Tandem 1 - tandem loading centered over webs of panels 5 and 6
- Tandem 2 – tandem loading centered over precast joints 4 and 5
- Tandem 3 – double tandem loading centered over webs of panels 4, 5, 6, 7
- Tandem 4 – double tandem loading centered over precast joints 3, 4, 5, 6
- Tandem 5 – double tandem loading with 12.5 kip patch loads over joints 4 and 6 and a double patch load (i.e., 25 kips) over joint 5.

The tandem 5 loading was considered to be a worst case loading scenario with respect to expected lane loading because the tandems were spaced much closer than would by physically possible. Table 2.4 shows the longitudinal curvatures calculated from the finite element models (FEM) and those calculated based on the effective lane width approach by using the design equations in AASHTO (denoted as “Design”).

The ratios between the curvatures obtained from the FEM and those calculated using AASHTO’s method indicated that live load distribution factors for the precast inverted T-beam system can be conservatively calculated using the equations in AASHTO. The ratios for the precast jointed models are slightly higher than those for the monolithic models, however they are always less than one. Even for the model that simulated a 15 in. crack (Run 9), the ratio was
0.84. This suggests that even when reflective cracking was assumed to have progressed vertically within 3 in. of the extreme compression fiber, the design effective lane widths provided by AASHTO\textsuperscript{20} Article 4.6.2.3 prove to be conservative, and should therefore be utilized for the design of composite bridges with precast inverted T-beams\textsuperscript{4}.

**Figure 2.45** Panel and joint numbering used in the placement of tandem loading for the center span of the continuous models and simple-span models\textsuperscript{4}.
Table 2.4 FEM and design longitudinal curvatures under Tandem 2 and Tandem 5 load cases

<table>
<thead>
<tr>
<th>Run</th>
<th>Max FEM</th>
<th>Design</th>
<th>FEM/Design</th>
<th>Max FEM</th>
<th>Design</th>
<th>FEM/DESIGN</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>3.72</td>
<td>7.37</td>
<td>0.50</td>
<td>6.76</td>
<td>8.13</td>
<td>0.83</td>
</tr>
<tr>
<td>2</td>
<td>4.76</td>
<td>11.17</td>
<td>0.43</td>
<td>8.81</td>
<td>12.32</td>
<td>0.72</td>
</tr>
<tr>
<td>3</td>
<td>4.41</td>
<td>11.17</td>
<td>0.39</td>
<td>8.25</td>
<td>12.32</td>
<td>0.67</td>
</tr>
<tr>
<td>4</td>
<td>4.35</td>
<td>11.17</td>
<td>0.39</td>
<td>8.14</td>
<td>12.32</td>
<td>0.66</td>
</tr>
<tr>
<td>5</td>
<td>2.68</td>
<td>5.64</td>
<td>0.48</td>
<td>NM(^1)</td>
<td>7.05</td>
<td>NM</td>
</tr>
<tr>
<td>6</td>
<td>3.4</td>
<td>8.34</td>
<td>0.41</td>
<td>NM</td>
<td>10.43</td>
<td>NM</td>
</tr>
<tr>
<td>7</td>
<td>3.26</td>
<td>8.34</td>
<td>0.39</td>
<td>6.03</td>
<td>10.43</td>
<td>0.58</td>
</tr>
<tr>
<td>8</td>
<td>3.23</td>
<td>8.34</td>
<td>0.39</td>
<td>5.97</td>
<td>10.43</td>
<td>0.57</td>
</tr>
<tr>
<td>9</td>
<td>5.18</td>
<td>11.17</td>
<td>0.46</td>
<td>10.38</td>
<td>12.32</td>
<td>0.84</td>
</tr>
</tbody>
</table>

\(^1\)Not measured
Shaded rows indicate monolithic models

2.5.8 Connection of the superstructure to the substructure

The connection detail between the superstructure and the substructure determines the level of restraint between the two components. Typically, bearing pads under precast components tend to reduce the level of restraint to the superstructure in both the longitudinal and transverse directions. The pad flexibility leads to a less restrained support, which in turn reduces the likelihood of developing tensile stresses as a result of time dependent and temperature effects. Conversely, if the superstructure is in contact with the pier cap, the increased frictional restraint between the two concrete surfaces could cause excessive tensile stresses as a result of time dependent and temperature effects\(^26\).

Halverson et al.\(^26\) report that all inverted T-beam bridges in Minnesota from Generation 1 through Generation 3 had an ½ in. elastomeric bearing pad placed underneath the precast beam ends along the centerline of bearing. In addition, polystyrene was placed in the regions directly under the precast beams, excluding the area of the bearing pad (Figure 2.46). Because there was polystyrene placed between the block-out regions or the ends of the beams the cast-in-place topping came in direct contact with the pier cap and in conjunction with the embedded dowels increased the level of restraint between the superstructure and substructure\(^26\).

For Generation 4 bridges scheduled to be built in 2011 MnDOT proposed a new bearing detail, in which polystyrene covered the entire surface of the abutments and the pier caps, excluding the area of the bearing pad. The primary function of the extended polystyrene was to
prevent contact between the cast-in-place topping and the abutment/piercaps with the purpose of reduce the level of restraint\textsuperscript{26}.

\begin{figure}[h]
\centering
\includegraphics[width=\textwidth]{image.png}
\caption{Bearing detail at the pier caps\textsuperscript{27}}
\end{figure}

A summary of the recommendations by Halverson et al.\textsuperscript{26} regarding the connection between the superstructure and the substructure is provided below:

- Anchorage dowels at the piers and abutments should continue to be placed in the block-out regions to aide in construction. Typically 1 in. diameter dowels have been used, with the latest generation of beams using stainless dowels at all locations. Dowels in block-outs away from the longitudinal bridge center should be minimized as these can add restraint to shrinkage which may induce cracking in the deck. Dowels at piers should be wrapped with polystyrene sleeves to reduce restraint-induced cracking. Dowels at abutments are recommended to not be wrapped in order to reduce the likelihood of promoting separation and water intrusion between the bridge and the approach panel. No more than four dowels should be placed in each block-out. If possible, dowels should be placed in one row per support rather than in two rows spaced longitudinally, in part to reduce moments developed in the dowel bar couple.
- MnDOT should evaluate the performance of the bridges built for Generation 4 in 2011, regarding the use of the new bearing detail aimed at reducing lateral restraint of the superstructure.
2.6 Summary

This literature review has provided a summary of some of the accelerated bridge construction techniques used in the United States and the motivation for the development of the inverted T-beam system in Minnesota. Some of the connections between precast elements presented in this literature review are similar and were illustrated as a potential solution for future implementations of the inverted T-beam system. A summary of the work performed in Minnesota since the adoption of the Poutre-Dalle system has been provided. Table 2.5 gives a summary of the design details of the four generations of inverted T-beam bridges with the changes from the previous generation in bold. As can be seen from Table 2, two changes related to the precast inverted T-beam from Generation 1 to Generation 4 bridges included making precast flanges as thin as possible and increasing the chamfer size at the web and flange corners.

Other changes included making the top of the flanges smooth to aid in the removal of forms and pre-wetting the surfaces of the precast beam prior to the placement of cast-in-place topping. Although it is stated in the report that some degree of roughness is desired to avoid delamination at the interface, smooth flanges have not shown to create delamination problems. Some level of roughness is believed to be provided by the shallow air voids created during concrete placement.

Table 2.5 Typical PCSSS details from the four design generations with changes from the previous generation in bold. Actual details may differ for particular bridges.

<table>
<thead>
<tr>
<th>PCSSS Details</th>
<th>Generation 1</th>
<th>Generation 2</th>
<th>Generation 3</th>
<th>Generation 4</th>
</tr>
</thead>
<tbody>
<tr>
<td>Development Year</td>
<td>2005</td>
<td>2007</td>
<td>2009</td>
<td>2011</td>
</tr>
<tr>
<td>Inspected Field Bridges</td>
<td>13004</td>
<td>33005, 33006</td>
<td>40007, 40036</td>
<td></td>
</tr>
<tr>
<td>Precast Beam</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Flange Thickness at End</td>
<td>5 1/8 in.</td>
<td>5 1/8 in.</td>
<td>3 in.</td>
<td>3 in.</td>
</tr>
<tr>
<td>Roughened Flange Surfaces</td>
<td>Yes</td>
<td>Yes</td>
<td>No</td>
<td>No</td>
</tr>
<tr>
<td>Web Corner Chamfer</td>
<td>3/4 in.</td>
<td>3/4 in.</td>
<td>2 in.</td>
<td>2 in.</td>
</tr>
<tr>
<td>Flange Top Corner Chamfer</td>
<td>Yes</td>
<td>Yes</td>
<td>1 in.</td>
<td>1 in.</td>
</tr>
<tr>
<td>Pretwist Prior to Continuity</td>
<td>No</td>
<td>No</td>
<td>Yes</td>
<td>Yes</td>
</tr>
<tr>
<td>Bearing Pad</td>
<td>Elastomer, polystyrene under beams only</td>
<td>Elastomer, polystyrene under beams only</td>
<td>Elastomer, polystyrene under beams only</td>
<td>Elastomer, polystyrene under beams only</td>
</tr>
<tr>
<td>Trough Reinforcement Spacing</td>
<td>10 in.</td>
<td>10 in.</td>
<td>6 in.</td>
<td>6 in.</td>
</tr>
<tr>
<td>Joint Treatment</td>
<td>None</td>
<td>None</td>
<td>No. 3 bars</td>
<td>Mastic</td>
</tr>
<tr>
<td>Continuity Reinforcement</td>
<td>4 No. 8 at 6 in.</td>
<td>4 No. 8 at 5 in.</td>
<td>4 No. 8 at 5 in.</td>
<td>4 No. 8 at 5 in.</td>
</tr>
<tr>
<td>Deck Region</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Longitudinal Reinforcement</td>
<td>1 No. 8 and 2 No. 7 at 4 in. = 1 ft. total</td>
<td>1 No. 8 and 2 No. 7 at 4 in. = 1 ft. total</td>
<td>No. 7 at 4 in.</td>
<td>No. 7 at 4 in.</td>
</tr>
<tr>
<td>Transverse Reinforcement</td>
<td>No. 5 at 1 ft.</td>
<td>No. 5 at 1 ft.</td>
<td>No. 4 at 2 ft.</td>
<td>No. 4 at 2 ft.</td>
</tr>
<tr>
<td>Composites Action Stirrup</td>
<td>2 No. 5 at 1 ft.</td>
<td>2 No. 5 at 1 ft.</td>
<td>2 in.</td>
<td>2 in.</td>
</tr>
<tr>
<td>Stirrup Projection above Web</td>
<td>Not Specified</td>
<td>Not Specified</td>
<td>Not Specified</td>
<td>Not Specified</td>
</tr>
<tr>
<td>Dowels</td>
<td>At Piers</td>
<td>At Piers</td>
<td>At Piers</td>
<td>At Piers</td>
</tr>
<tr>
<td>At Joints</td>
<td>No. 5 at 12 in. between beam ends, center 50% of pier width</td>
<td>No. 6 at 9 in. in blockouts</td>
<td>No. 6 at 9 in. in blockouts</td>
<td>No. 6 at 9 in. in blockouts</td>
</tr>
<tr>
<td>Wrapping</td>
<td>3 No. 6 at 9 in. in blockouts</td>
<td>3 No. 6 at 9 in. in blockouts</td>
<td>3 No. 6 at 9 in. in blockouts</td>
<td>3 No. 6 at 9 in. in blockouts</td>
</tr>
</tbody>
</table>

1. Trough Reinforcement Spacing: Maximum spacing between any transverse reinforcement in the trough, either trough hooks or cage stirrups.
2. Joint Treatment: No. 3 bars tied to trough hooks to control reflective crack width; mastic bond breaker applied to longitudinal joint to prevent reflective crack initiation.
3. Outer 2/3rds: Two-thirds of the blockouts in each outboard portion of the deck had wrapped dowels, leaving the center one-third of the total bridge width with unwrapped dowels.
Although the original French concept has proved to provide an accelerated bridge construction alternative, signs of longitudinal and transverse cracking are still manifested in some cases. In addition, the accommodation of the transverse bars during the fabrication of the precast inverted T–beams is a challenge for the precaster during form installation and removal. The goal of the research presented in this study is to simplify the fabrication of precast inverted T-beams by investigating an alternative precast beam cross-sectional shape and various transverse connections while being able to emulate monolithic construction. Addressing the issue of longitudinal reflective cracking specifically and deck cracking in general, requires a proper evaluation of the causes that lead to cracking. In this study the effects of transverse bending and time dependent effects are evaluated individually and their effects quantified by taking the first inverted T-beam Bridge in Virginia as an example. After the causes of deck cracking are identified strategies for how to reduce the likelihood of cracking are proposed.

The introduction of the precast inverted T-beam bridge system in the state of Minnesota presented a unique cross-sectional shape for calculating stresses in the pre-tensioned anchorage zones and for determining the ability of the system to remain composite during service and ultimate loads. The literature review presented in this chapter described the efforts of the researchers at the University of Minnesota to provide design recommendations in these frameworks. The introduction of a new precast cross-sectional shape in Virginia presented the same challenges. Stresses in the pre-tensioned anchorage zones of the new precast shape are investigated and the existing design recommendations are evaluated. The ability of the new precast shape to remain composite with the cast-in-place topping and the required amount of extended stirrups is investigated. Finally, live load distribution factors are quantified analytically and by performing four live load tests on the US 360 Bridge.
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Chapter 3: Investigation of the Effects of Transverse Bending In the Inverted T-Beam Bridge System

(Originally published in 2012 PCI Convention and National Bridge Conference – Includes additional work)
Investigation of the Effects of Transverse Bending in Inverted T-Beam Bridge Systems

Abstract

The inverted T-beam system is a bridge system that provides an accelerated bridge construction alternative for short-to-medium-span bridges. The system consists of adjacent precast inverted T-beams finished with a cast-in-place concrete topping. This bridge system is intended to address reflective cracking problems manifested in short-to-medium-span bridges constructed with the traditional adjacent voided slab or adjacent box beam systems. When concentrated loads are applied to a bridge of this type, the bridge deforms as a two-way flat plate. This paper presents the results of an analytical and experimental investigation focused on the first inverted T-beam bridge in Virginia on US 360 over the Chickahominy River to study the relationship between transverse bending and reflective cracking. Transverse bending moment demands are quantified using a finite element model and compared to tested transverse bending moment capacities provided by several sub-assemblage specimens. The tested sub-assemblage specimens feature a combination of various precast inverted T-beam cross-sectional shapes and transverse connections. It is concluded that all tested specimens performed well at service load levels. The detail which features a precast inverted T-beam with tapered webs and no mechanical connection with the adjacent inverted T-beams and cast-in-place topping is the simplest and most economical.

3.1 Introduction

The supply of funds available to keep up with the demand for new bridge construction and rehabilitation is becoming scarce. The construction of new bridges and the rehabilitation of existing ones typically create detours or congestion in traffic, which in turn affect user’s costs. Therefore, state departments of transportation are faced with the challenge of finding new and innovative ways to address infrastructure needs while minimizing traffic re-routing or congestion. A report by the United States Department of Transportation states that: “New bridge systems are needed that will allow components to be fabricated offsite and moved into place for quick assembly while maintaining traffic flow. Depending on the specific site conditions, the use of prefabricated bridge systems can minimize traffic disruption, improve work zone safety, minimize impact to the environment, improve constructability, increase quality, and lower lifecycle costs. This technology is applicable and needed for both existing and new bridge construction”.

The precast inverted T-beam system is a composite bridge system intended for short-to-medium-span bridges that eliminates the installation of deck formwork. The system consists of adjacent precast inverted T-beams finished with a cast-in-place concrete topping (Figure 3.1). Figure 3.2 (a) shows the inverted T-beam system observed in France during a scanning tour in 2004 (Poutre-Dalle system) and Figure 3.2 (b) shows an alternative inverted T-beam system.
which will be implemented for the first time in Virginia on US 360 over the Chickahominy River. This bridge system is ideal for sites with stringent vertical clearance requirements because the combination of precast inverted T-beams and cast-in-place concrete topping represents the entire depth of the superstructure. Typically, short-to-medium span concrete bridges are constructed with adjacent voided slabs or adjacent box beam systems. Although these traditional systems promote faster construction schedules, they suffer from problems associated with the performance of joints between adjacent members. One of the causes of deterioration of such composite systems is reflective cracking, which occurs along the longitudinal joints. Typically, in bridges built with adjacent voided slabs or box beams, a cast-in-place topping or overlay is used to provide a continuous riding surface. Grouted shear keys are used at the joint between the precast members and are intended to prevent the adjacent components from moving relative to one another. However, often these grouted shear keys fail and monolithic action in the transverse direction is lost, which gives rise to reflective cracking. One of the causes of reflective cracking is the transverse bending of the bridge superstructure when subject to concentrated loads such as truck wheel loads. Transverse bending creates transverse tensile stresses below the neutral axis. In adjacent box structure systems without transverse post-tensioning, the resisting mechanism against these transverse tensile stresses consists primarily of the tensile strength of the bond between the grout in the shear key and the side of the precast beams. If the magnitude of the transverse tensile stresses exceeds the tensile strength of the bond the connection between the adjacent box members fails and reflective cracking may develop.

![Figure 3.1 Precast inverted T-beam system](image1)

(a) Poutre-Dalle system

![Figure 3.2 Deformed shapes in transverse direction](image2)

(b) Virginia’s Inverted T-beam system

Figure 3.1 Precast inverted T-beam system

Figure 3.2 Deformed shapes in transverse direction
Reflective cracking provides an avenue for water and deicing salts to penetrate the topping and corrode the reinforcing steel. In addition, reflective cracking reduces redundancy and increases live load distribution factors. Post-tensioning the voided slabs or adjacent box beams in the transverse direction helps emulate monolithic action but it also introduces an operation that needs to be carried out in the field, which reduces the advantages associated with accelerating bridge construction. Using a thicker topping or overlay also helps but it is not the most economical solution to address reflective cracking.

The inverted T-beam system offers two improvements. It provides a thicker cast-in-place topping over the joint so that any crack over the joint would have to travel through the entire thickness of the cast-in-place topping to reach the deck surface. It also provides a greater interface area between the precast and cast-in-place topping by creating a horizontal interface in addition to the vertical interface.

The precast inverted T-beam system will be used in the construction of a two-span continuous bridge, which carries US 360 over the Chickahominy river near Richmond, VA. The investigation described in this paper focuses on a combination of transverse connections and cross-sectional shapes of precast inverted T-beams to investigate their performance under service and failure loads so that recommendations could be made for the design and construction of the U.S.360 Bridge. Even though the study presented herein is centered on the U.S.360 Bridge many of the conclusions drawn apply to any short or medium span bridges for which the inverted T-beam system is used.

3.2 System Development

In 2004 the Federal Highway Administration sponsored a scanning tour to Europe and Japan to identify accelerated bridge construction systems that could be adopted in the United States. The group of engineers and researchers presented their findings in a document titled “Prefabricated Bridge Elements and Systems in Japan and Europe”\textsuperscript{1}. One of the systems that was selected as a potential system for implementation was the Poutre-Dalle System observed in France. Poutre-Dalle in French means Beam-Slab, which describes the system in question. The precast inverted T-beam in the original French system is shown in Figure 3.3 (a) and features transverse reinforcing steel with 180° hooks protruding from the webs of the precast inverted T-beams. The transverse reinforcing steel in adjacent beams overlaps and provides a connection with the cast-in-place concrete topping and adjacent inverted T-beams.

The Minnesota Department of Transportation (MnDOT) was the first in US to implement this system. MnDOT refers to the inverted T-beam system as Precast Concrete Slab Span System (PCSSS). Hagen et al.\textsuperscript{3} describe the first application of this system in two bridges in the state of Minnesota near Waskish and Center City. The system developed by MnDOT is similar to the French system with the exception that the bars extending from the webs have a 90° hook (Figure 3.3 (b) and (c)). The bars with the 90° hook facilitate the placement of a drop-in cage, which consists of stirrups and longitudinal bars (Figure 3.4). The stirrups provide additional reinforcing
in the transverse direction over the joint. The longitudinal bars serve as temperature and shrinkage reinforcement in addition to providing positive restraint moment reinforcing over the piers. The main challenge associated with the Poutre-Dalle system is the selection of forms that allow the transverse bars to protrude through the section.

Figure 3.3 Illustration of the inverted T-beam system. (a) French system (USDOT), (b) MnDOT precast inverted T-beam (Molnau and Dimaculangan), (c) Field installation of the precast inverted T-beams (Molnau and Dimaculangan)

Figure 3.4 Drop-in reinforcement (Hagen et al.)

Bell et al. instrumented the Center City Bridge to validate some of the assumptions used for the design of the system and to monitor the performance of the completed structure under traffic. One of the objectives of monitoring was to check for reflective cracking in the region between adjacent inverted T-beams. Smith et al. reported that longitudinal cracking had initiated in the cast-in-place concrete at mid-span where some of the transverse gages were located. This cracking was believed to be a result of restrained shrinkage and environmental effects rather than due to vehicular loads. In addition to the monitoring of the Center City Bridge, Smith et al. constructed and load tested a two-span laboratory specimen to investigate the effects of various parameters including variations in flange thickness. Test results indicated that the reduction in flange thickness to 3 in. improved transverse load distribution and that there was
no indication that the reduced flange was not durable enough for transportation and construction. Therefore it was recommended to use the 3 in. flange in future implementations of the MnDOT PCSSS\textsuperscript{5,6}.

Because the results from initial field implementation projects were relatively favorable, six more bridges were planned and constructed in Minnesota with the intent to improve the system\textsuperscript{7}. Dimaculangan and Lesch\textsuperscript{7} report that three “2\textsuperscript{nd} generation bridges” were designed and built in 2007. In 2009, three additional “3\textsuperscript{rd} generation bridges” were designed and constructed that included a large number of modifications with the hopes of reducing cracking. To phase in the design changes only a few of the proposed changes were incorporated in each generation of bridges\textsuperscript{7}.

French et al.\textsuperscript{8} continued the research on the inverted T-beam systems as part of an NCHRP study titled “Cast-in-Place Concrete Connections for Precast Deck Systems”. The NCHRP study included large-scale laboratory tests on two-span and simple-span inverted T-beam bridges, as well as seven sub-assemblage tests. The presence of a smooth flange surface was studied with the expectation that it would distribute the transverse tensile stresses throughout the bottom of the cast-in-place concrete, thereby reducing the potential stress concentration immediately above the longitudinal joint\textsuperscript{8}. However, even with the presence of a smooth flange it was determined that a single crack was present and that the transverse tensile stress was not distributed adequately well. In addition, it was concluded that a smooth flange would not be desirable as it would likely promote delamination of the horizontal precast flange cast-in-place concrete interface. While a 9 in. maximum transverse spacing appeared to be sufficient to control cracking in the sub-assemblage study it did not correlate to an improvement in crack control relative to the 12 in. transverse spacing in the simply-supported laboratory bridge specimen\textsuperscript{8}.

The NCHRP study culminated in recommended design and construction specifications for the inverted T-beam system. The design recommendations related to reflective cracking included equations for determining the amount and spacing of transverse load distribution reinforcing and reflective crack control reinforcing. Transverse load distribution reinforcing was considered to include the transverse hooked bars that extend through the precast webs and lap with companion hooked bars protruding from the adjacent precast webs. It was recommended that only one of the hooked bars in the lap be used in transverse load distribution reinforcement calculation because the lapped bars must transfer load to each other between the adjacent panels\textsuperscript{8}. The NCHRP report recommends that Equation 1 be used to calculate the area of transverse load distribution reinforcing. This equation combines the percentages of longitudinal mild and prestress flexural reinforcement that need to be provided in the transverse direction based on AASHTO LRFD Equations 5.14.4.1-1 and 5.14.4.1-2, respectively. Because the AASHTO equations were developed for cast-in-place slab systems in which the transverse reinforcement is located immediately above the longitudinal reinforcement, an adjustment is made to account for the fact that the depths to the center of gravities for the longitudinal prestress and transverse load distribution reinforcement could be significantly different\textsuperscript{8}. This adjustment is made multiplying the second part of Equation 1 with α, which is defined below.
\[ A_{\text{tld}} = k_{\text{mild}} A_{l-\text{mild}} + \alpha k_{ps} A_{l-ps} \]  
\[ k_{\text{mild}} = \frac{100}{\sqrt{L}} \leq 50\% \]  
\[ k_{ps} = \frac{100 f_{pe}}{\sqrt{L}} \leq 50\% \]  
(1)  
(2) \quad \text{– (LRFD 5.14.4.1 – 1)}  
(3) \quad \text{– (LRFD 5.14.4.1 – 2)}

where:

\( A_{\text{tld}} \) = area required for transverse load distribution reinforcement (in\(^2\))

\( A_{l-\text{mild}} \) = area of longitudinal mild flexural reinforcement (in\(^2\))

\( A_{l-ps} \) = area of longitudinal prestressed flexural reinforcement (in\(^2\))

\( k_{\text{mild}} \) = percentage of longitudinal mild flexural reinforcement

\( k_{ps} \) = percentage of longitudinal prestressed flexural reinforcement

\( \alpha = \frac{d_{cgs}}{d_{\text{trans}}} \geq 1.0 \)

\( d_{cgs} \) = depth of center of gravity of prestressed reinforcement (in.)

\( d_{\text{trans}} \) = depth of center of gravity of transverse reinforcement (in.)

\( L \) = span length (ft)

\( f_{pe} \) = effective stress in prestressing strand (ksi)

On the other hand, reflective crack control reinforcing is considered to include both lapped transverse hooked bars plus the lower leg of the cage hoops. It is noted that the drop-in-cage is only effective in controlling cracks that develop in the vicinity over the longitudinal joint and that cracks that might form at the interface between the cast-in-place topping and precast web would only be restrained by the reinforcement provided for transverse load distribution\(^8\).

The recommended spacing for crack control reinforcing included an evaluation of existing provisions in AASHTO LRFD Bridge Design Specifications\(^9\), ACI 318-08\(^10\) and recommendation by Frosch et al.\(^11\) related to crack control in bridge decks. These recommendations are summarized in Table 3.1. Based on the test results of large scale laboratory bridges and sub-assembleage specimens French et al.\(^8\) concluded that the amount of reflective crack control reinforcing should be based on the equation recommended by Frosch et al.\(^11\) and that maximum spacing could be based on AASHTO’s equation bounded to a maximum spacing of 12 in.

According to the design guide in the NCHRP report\(^6\), it is recommended that when using the equation recommended by Frosch et al.\(^11\), the gross area of section be taken as the distance between the top of the precast flange and the top of the precast web times the width under consideration. This approach assumes that the entire cast-in-place topping over the joint, from the top of the precast flange to the top of the precast web, will be cracked under a tensile stress of \(6\sqrt{f'_{c}}\) (where \(f'_{c}\) is in psi) and that enough steel should be provided to prevent yielding. In addition, \(d_c\), in AASHTO’s equation, should be taken equal to the distance between the top of the joint to the center of transverse steel.
Halverson et al.\textsuperscript{12} evaluated the performance of existing bridges built with precast inverted T-beams and reviewed a number of design details. Field inspections were conducted for five existing inverted T-beam bridges, consisting of crack mapping surveys and core sample examinations to evaluate design changes related to reflective crack control\textsuperscript{12}. Reflective cracking was observed on both Bridge No. 13004 near Center City, MN and Bridge No. 33008 near Mora, MN. The latter exhibited long surface cracks over a number of longitudinal joints and two reflective cracks determined from the observation of core specimens. Chamfering the corners of the precast web and flange was considered favorable because it reduces the severity of the discontinuity by increasing the surface area over which stresses in the cast-in-place topping develop at these points.

**Table 3.1** Spacing and reinforcement ratio limits for flexural and crack control reinforcement (French et al.\textsuperscript{8})

<table>
<thead>
<tr>
<th>Reinforcement Type</th>
<th>Reinforcement Design Limits</th>
<th>Source/Article in Spec.</th>
</tr>
</thead>
<tbody>
<tr>
<td>Crack control and shrinkage and temperature</td>
<td>( s &lt; \frac{700\gamma_f}{\beta_s f_{fs} - 2_{dc}} ), where ( \beta_s = 1 + \frac{d_c}{0.7(n - d_c)} )</td>
<td>AASHTO (2010) 5.7.3.4</td>
</tr>
<tr>
<td></td>
<td>( s \leq \text{min}(1.5h,18 \text{ in.}) )</td>
<td></td>
</tr>
<tr>
<td>Shrinkage and temperature</td>
<td>( A_{\text{AASHTO}} \geq \frac{1.30bh}{(b + h)f_y} ; 0.11 \leq A_{\text{AASHTO}} \leq 0.60 )</td>
<td>5.10.2</td>
</tr>
<tr>
<td>Shrinkage and temperature</td>
<td>( A_{\text{ACI}} = 0.091bhh ) (when using Gr. 60 bar's)</td>
<td>ACI 318-08 7.12.2.1</td>
</tr>
<tr>
<td></td>
<td>( s \leq \text{min}(5h,18 \text{ in.}) )</td>
<td></td>
</tr>
<tr>
<td>Crack control and shrinkage and temperature</td>
<td>( s &lt; 15 \cdot \frac{40000}{f_s} - 2.5 \cdot c_c )</td>
<td>10.6.4</td>
</tr>
<tr>
<td>Shrinkage and temperature</td>
<td>( A_{\text{ACI}} = 0.091bhh ) (when using Gr. 60 bar's)</td>
<td>ACI 318-08 7.12.2.2</td>
</tr>
<tr>
<td>Crack control</td>
<td>( s &lt; 9 \cdot \left(2.5 - \frac{c_c}{2}\right) \leq 9\text{ in.} )</td>
<td>Frosch et al. (2006)</td>
</tr>
<tr>
<td></td>
<td>( \rho_{\text{Frosch}} = 6\sqrt{\frac{f_c}{f_y}} )</td>
<td></td>
</tr>
</tbody>
</table>

\textsuperscript{1}Variables in Table 3.1 are defined as follows:
- \( \gamma_f \) = 1.0 for Class 1 (No corrosion concerns); 0.75 for Class 2 (Corrosion concerns)
- \( f_{fs} \) = Stress in reinforcement at service [ksi]
- \( d_c \) = Depth of cover measured from tension fiber of concrete to center of reinforcement, defined by AASHTO (2010) Section 5.7.3.4 [in.]
- \( c_c \) = Depth of cover measured from tension fiber of concrete to face of reinforcement, defined by ACI 318-08 Section 10.6.4 [in.]
- \( h \) = Total section depth [in.]
- \( A_{\text{AASHTO}} \) = Area of reinforcement in each direction and each face [in.\(^2\)/ft.]
- \( A_{\text{ACI}} \) = Area of shrinkage and temperature reinforcement [in.\(^2\)]
- \( b \) = Least width of component section [in.]
- \( f_y \) = Stress in reinforcement at service [ksi]
- \( f_c \) = 28-day concrete compressive strength [ksi]
- \( f_y \) = Reinforcement yield strength [ksi]
- \( \rho_{\text{Frosch}} \) = Reinforcement ratio defined by Frosch et al. (2006). Equivalent to:
  area of reinforcement/gross area of section [dim]
Table 3.2 provides a summary of the design details of the four generations of inverted T-beam bridges with the changes from the previous generation in bold. As can be seen from Table 2, two changes related to the precast inverted T-beam from Generation 1 to Generation 4 bridges included making precast flanges as thin as possible and increasing the chamfer size at the web and flange corners. Other changes included making the top of the flanges smooth to aid in the removal of forms and pre-wetting the surfaces of the precast beam prior to the placement of cast-in-place topping. Although it is stated in the report that some degree of roughness is desired to avoid delamination at the interface, smooth flanges have not shown to create delamination problems. Some level of roughness is believed to be provided by the shallow air voids created during concrete placement.

Table 3.2 Typical Precast Concrete Slab Span System (PCSSS) details from the four design generations with changes from the previous generation in bold. Actual details may differ for particular bridges (Halverson et al.12)

<table>
<thead>
<tr>
<th>PCSSS Details</th>
<th>Generation 1</th>
<th>Generation 2</th>
<th>Generation 3</th>
<th>Generation 4</th>
</tr>
</thead>
<tbody>
<tr>
<td>Development Year</td>
<td>2005</td>
<td>2007</td>
<td>2009</td>
<td>2011</td>
</tr>
<tr>
<td>Inspected Field Bridges Nos.</td>
<td>13004</td>
<td>33005, 33008</td>
<td>49007, 49008</td>
<td>None inspected</td>
</tr>
<tr>
<td>Flange Thickness at End</td>
<td>5 ¼ in.</td>
<td>5 ¼ in.</td>
<td>3 in.</td>
<td>3 in.</td>
</tr>
<tr>
<td>Roughened Flange Surfaces</td>
<td>Yes</td>
<td>Yes</td>
<td>No</td>
<td>No</td>
</tr>
<tr>
<td>Web Corner Chamfer</td>
<td>¾ in.</td>
<td>¾ in.</td>
<td>2 in.</td>
<td>2 in.</td>
</tr>
<tr>
<td>Flange Top Corner Chamfer</td>
<td>¾ in.</td>
<td>¼ in.</td>
<td>1 in.</td>
<td>1 in.</td>
</tr>
<tr>
<td>Prestressed Prior to Continuity</td>
<td>No</td>
<td>No</td>
<td>Yes</td>
<td>Yes</td>
</tr>
<tr>
<td>Bearing Pad</td>
<td>Elastomer, polystyrene under beams only</td>
<td>Elastomer, polystyrene under beams only</td>
<td>Elastomer, polystyrene under beams only</td>
<td>Elastomer, polystyrene over entire pier cap</td>
</tr>
<tr>
<td>Trough</td>
<td>Trough Reinforcement Spacing&lt;sup&gt;1&lt;/sup&gt;</td>
<td>10 in.</td>
<td>10 in.</td>
<td>6 in.</td>
</tr>
<tr>
<td>Joint Treatment&lt;sup&gt;2&lt;/sup&gt;</td>
<td>None</td>
<td>None</td>
<td>No. 3 bars</td>
<td>Mastic</td>
</tr>
<tr>
<td>Continuity Reinforcement</td>
<td>4 No. 8 at 6 in.</td>
<td>4 No. 8 at 5 in.</td>
<td>4 No. 8 at 5 in.</td>
<td>4 No. 8 at 5 in.</td>
</tr>
<tr>
<td>Dock Region</td>
<td>Longitudinal Reinforcement</td>
<td>1 No. 8 and 2 No. 7 at 4 in. = 1 ft. total</td>
<td>1 No. 8 and 2 No. 7 at 4 in. = 1 ft. total</td>
<td>No. 7 at 4 in.</td>
</tr>
<tr>
<td></td>
<td>Transverse Reinforcement</td>
<td>No. 5 at 1 ft.</td>
<td>No. 5 at 1 ft.</td>
<td>No. 4 at 6 in.</td>
</tr>
<tr>
<td></td>
<td>Composite Action Stirups</td>
<td>2 No. 5 at 1 ft.</td>
<td>4 No. 4 at 2 ft.</td>
<td>2 No. 4 at 2 ft.</td>
</tr>
<tr>
<td></td>
<td>Stirrup Projection above Web</td>
<td>Not Specified</td>
<td>2 in.</td>
<td>2 in.</td>
</tr>
<tr>
<td>Dovels</td>
<td>At Piers</td>
<td>No. 5 at 12 in. between beam ends, center 50% of pier width</td>
<td>6 No. 6 at 9 in. in blockouts</td>
<td>4 No. 8 at 12 in. in blockouts</td>
</tr>
<tr>
<td></td>
<td>At Abutments</td>
<td>3 No. 6 at 9 in. in blockouts</td>
<td>3 No. 6 at 9 in. in blockouts</td>
<td>2 No. 8 at 12 in. in blockouts</td>
</tr>
<tr>
<td></td>
<td>Wrapping</td>
<td>None</td>
<td>None</td>
<td>Outer 2/3rd&lt;sup&gt;3&lt;/sup&gt;</td>
</tr>
</tbody>
</table>

1. Trough Reinforcement Spacing: Maximum spacing between any transverse reinforcement in the trough, either trough hooks or cage stirrups.
2. Joint Treatment: No. 3 bars tied to trough hooks to control reflective crack width; mastic bond breaker applied to longitudinal joint to prevent reflective crack initiation.
3. Outer 2/3rd: Two-thirds of the blockouts in each outward portion of the deck had wrapped dovels, leaving the center one-third of the total bridge width with unreinforced dovels.

Although the original French concept has proved to accelerate bridge construction and improve the performance of short-to-medium-span composite bridges against reflective cracking, the fabrication of precast inverted T-beams with the transverse hooked bars protruding from the precast webs provides a challenge for the precaster during form installation and removal. The motivation behind the research presented in this paper was to simplify the fabrication of precast inverted T-beams by eliminating the need for extended transverse bars while emulating monolithic construction in the completed bridge. To accomplish this goal an alternative precast beam cross-sectional shape and two different transverse connection details are investigated.
3.3 Implementation of the Inverted T-beam System in Virginia

Being aware of reflective cracking problems present in short-to-medium-span bridges built with adjacent voided slabs and adjacent box girder systems, the Virginia Department of Transportation expressed interest in implementing the precast inverted T-beam system for the first time in Virginia. The application was a bridge replacement project near Richmond, VA on US 360 over the Chickahominy River and featured four bridges. Three of these bridges were targeted to be replaced with the traditional adjacent voided slab system and one of them with the new inverted T-beam system. In addition, the bridge that was targeted for replacement using the inverted T-beam system was identical in terms of number of spans, span lengths, bridge width, traffic volume and environmental conditions with one of the neighboring bridges, which was scheduled to be replaced using the traditional adjacent voided slab system. Both of these bridges were two span continuous bridges with span lengths approximately 43 ft. This provided an opportunity to observe the relative performances of these two bridges over time. A preliminary plan view and elevation of the two-span continuous inverted T-beam Bridge is shown in Figure 3.5. The depth of the superstructure consisting of the inverted T-beams and the cast-in-place topping is limited to 25 in. The depth of the precast inverted T-beams is 18 in. and the depth of the cast-in-place concrete topping is 7 in. There are three lanes in each direction and a median. The clear width from edge of barrier to edge of barrier is 110 ft. Because transverse bending moments are one of the causes of reflective cracking, their quantification was of interest.

3.4 Analytical Investigation

Two finite element models of the U.S.360 Bridge were created in Abaqus to determine the worst case transverse bending moment. One model featured precast inverted T-beams with straight webs and the other included precast inverted T-beams with tapered webs. To obtain the maximum transverse bending moment created in the bridge, a linear elastic analysis, which is appropriate up to the initiation of cracking, was performed using 3D solid elements and un-cracked concrete properties for both the precast and cast-in-place components. The quantification of the worst case transverse bending moment was done by systematically placing the live load (combination of truck and lane load or tandem and lane load) on the bridge and by monitoring the magnitude of the transverse moments. The stresses created in the cast-in-place topping due to design live loads were smaller than the modulus of rupture regardless of the mesh size. The bond between the precast inverted T-beams and cast-in-place topping was assumed to be a perfect bond. The worst case transverse bending moment including the dynamic load allowance was determined to be 14.5 ft-kips/ft.

Figure 3.6 illustrates the two-way plate bending behavior of the US 360 Bridge superstructure when subject to concentrated loads such as truck wheel loads. The transverse section demonstrates the effect of transverse bending. Figure 3.7 shows a close up view of the longitudinal joint area, which is the most vulnerable to cracking as a result of transverse bending, because the presence of a joint between the precast beams creates a weakened plane.
Figure 3.8 zooms in on the transverse section and shows how the effects of transverse bending on the bridge can be investigated by using a simply supported beam setup in the laboratory. Figure 3.9 illustrates the orientation of principal stress vectors in the full bridge model and in the sub-assemblage model for the case when the precast inverted T-beam has tapered webs. As can be seen from the similarity of the orientation of principal stress vectors in both models, the simply supported sub-assemblage specimen can represent the stress condition around the longitudinal joint in the full bridge fairly well. In both models the orientation of the principal tensile stress vectors below the neutral axis is horizontal, which indicates that normal stresses dominate over shear stresses. Figure 3.10 illustrates the similarity in the orientation of principal stress vectors in the full bridge model and the sub-assemblage model when the precast webs are straight rather than tapered. Because the orientation of principal tensile stress vectors below the neutral axis is horizontal, the resistance against cracking at the vertical interface between the precast beam and the cast-in-place concrete topping in the case when the precast beam has tapered webs is provided by the combination of the shear and tensile strength of the bond. Conversely, in the case when the precast beam has straight webs the resistance against cracking at the vertical interface is provided predominantly by the tensile strength of the bond. Because the shear strength of an interface between two concretes cast at different times is higher than its tensile strength the precast inverted T-beam with tapered webs promises to provide an enhanced resistance against cracking caused by transverse bending. Applied and resisting forces at the vertical precast web cast-in-place topping interface will be illustrated in detail in the subsequent sections.

Because the simply supported sub-assemblage specimen could replicate fairly well the stress state around the longitudinal joint observed in the full bridge model, several sub-assemblage specimens were tested to compare their capacity in transverse bending with that demanded during service.
Figure 3.5 Preliminary plan and elevation of the U.S.360 Bridge over the Chickahominy River\textsuperscript{13}
**Figure 3.6** Isometric view of plate bending in bridge deck and a transverse section (deformed shape)

**Figure 3.7** Transverse normal stress contours (S11) around the longitudinal joint in the full bridge model

**Figure 3.8** Transverse normal stresses contours (S11) around the joint in the sub-assemblage specimen model
Figure 3.9 Principal stress vectors, (a) Full model, (b) Sub-assemblage model

Figure 3.10 Principal stress vectors, (a) Full model, (b) Sub-assemblage model
3.5 Experimental investigation – Phase I

Figure 3.11 illustrates the concept behind the selection of the simply supported beam setup, which features two adjacent precast inverted T-beams and the associated concrete topping. The precast web was extended to replace the precast flanges at the supports to create a better bearing condition. The two point loads represent either tandem loading or HL-93 truck loading. Figure 3.12 shows a photograph of the test setup featuring a loading frame, actuator, spreader beam and two steel beams providing supports for the test specimens. The two point loading was applied by distributing the actuator load using the spreader beam on two tire prints. This loading arrangement created a region of constant moment and zero shear due to actuator loads in the region around the longitudinal joint, which is consistent with the fact that the orientation of principle tensile stress vectors in the full bridge model was mainly horizontal. The specimens were supported on 6 in. wide by 0.75 in. thick neoprene bearing pads. The superstructure for the two-span continuous U.S.360 Bridge was designed per AASHTO LRFD Bridge Design Specifications and the reinforcing for the sub-assemblage specimens was selected accordingly. A finite element model of the sub-assemblage test specimen was used to determine the actuator force ($P_{\text{actuator}}$ in Figure 3.11) that would create the worst case transverse bending moment during service computed from the finite element model of the full bridge. This load was found to be 27 kips and is in addition to the self-weight of the test specimen.

![Diagram](image)

Figure 3.11 Sub-assemblage test specimen concept

The design compressive strength at 28 days for the cast-in-place concrete topping and precast beams were $f'_{c} = 4000$ psi and $f'_{c} = 8000$ psi, respectively. All mild reinforcing steel was ASTM A615, Grade 60, deformed bare steel. Prestressing strands were ASTM A416, Grade 270,
uncoated, low-relaxation strands. The prestressing strands in the test specimens were not prestressed because they do not have a significant influence on the behavior of the specimens in the transverse direction.

**Figure 3.12** Photograph of test setup

All surfaces of the precast inverted T-beams in contact with the cast-in-place topping were roughened in the longitudinal direction of the bridge to a 1/4 in. amplitude to emulate monolithic action in the transverse direction. The roughened surface in the side of the webs and the top of the precast flange was created by introducing grooves in the wooden formwork. The roughened surface at the top of the web was created by raking the fresh concrete immediately after placement to a 1/4 in. amplitude. Figure 3.13(a) shows a photograph of the roughened surface in the precast members. Figure 3.13(b) shows the profile of the corrugated wood forms used to create the roughened surface on the side of the webs and top of the precast flange. An alternative profile is shown in Figure 3.13(c) and features corrugations with tapered sides, which are intended to facilitate the removal of forms. Heavy surface retarding chemicals may be another way of creating an acceptably rough surface.

During a trial pour it was observed that concrete did not flow all the way to the edge of the precast flanges. This was attributed to the fact that the flanges were formed on all sides without providing any air pressure relief during concrete placement. To correct this, an air relief strip was provided in the subsequent pours. In addition coarse aggregates were changed from No.57 stone to No.78 stone to facilitate the flow of fresh concrete through the 3 in. precast flange Figure 3.14 shows the two precast pieces which were used in constructing the trial specimen. As can be seen, the shape of the flange could not be formed as intended. However, this specimen was tested to see how such a specimen with incomplete or damaged flanges would perform, in case situations like this were to occur during the fabrication of precast inverted T-beams.

**3.5.1 Investigation of Two Precast Beam Cross-sectional Shapes**

In this experimental study two cross-sectional shapes were investigated. These are shown in Figure 3.15. The first cross-sectional shape is similar to the original French detail and also to the one used by MnDOT. It features a precast inverted T-beam with straight vertical webs. In the second cross-sectional shape the webs of the precast inverted T-beam were tapered with the
purpose of providing a higher bond strength against normal transverse tensile stresses at the precast web cast-in-place topping interface. As can be seen from Figure 3.16, when transverse bending takes place in the bridge, normal tensile stresses are created in the transverse direction below the neutral axis in the precast web region. In the cross-sectional shape with the straight webs the resistance against interface bond failure at the precast web is mainly limited to the bond strength in tension between the precast web and cast-in-place concrete topping. This phenomenon is described in Figure 3.16 (a)-(d). Figure 3.16 (a) and (b) illustrates the application of the transverse tensile stresses at the precast web cast-in-place topping interface in the systems with a straight precast web and tapered precast web, respectively. In both systems the interface was intentionally roughened as described earlier. Figure 3.16 (c) and (d) zooms in on a typical roughened surface pattern and illustrates the resisting mechanisms against interface bond failure.

**Figure 3.13** (a) Photograph of roughened surface, (b) profile of corrugated wood forms used to create the roughened surface, (b) alternative profile

**Figure 3.14** Trial Specimen with straight web, extended bars and incomplete flanges.
In the precast beam with straight webs, such a resisting mechanism consists primarily of the tensile bond strength between the precast and cast-in-place component and slightly on the shear bond strength at the horizontal surfaces of the corrugated pattern (Figure 3.16 (c)). In the
precast beam with tapered webs, the resisting mechanisms relies on the combination of the shear and tensile bond strength between the two components as well as on the mechanical interlock offered by the roughened surface in the tapered webs. Figure 3.16 (e) illustrates the resistance against interface bond failure in a precast beam with tapered webs but with the alternative roughened surface profile illustrated earlier. In this case, the mechanical interlock between the precast web and cast-in-place topping is lost, however the interface bond strength against transverse bending is still likely to be higher than the system with straight precast webs, because the resisting mechanism consists predominately of the combined shear and tensile strength of the bond.

3.5.2 Investigation of Three Transverse Connections

Another key aspect of the inverted T-beam system is the connection between the precast components and cast-in-place topping in the transverse direction. Therefore three different connection details were investigated as shown in Figure 3.17. The first connection detail is similar to the one used by MnDOT, which features extended bars from the sides of the precast webs (Figure 3.17 (a)).

Figure 3.17 The Three Connections investigated
The advantage of this detail is that the extended bars provide a mechanical connection between the precast component and cast-in-place topping. They also serve as a continuous tension tie in the transverse direction of the bridge. This detail minimizes the amount of work done at the construction site because the placement of concrete topping and reinforcing steel are the only activities required to deliver a bridge superstructure that will mimic monolithic construction in both directions. The disadvantage of this detail is that it creates challenges for the precaster because the formwork for the webs of the precast inverted T-beams needs to accommodate the reinforcing steel protruding from the webs.

The second connection detail features discrete embedded steel plates in the precast flanges at 2 ft – 0 in. on center and welded reinforcing bars (Figure 3.17 (b)). This detail does not have any reinforcing steel protruding from the precast webs. Instead, the precast flanges are connected by field welding a drop-in piece of reinforcing steel to each embedded steel plate. The orientation of the embedded steel plate is inclined so that it can receive the drop-in bar as well as accommodate any differences in elevation due to construction tolerances between the precast inverted T-beams. Each embedded steel plate is welded to two reinforcing steel bars which run for the entire width of the precast beam and are welded to the embedded steel plate on the other side. Alternatively, each embedded steel plate can have its own set of transverse reinforcing steel bars welded to the back of it to provide some tolerance during fabrication. These transverse bars can then be tension spliced with those coming from the embedded steel plate on the opposite side. One advantage of this detail is the shift in location of the tension tie towards the bottom of the precast, where the tensile stresses as a result of transverse bending are maximum. This detail avoids having a complete separation between the precast flanges. Another advantage of this detail is the relative ease of forming the precast inverted T-beams when compared to the original French detail, in which the forms need to accommodate the protruding bars. One of the disadvantages is the field welding, which goes against the concept of accelerating bridge construction because it adds an operation which needs to be done in the field.

The third connection detail relies solely on the bond between the cast-in-place topping and the precast inverted T-beam to emulate monolithic construction (Figure 3.17 (c)). This detail is the simplest and most economical because it does not have any reinforcing steel protruding from the sides of the webs. It also does not have a mechanical connection between the precast members, which takes additional time in the field. Although all surfaces of the precast beam in contact with the cast-in-place topping should be intentionally roughened regardless of which detail is used, it is particularly important that this is done when the detail with no mechanical connection is selected. It is through the cohesion provided by the roughened surface that monolithic action is emulated.

3.5.3 Investigation of Five Sub-assemblage Specimens

The two cross-sectional shapes and three connections were combined to produce a total of five specimens with different configurations and details. The depths of all specimens were
selected such that they matched the overall depth of the bridge superstructure that will be used on the U.S.360 Bridge. The overall length of each specimen is 12 ft, which is equal to the width of two adjacent precast inverted T-beams. The width of the specimens was selected to be 4 ft - 0 in., which is a multiple of the spacing of the embedded plate connectors with the welded rebars (24 in.) and the spacing of the extended bars from the precast webs (12 in.). Table 3.3 shows the test specimen matrix for experimental phase I.

Table 3.3 Test Specimen Matrix for Experimental Phase I

<table>
<thead>
<tr>
<th>Specimen ID</th>
<th>Cross-sectional shape</th>
<th>Connection</th>
<th>Transverse bottom reinforcing in CIP trough</th>
<th>Transverse bottom reinforcing in Precast</th>
<th>Loading</th>
</tr>
</thead>
<tbody>
<tr>
<td>Trial</td>
<td>Straight web</td>
<td>Extended bars</td>
<td>No.6 at 12 in. plus No.4 stirrups at 12 in</td>
<td>No.6 at 12 in. hooked bars plus No.3 stirrups at 18 in.</td>
<td>¼ point</td>
</tr>
<tr>
<td>1</td>
<td>Straight web</td>
<td>Extended bars</td>
<td>No.6 at 12 in. plus No.4 stirrups at 12 in</td>
<td>No.6 at 12 in. hooked bars plus No.3 stirrups at 18 in.</td>
<td>¼ point</td>
</tr>
<tr>
<td>2</td>
<td>Straight web</td>
<td>Embedded plate and welded rebar</td>
<td>None</td>
<td>4-No.6 bars</td>
<td>¼ point</td>
</tr>
<tr>
<td>3</td>
<td>Tapered web</td>
<td>Embedded plate and welded rebar</td>
<td>No.3 at 12 in</td>
<td>4-No.6 bars</td>
<td>¼ point</td>
</tr>
<tr>
<td>4</td>
<td>Tapered web</td>
<td>No connection</td>
<td>No.6 at 12 in</td>
<td>No.3 at 18 in</td>
<td>¼ point</td>
</tr>
</tbody>
</table>

3.5.3.1 Trial Specimen and Specimen No.1

The trial specimen and Specimen No.1 were identical with the exception that the precast flanges in the trial specimen were incomplete due to the absence of an air relief strip in the forms for the precast flanges. Both of these specimens are similar in concept to the detail used by MnDOT. Figure 3.18 provides information on the reinforcing details. The transverse bars in the precast flange were sized to support the weight of the wet cast-in-place concrete topping. The size and spacing of the hooked bars and the pre-tied cage were based on the recommendations by French et al.8. The remaining reinforcing steel in the cast-in-place topping was based on minimum requirements for temperature and shrinkage.
Figure 3.18 Reinforcing details for Trial Specimen and Test Specimen No.1

3.5.3.2 Specimen No.2 This specimen has the same cross-sectional shape as Specimen No.1. Reinforcing details are provided in Figure 3.19. There is no reinforcing steel protruding from the webs of the precast inverted T-beams. Instead, the flanges of the inverted T-beams were connected by welding a 6 in. long piece of reinforcing steel to an inclined steel plate embedded in the precast flange. The size of the transverse bars in the precast flange was based on the transverse moment at service created as a result of live loads using allowable stress design principles and ignoring the contribution of concrete in tension. The allowable stress for the reinforcing steel was taken equal to 30 ksi. The area of transverse steel in the precast flange calculated based on the transverse live load moment at service was similar to the area of transverse load distribution reinforcing calculated based on Equation 1. Accordingly, Equation 1 proposed by French et al. predicted fairly well the demand for transverse steel. Reinforcing steel in the cast-in-place topping was based on minimum requirements for temperature and shrinkage.

Figure 3.19 Reinforcing details for Test Specimen No.2
3.5.3.3 Specimen No.3 In this specimen the connection between the precast flanges is identical to the one used in the Specimen No.2. Unlike Specimen No.2, this specimen has a tapered cross-sectional shape. Another difference is that the bottom layer of deck steel in this specimen is detailed such that it follows the shape of the cast-in-place topping as opposed to the specimen with the straight web where this layer was straight. Similar to Specimen No.2, the amount of transverse steel in the precast flanges was based on the transverse live load moment at service. The No.3 bent bars at 12 in. on center at the bottom of the cast-in-place topping were not relied upon when calculating the transverse flexural strength of the specimen, but were provided as temperature and shrinkage reinforcing together with the top layer of No. 4 at 12 in. on center. Reinforcing details are provided in Figure 3.20.

![Diagram of reinforcing details for Test Specimen No.3](image)

**Figure 3.20** Reinforcing details for Test Specimen No.3

3.5.3.4 Specimen No.4 Finally, the specimen shown in Figure 3.21 differs from the other specimens because it has no mechanical connection between the inverted T-beams and the cast-in-place topping. It is believed that composite action in the transverse direction will be achieved due to the bond between the roughened surface in the precast inverted T-beam and the cast-in-place concrete topping. Similar to Specimens No.2 and No.3, the transverse bottom steel in the cast-in-place topping was based on the transverse live load moment at service. However, the transverse reinforcing in the precast flanges, was only designed to resist the weight of the wet cast-in-place concrete topping. In this specimen, a complete tension tie can be developed only if the tensile force resisted by the bottom layer of bars in the deepest portion of the cast-in-place topping (No. 6 at 12 in. on center) can be transferred via a non-contact splice to the transverse bars in the precast inverted T-beam (No. 3 at 18 in. on center). Clearly, the weak link in this case is the flexural capacity of the composite section in the transverse direction provided by the No. 3 at 18 in. on center.
3.5.4 Test protocol

Each specimen was loaded in increments of 5 kips up to 30 kips, which was slightly over the service level load ($P_{\text{service}}$) of 27 kips. Subsequently, the load was increased in 10 kip increments up to first cracking. First cracking load ($P_{\text{cr}}$) was taken equal to the actuator load that caused the first crack. The width of the first crack was recorded and the specimens were subjected to five cycles of loading with the maximum load being the load that caused the first crack and the minimum load being equal to zero kips. At the end of five cycles the crack width was re-measured to determine whether there was any increase in the crack width. The crack length was also monitored to determine whether the crack propagated as a result of the five load cycles. Then the specimen was subject to three more load cycles and the crack measurement procedure was repeated. Monitoring of crack widths and propagation was done for the 9th and 10th load cycles, at the end of which repeated loading was terminated if it were determined that there was no increase in crack width or any crack propagation. If at the end of the 10th cycle the specimen showed signs of crack growth or propagation then the specimen was subjected to additional load cycles until crack stability was achieved. After the repeated loading was terminated, the specimens were loaded monotonically up to failure or up until the capacity of the loading frame was reached ($P_u$). The capacity of the loading frame was 300 kips. The load step after the repeated loading was 10 kips and at every load step the crack width was recorded and the crack pattern was marked on the specimens.

3.6 Test results from Experimental Phase I

The results of the five sub-assembleage tests are summarized in Table 3.4. The second column shows the load at first cracking. The third column shows either the actuator load at failure or the capacity of the loading frame, whichever was met first. The fourth column shows the factor of safety against cracking, which is calculated based on Equation 4. In the case of the specimen with the tapered web and embedded plate connection and the trial specimen with the straight web and extended bars, the ultimate load could not be achieved because the capacity of

Figure 3.21 Reinforcing details for Test Specimen No.4
the loading frame was reached before the specimens failed. The last column shows the factor of safety at failure, which is calculated based on Equation 5.

\[
FS_{\text{cracking}} = \frac{(M_{\text{cracking}} + M_{\text{selfweight}} + M_{\text{spreader beam}})}{M_{\text{service}}} \tag{4}
\]

\[
FS_{\text{ultimate}} = \frac{(M_{\text{ultimate}} + M_{\text{selfweight}} + M_{\text{spreader beam}})}{M_{\text{service}}} \tag{5}
\]

where:
- \(FS_{\text{cracking}}\) = Factor of safety against cracking
- \(FS_{\text{ultimate}}\) = Factor of safety at failure
- \(M_{\text{cracking}}\) = Moment at mid-span due to actuator load at first cracking \((P_{cr})\)
- \(M_{\text{ultimate}}\) = Moment at mid-span due to actuator load at failure \((P_u)\)
- \(M_{\text{selfweight}}\) = Moment at mid-span due to self-weight of specimen
- \(M_{\text{spreader beam}}\) = Moment at mid-span due to weight of spreader beam
- \(M_{\text{service}}\) = Moment at service in the transverse direction

### Table 3.4 Phase I Test Results

<table>
<thead>
<tr>
<th>Specimen ID</th>
<th>(P_{cr}) (kips)</th>
<th>(P_u) (kips)</th>
<th>(FS_{cr})</th>
<th>(FS_{\text{ultimate}})</th>
</tr>
</thead>
<tbody>
<tr>
<td>Trial</td>
<td>80</td>
<td>300 (test stopped due to capacity of the frame)</td>
<td>2.27</td>
<td>7.48</td>
</tr>
<tr>
<td>1</td>
<td>90</td>
<td>260 (many cracks in CIP topping in all directions)</td>
<td>2.50</td>
<td>6.53</td>
</tr>
<tr>
<td>2</td>
<td>100</td>
<td>225 (fracture of weld at one location and rebar at another)</td>
<td>2.74</td>
<td>5.7</td>
</tr>
<tr>
<td>3</td>
<td>110</td>
<td>300 (test stopped due to capacity of the frame)</td>
<td>2.98</td>
<td>7.48</td>
</tr>
<tr>
<td>4</td>
<td>60</td>
<td>90 (big crack through precast section)</td>
<td>1.80</td>
<td>2.5</td>
</tr>
</tbody>
</table>

The test results obtained from experimental phase I are discussed in the following three frameworks.

a) Behavior up to service level loads

Figure 3.22 shows the load versus vertical mid-span deflection for all five test specimens up to first crack. The service level load is shown by a dashed line. It can be seen that all specimens performed similarly. Even the specimen which had a tapered web and no mechanical connection between the precast components and the cast-in-place topping had its first crack at a load that was 1.8 times the corresponding service level load.

Figure 3.24 shows the crack pattern of the Trial Specimen and Specimen No.1 at failure as well as a close up of the first crack. In both of these specimens the first crack occurred at the intersection of the precast flange and precast web coupled with a bond failure at the interface between the sides of the precast webs and the cast-in-place topping. The first crack for the Trial and Specimen No.1 occurred at 80 kips and 90 kips, respectively. The intersection of the precast
flange and web represents a radical change in the geometry of the specimen and is prone to stress concentrations. In addition, weak areas may be created due to lack of proper consolidation of concrete during placement.

Specimen No.2 exhibited a similar crack pattern when the first crack occurred. The first crack in Specimen No.2 occurred at 100 kips. The crack at the intersection of the precast flange and web was associated with a bond failure at the vertical interface between the precast and cast-in-place concrete (Figure 3.25). Such a location for the first crack was expected due to the presence of welded connections between the tips of the precast flanges.

In Specimen No.3 first cracking included a crack at the intersection of the precast flange and web, bond failure between the tapered precast web and cast-in-place topping and another crack in the precast section within the constant moment region (Figure 3.25). The first crack in Specimen No.3 occurred at 110 kips.

Finally, in Specimen No.4 the first crack occurred over the joint between the precast components in the cast-in-place topping (Figure 3.25). The cast-in-place section over the joint is the most prone to cracking because of the reduction in the cross-section due to the presence of the joint. The first crack in Specimen No.4 occurred at 60 kips.

b) Behavior up to failure

Figure 3.23 shows the relationship between the load and the vertical deflection at mid-span for all five test specimens up to failure. The specimens with the straight web and the extended bars and the specimen with the tapered web and the embedded plate connection performed relatively better than the other two specimens, because they achieved higher ultimate loads. However, such a relative comparison is not really useful or practical because the ultimate loads for these two specimens were at least six times the service level load. The presence of damaged flanges in the Trial Specimen did not adversely affect its behavior. In fact, the Trial Specimen was one of the two specimens in which the capacity of the loading frame was met before the specimen failed.

The maximum capacity of Specimen No.2 was met when the welded connection between the back of the embedded plate and the transverse bars in the precast flange failed (Figure 3.25). This failure is attributed to any defects in the welded joint and highlights the importance of creating a connection with a high quality full penetration weld. Although, it is important to note that this failure occurred well past the service level load.

The ultimate failure mode for Specimen No.4 was a large crack in the precast component at a load equal to 90 kips, which is 2.5 times the equivalent service level load (Figure 3.25). This occurred because the transverse reinforcing steel in the precast flanges consisted only of No. 3 bars at 18 in. on center, which was less than the bottom transverse steel provided in the cast-in-place topping, which consisted of No.6 at 12 in. on center. Therefore this failure load is expected to improve by increasing the area of transvers steel in the precast flange so that they match that provided in the cast-in-place topping.
c) Simulation of monolithic action

Displacement sensors were installed at the bottom of the specimens near the supports at quarter points and at mid-span to obtain the deflected shape of the specimens and to determine whether they would deflect as two rigid bodies hinged at mid-span, where there is a joint between the flanges of the inverted T-beams, or whether they would deflect as one monolithic body. Figure 3.26 illustrates a typical deflected shape based on the deflections recorded from the displacement sensors. The deflection near the supports is a result of the deformation of the neoprene bearing pads. As can be seen, this deformed shape is closer to the behavior of a monolithic beam than that of two independent rigid bodies. This provides evidence that the inverted T-beam concept can deliver the advantages of jointless, monolithic, cast-in-place concrete construction while saving time in the field by eliminating the need for constructing formwork.

Figure 3.22 Comparison of load deflection curves for all five specimens up approximately first crack
Figure 3.23 Comparison of Load Deflection Curves for all four test specimens

Figure 3.24 Photographs of Trial Specimen and Specimen No.1
Figure 3.25 Photographs of Specimen No.2-4
3.7 Experimental Phase II

3.7.1 Motivation

Because the specimen with the tapered web and no mechanical connection exhibited satisfactory performance under the service level load, a second phase of experimental testing was undertaken to improve upon this detail while maintaining its simplicity. The first goal was to increase the failure load by increasing the area of transverse steel in the precast flanges. The size and spacing of these bars was selected such that they matched the area of the bent bars in the cast-in-place topping. In this manner a continuous tension tie with the same strength would be provided along the entire transverse cross-section of the bridge assuming that the non-contact splices would perform satisfactorily. The second goal was to investigate the performance of the region around the joint when the specimens were subject to a combination of flexure and shear. To accomplish these goals, three additional specimens were tested. Table 3.5 shows the test specimen matrix for Experimental Phase II.

Table 3.5 Test Specimen Matrix for Experimental Phase II

<table>
<thead>
<tr>
<th>Specimen ID</th>
<th>Cross-sectional shape</th>
<th>Connection</th>
<th>Transverse bottom reinforcing in CIP trough</th>
<th>Transverse bottom reinforcing in Precast flange</th>
<th>Loading</th>
</tr>
</thead>
<tbody>
<tr>
<td>5</td>
<td>Tapered web</td>
<td>No connection</td>
<td>No.6 at 12 in</td>
<td>No.6 at 12 in</td>
<td>¼ point</td>
</tr>
<tr>
<td>6</td>
<td>Tapered web</td>
<td>No connection</td>
<td>No.4 at 6 in</td>
<td>No.4 at 6 in</td>
<td>¼ point</td>
</tr>
<tr>
<td>7</td>
<td>Tapered web</td>
<td>No connection</td>
<td>No.4 at 6 in</td>
<td>No.4 at 6 in</td>
<td>offset</td>
</tr>
</tbody>
</table>
3.7.2 Specimen No.5 This specimen is identical to Specimen No.4 with the exception that the bent bars in the cast-in-place concrete and the transverse bars in the precast flanges consisted of No.6 at 12 inches on center. Reinforcing details for this specimen are provided in Figure 3.27. This specimen was loaded at ¼ points. The increase in bar size and reduction in spacing in the precast flanges was intended to provide an increase in the flexural capacity of the precast section in the transverse direction by replacing the No.3 bars at 18 in. on center with No.6 bars at 12 in. on center.

Figure 3.27 Reinforcing details for Test Specimen No.5

3.7.3 Specimen No.6 This specimen is also identical to Specimen No.4 with the exception that the bent bars in the cast-in-place concrete and the transverse bars in the precast flanges consisted of No.4 at 6 inches on center. Reinforcing details for this specimen are provided in Figure 3.28. This change was also intended to provide an increase in the flexural capacity of the precast section in the transverse direction by replacing the No.3 bars at 18 in. on center with No.4 bars at 6 in. on center. The area of steel provided by No.4 at 6 in. on center is close to the area provided by No.6 at 12 in. on center (0.4 in² and 0.44 in², respectively). Both of these areas were big enough to resist the transverse bending moment due to live loads. This specimen was tested to determine the influence of tighter reinforcement spacing.

Figure 3.28 Reinforcing details for Test Specimen No.6 and No.7
3.7.4 Specimen No.7 This specimen is identical to Specimen No.6 with the exception that the loading arrangement is as shown in Figure 3.29. This specimen was tested to observe the performance of the joint between the precast inverted T-beams when subject to a combination of shear and flexural stresses. As it was stated earlier the predominant stresses in the transverse direction in the region around the joint for the U.S.360 Bridge near Richmond, VA, were flexural in nature. However, other situations may exist that create shear stresses around the joint which may not be negligible. Reinforcing details for this specimen are provided in Figure 3.28.

![Figure 3.29 Loading arrangement for Specimen No.7](image)

3.8 Test results from experimental Phase I and II

Table 3.6 provides a summary of the cracking loads and ultimate loads for all tests performed in Experimental Phase I and II. It is important to note that the results from the first seven tests can be compared with each other because the loading arrangement was the same (loading at ¼ points).

<table>
<thead>
<tr>
<th>Specimen ID</th>
<th>P&lt;sub&gt;cr&lt;/sub&gt; (kips)</th>
<th>P&lt;sub&gt;u&lt;/sub&gt; (kips)</th>
<th>FS&lt;sub&gt;cr&lt;/sub&gt;</th>
<th>FS&lt;sub&gt;ultimate&lt;/sub&gt;</th>
</tr>
</thead>
<tbody>
<tr>
<td>Trial</td>
<td>80</td>
<td>300 (test stopped due to capacity of the frame)</td>
<td>2.27</td>
<td>7.48</td>
</tr>
<tr>
<td>1</td>
<td>90</td>
<td>260 (many cracks in CIP topping in all directions)</td>
<td>2.50</td>
<td>6.53</td>
</tr>
<tr>
<td>2</td>
<td>100</td>
<td>225 (fracture of weld at one location and rebar at another)</td>
<td>2.74</td>
<td>5.70</td>
</tr>
<tr>
<td>3</td>
<td>110</td>
<td>300 (test stopped due to capacity of the frame)</td>
<td>2.98</td>
<td>7.48</td>
</tr>
<tr>
<td>4</td>
<td>60</td>
<td>90 (large crack through precast section)</td>
<td>1.80</td>
<td>2.50</td>
</tr>
<tr>
<td>5</td>
<td>70</td>
<td>240 (large crack in CIP topping above the joint)</td>
<td>2.00</td>
<td>6.00</td>
</tr>
<tr>
<td>6</td>
<td>70</td>
<td>140 (large crack in CIP topping above the joint and parallel with the tapered interface on one side)</td>
<td>2.00</td>
<td>3.70</td>
</tr>
<tr>
<td>7</td>
<td>50</td>
<td>81</td>
<td>2.00</td>
<td>3.10</td>
</tr>
</tbody>
</table>
The last specimen (Specimen No.7), was subject to a point load offset from the center, as it is shown in Figure 3.29, and therefore cracking and failure loads cannot be directly compared with the rest of the specimens.

One of the goals of experimental phase II was to improve the ultimate strength of the specimen with no mechanical connection by increasing the size of the bars and decreasing their spacing. A comparison of ultimate loads for Specimens No.4, No.5 and No.6 reveals that this goal was achieved. The same conclusion can be drawn by looking at Figure 3.30, which shows the load deflection curves for Specimens No.4, No.5 and No.6. It can be seen that Specimens No.5 and No.6 performed much better than Specimen No.4.

![Load vs Midspan Deflection to Failure](Image)

**Figure 3.30** Comparison of Load Deflection Curves for specimens with no mechanical connection tested in flexure

The cracking loads for these three specimens are similar and the first crack for all three specimens was over the joint between the precast flanges. This similarity in behavior up to the first crack is illustrated in Figure 3.31. Figure 3.32 shows the crack patterns at first crack and at failure for all three specimens tested in Phase II. The addition of reinforcing steel in the precast component was expected to prevent a failure mode with a large crack in the precast section, which was observed in Specimen No.4, even though this failure occurred at a much higher load than the service load. The failure loads for Specimen No.5 and No.6 were 140 kips and 240 kips respectively compared to a failure load of 90 kips for Specimen No.4. The difference in the ultimate load for Specimens No.5 and No.6 is attributed primarily to the performance of the bond at the interface between the precast and cast-in-place concrete.
Figure 3.31 Comparison of Load Deflection Curves for all three specimens tested in Phase II up to first crack

For Specimen No.5 bond failure started at an actuator load of approximately 120 kips, whereas in Specimen No.6 the bond performed well almost until failure. This stresses the importance of creating a clean roughened surface and free of laitance. During the removal of the grooved wooden formwork sometimes portions of the corrugations created in concrete would break. This phenomenon was more pronounced in Specimen No.5. Based on this discussion, it is recommended that the precaster uses forms that can deliver the roughened surface while being relatively easy to remove. The use of tighter reinforcement spacing did not correlate to significantly better crack control. Therefore the No.6 bars at 12 in. on center are favored as they require less labor for placement. The presence of shear and flexural stresses did not affect the behavior up to the first crack, which occurred at a load twice the service level load. The factor of safety at failure for Specimen No.7 was 3.1 as opposed to 2.5 for Specimen No.4. This indicates an improvement as a result of using larger bars and tighter spacing in the precast despite the presence of the shear and flexural stresses. Specimen No.6, which was subject to quasi pure bending, had a higher factor of safety at failure compared to Specimen No.7. However, a 20% difference in factors of safety at failure that were at least equal to 3.0 does not represent a concern even if the joint was subject to a combination of flexural and shear stresses.
Figure 3.32 Photographs of Specimen No.5-7
3.9 Conclusions

General

Composite bridge superstructure systems that promote rapid construction such as voided slabs and adjacent box girders suffer from reflective cracking problems over the joints between the precast members. Reflective cracking is primarily a serviceability issue because it provides an avenue for water and deicing salts to get to the reinforcing steel more quickly and corrode it. Corrosion leads to expensive repairs and rehabilitations. However, when reflective cracking becomes excessive, then the assumptions about live load distribution factors used in design are called into question. The inverted T-beam concept is a useful and promising concept for short-to-medium-span bridges which delivers the advantages of jointless, cast-in-place, concrete construction while eliminating the need for installing formwork. It addresses the reflective cracking problem by providing a thicker cast-in-place topping over the joint between the precast members.

Designing for Transverse Bending

- It is difficult to propose a simple design methodology for transverse bending that would be suitable for all applications involving precast inverted T-beams. For example the amount of transverse steel in the cast-in-place topping over the longitudinal joint depends on the thickness of the topping over the joint. The greater the thickness the lower the steel demand and vice versa. Accordingly, the most rational approach is to perform an analysis that can capture transverse plate bending behavior and quantify transverse bending moments at service. Transverse bottom reinforcing can then be sized using allowable stress design principles by ignoring any contribution from concrete in tension. This was the approach adopted in sizing transverse reinforcing in Specimens 2-7. The transverse reinforcing sized in this manner will act as transverse load distribution reinforcing and reflective crack control reinforcing for details that are similar to the ones tested in Specimen No.2-7.

- In lieu of performing such an analysis, if the ratio of the thickness of the cast-in-place topping over the joint to the thickness of the precast flange is similar to the one used in the US 360 Bridge described in this paper then the transverse bottom reinforcing (in the cast-in-place topping and in the precast beam) can be sized based on Equation 1. The areas of transverse steel calculated based on the transverse live load moment obtained from finite element analyses and Equation 1 were similar for the US 360 Bridge.

- For transverse connection details that feature transverse bars with 90° hooks through the web of the precast inverted T-beam (Minnesota’s detail) reinforcing in the trough region may be based on recommendations by French et al.⁸.
Cross-sectional Shape and Transverse Connection

Tapering the webs of the precast inverted T-beams provides a higher resistance against normal tensile stresses in the transverse direction. Roughening the surfaces of the precast inverted T-beams that will be in contact with the cast-in-place topping appears to emulate monolithic action and provide the necessary integrity to prevent cracking due to service level loads as a result of plate bending in the transverse direction. All eight specimens, which featured combinations of two cross-sectional shapes and three connections, performed well and similarly at service load levels. The detail with the tapered webs and no mechanical connection is the least expensive one and the easiest to fabricate. This is due to the fact that it does not have any reinforcing steel protruding from the sides of the webs, which presents a forming challenge for the precaster. It also does not have any mechanical connection between the precast members, which might take additional time in the field and work against the concept of accelerating bridge construction.

References

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11. Frosch, R. J., J. K. Bice, and J. B. Erickson. Design Methods for the Control of Restrained Shrinkage Cracking. Publication FHWA/IN/JTRP-2006/32. Joint Transportation Research Program, Indiana Department of Transportation and Purdue University, West Lafayette, Indiana, 2006. doi: 10.5703/1288284313363


**Notation**

$E$ = concrete modulus of elasticity  
$f'_{c}$ = 28-day concrete compressive strength  
$f_{c}$ = concrete compressive strength  
$f_{t}$ = concrete tensile strength  
$FS_{cracking}$ = factor of safety against cracking  
$FS_{ultimate}$ = factor of safety at failure  
$M_{cracking}$ = moment at mid-span due to actuator load at first cracking ($P_{cr}$)  
$M_{ultimate}$ = moment at mid-span due to actuator load at failure ($P_{u}$)  
$M_{selfweight}$ = moment at mid-span due to self-weight of specimen  
$M_{spreaderbeam}$ = moment at mid-span due to weight of spreader beam  
$M_{service}$ = Moment at service in the transverse direction  
$No.$ = number  
$oc$ = on center  
$P_{cr}$ = actuator load at first cracking  
$P_{service}$ = service level actuator load  
$P_{u}$ = actuator load at failure or capacity of loading frame
Chapter 4: Investigation of Time Dependent and Temperature Effects on Composite Bridges with Precast Inverted T-Beams

(Published in 2013 PCI Convention and National Bridge Conference)
Investigation of Time Dependent and Temperature Effects on Composite Bridges with Precast Inverted T-Beams

Abstract

This paper investigates time dependent effects on composite bridges with precast inverted T-beams. The analysis is performed for a two-span continuous bridge. This system provides enhanced performance against reflecting cracking because it offers a thicker cast-in-place topping over the joint between the precast members. An analytical study is performed to quantify the stresses generated as a result of differential shrinkage, creep and temperature gradient at various sections in both directions. At the cross-sectional level, an elastic sectional analysis approach using the age adjusted effective modulus method is used to perform the investigation. At the structure level the effects of uniform temperature changes, thermal gradients and differential shrinkage and creep are investigated and quantified in terms of axial restraint forces and restraint moments. It is shown that by paying attention to detailing and by selecting a mix for the cast-in-place topping that has relatively low shrinkage and high creep the potential for excessive cracking can be reduced and the longevity of the bridge prolonged. Results are presented and recommendations are made for strategies to reduce the magnitude of tensile stresses created as a result of these effects.

4.1 Introduction

On March 19, 2013 the ASCE 2013 Report Card for America’s Infrastructure was published and the United States received a D+\(^1\). The Report Card provides a comprehensive assessment of the current infrastructure condition and needs by assigning grades and by making recommendations for improvements. According to the Report Card one in nine of the nation’s bridges are rated as structurally deficient, while the average age of the nation’s 607,380 bridges is 42 years. Relatively speaking bridges and railroads did better than the rest of the infrastructure by receiving a C+. The Federal Highway Administration (FHWA) estimates that to eliminate the nation’s bridge deficient backlog by 2028, United States would need to invest $20.5 billion annually, while only $12.8 billion is being spent currently.

While many engineers design for the effects of dead, live, seismic and wind loads, not many consider the effects of shrinkage, creep and temperature as significant. Load cases including these effects can in certain bridge types lead to tensile stresses in excess of the tensile strength of concrete. Good examples are composite concrete bridges that consist of precast and cast-in-place elements. To accelerate bridge construction the typical construction sequence for a composite concrete bridge entails the precast elements to be cast long before the cast-in-place topping is placed. This sequence creates a more pronounced difference in the shrinkage and creep properties of the precast and cast-in-place elements because these properties are time dependent.
Short-to-medium-span bridges can be constructed using precast voided slabs or adjacent box girders finished with a cast-in-place topping. This type of construction has manifested longitudinal reflective cracking along the girder-to-girder interface. To address this problem, a new bridge system has been proposed: precast inverted T-beams with a cast-in-place topping. This system was identified during a scanning tour in Europe and Japan funded by the Federal Highway Administration (FHWA)\textsuperscript{2}. The state of Minnesota was the first state in US to implement this system in several bridges\textsuperscript{3,4,5,6,7,8}. Virginia will build its first inverted T-beam bridge in 2014 on U.S.360 over Chickahominy River near Richmond, VA. The research team at Virginia Tech built upon the findings and experiences of the state of Minnesota and made several recommendations optimizing the cross-sectional shape and the inverted T-beam to inverted T-beam connection\textsuperscript{9}. This paper provides the results of a time dependent analysis for the two span continuous bridge that will be built in Virginia by considering the effects of differential shrinkage, creep and temperature. The objective was to investigate the effect of time dependent properties of the concrete topping, cross-sectional shape of the precast inverted T-beams, amount of mild steel in the deck, boundary conditions at the abutments and age of continuity on the magnitude of tensile stresses in the composite cross-section in the longitudinal and transverse directions. Controlling the magnitude of these tensile stresses is important to avoid excessive transverse and longitudinal cracks.

4.2 Implementation of the Inverted T-Beam System in Virginia

Implementation of the inverted T-beam system will occur in the U.S.360 Bridge over Chickahominy River. Figure 4.1 shows an elevation view of the two-span river crossing. Figure 4.2 shows a transverse cross-section of the bridge. The bridge has two equal spans of 43 ft and has an approximate width of 110 ft.

![Figure 4.1 Elevation view of the U.S.360 Bridge (Courtesy of VDOT)]
Figure 4.2 Transverse cross-section of the bridge

The depth of the precast inverted T-beams is 18 in. and the depth of the cast-in-place concrete topping over the web of the precast beams is 7 in. Design concrete strengths ($f'_c$) for the precast inverted T-beams and cast-in-place topping are $f'_c = 8000$ psi and $f'_c = 4000$ psi, respectively. Figure 4.3 shows reinforcing details for a typical 6'-0" section of the bridge. The cross-sectional dimensions shown in Figure 4.2 and reinforcing details shown in Figure 4.3 represent one of the options considered by VDOT during the design phase and not necessarily the final design.

4.3 Time Dependent and Temperature Analysis

Bridges constructed with prefabricated elements offer many advantages over conventional construction methods. In many cases the precast components serve as stay-in-place formwork for the cast-in-place topping or deck. This eliminates the need to erect and remove formwork and results in shorter construction time and reduction in traffic disruption. However many existing bridges with precast components have durability issues, such as excessive cracking, which results in significant maintenance and replacement costs. This can eclipse the advantages that would otherwise be associated with these types of systems. Time dependent
effects, such as differential shrinkage between the cast-in-place and precast components, are a major reason for the development of this cracking.

In conventional cast-in-place, shored construction, the self-weight of concrete will cause compressive stresses in the top surface of the deck in positive moment regions, and tensile stresses in negative moment regions. Additionally, tensile stresses created due to time dependent effects are limited to differential shrinkage between cast-in-place concrete and reinforcing steel and those created due to temperature gradients. Consequently potential cracking is limited to negative moment regions because elsewhere compressive stresses due to the self-weight of concrete counterbalance any tensile stresses created due to time dependent effects. The situation is different in systems that involve precast elements. Because the precast components provide support to the cast-in-place topping, the weight of the topping causes stress in the precast beams. As a result, the effects of differential shrinkage are more pronounced and can cause critical stress situations in the topping.

The following presents a time dependent analysis at the cross-sectional level as well as at the structural level to quantify stresses developed as a result of differential shrinkage, shrinkage induced creep, negative/positive temperature gradients and a uniform decrease in the temperature.

To promote a comfortable ride and to reduce the likelihood of leakage to the substructure many engineers design precast girder bridges as continuous for live loads. Continuity is provided by placing a cast-in-place concrete topping over the precast elements, which creates a continuity diaphragm at the interior supports. Additionally, reinforcing steel is provided to connect the bottom of the precast girders over interior supports. The age of the precast beams when this continuity is established plays an important role in the development of time dependent effects. The analysis performed in this paper assumes a precast girder age of 90 days or more, before continuity is established. At this age most of the shrinkage and creep in the precast girder has occurred.

The advantage of specifying a high age for continuity is the reduction of positive restraint moments at the intermediate supports. These positive restraint moments may develop due to creep of the precast beam, as well as due to positive thermal gradients. These positive restraint moments can be high enough to overcome the effects of negative live load moments\(^3\). In addition, these positive restraint moments can also be high enough to render the positive moment connection over the piers as not providing 100% continuity.

One of the disadvantages of waiting for 90 days is that the differences in shrinkage and creep properties between the precast and cast-in-place components become more pronounced. Because the age of continuity for the bridge under consideration was assumed to be 90 days the ultimate shrinkage strain and creep coefficient for the precast girder were neglected. The corresponding values for the cast-in-place topping were taken as follows:

\[ \epsilon_{sh\,deck} = -466 \times 10^{-6}, \quad \phi_{deck} = 1.87 \]
These values were based on testing of four different concrete mixes with a design compressive strength at 28 days of $f'_{c} = 4000$ psi. The goal was to identify a mix with low shrinkage and high creep. The aggregates used in the four mixes consisted of normal weight and light weight aggregates. Additionally, the cementitious materials consisted of fly ash and slag. The ultimate shrinkage and creep coefficient values provided above represent the concrete mix with the lowest shrinkage and highest creep.

4.3.1 Time Dependent and Temperature Analysis at the Cross-Sectional Level

4.3.1.1 Differential Shrinkage and Shrinkage Induced Creep

Regardless of the boundary conditions, the inherent difference in shrinkage and creep properties between the cast-in-place topping and precast girders will cause self-equilibrating stresses at the cross-sectional level. Even if the composite beam is used in a single span simply supported bridge, these self-equilibrating stresses will form along the entire span of the bridge. The difference in shrinkage properties is exacerbated by the difference in age between the two components. As a result, when the topping is placed, it will tend to shrink while the majority of the shrinkage in the precast component has already taken place. The restraint provided by the precast component to the free shrinkage of the deck will create a tensile force in the deck while the free shrinkage of the deck will exert a compressive force in the precast beam.

In addition, because the centroids of the precast and cast-in-place components are at different locations, this differential shrinkage will cause a positive curvature. The curvature will result in a prestress gain in the bottom layer of prestressing in the precast beam, whereas the compression force from the shrinkage of the deck will cause a prestress loss. Another advantage of the precast inverted T-beam system is that the difference between the centroids of the cast-in-place and precast components is smaller compared to a similar voided slab or adjacent box girder system. Consequently the curvature induced due to differential shrinkage is smaller.

Mild steel in the deck will provide an additional level of restraint against the free shrinkage of the deck and will therefore increase the tensile stresses in the concrete topping. Figure 4.4 shows the idealized locations of mild steel and prestressing steel used in the time-dependent analysis. The amount of mild steel and prestressing steel was based on the design of the U.S.360 Bridge per 2010 AASHTO LRFD Bridge Design Specifications. The quantification of forces and stresses created due to differential shrinkage and shrinkage induced creep can be done using the principles of equilibrium, compatibility and material constitutive relationships. Menn provides detailed guidance on how this analysis can be performed. Some of the theoretical background provided in Menn’s book is presented below for convenience. Figure 4.5 shows composite cross-section 2 and the change in strain and forces due to differential shrinkage and shrinkage induced creep. The internal forces created because of the shrinkage of the cast-in-place topping will cause the cast-in-place and precast components to creep over time. This shrinkage induced creep is captured by using the age-adjusted effective modulus method. The aging coefficient is assumed to be 0.7. For example, the change in strain...
at the centroid of deck and girder can be determined by computing elastic and creep strains due to the change in axial force plus the strain due to free shrinkage (Equations 1 and 4). Similarly, the change in curvature can be determined by calculating elastic and creep curvatures due to the change in moment (Equations 2 and 5). The change in strain in any given steel layer can simply be determined by computing the elastic strain due to the change in axial force in the corresponding layer. In addition, because there are no externally applied axial forces or moments the sum of the change in axial forces and moments needs to be equal to zero (Equations 6 and 7). Assuming that there is a perfect bond between the cast-in-place deck, precast inverted T and mild steel, the axial strains at the centroid of each component can related by utilizing the curvature and the relative distances (principle of compatibility). Equation 8 provides one such example. By using Equations 1-8 a set of 15 equations and unknowns can be created and solved simultaneously. The unknowns would include changes in strain and forces in each component and the change in curvature. After solving for the unknowns, the change in stress at any given location in the precast inverted T-beam, deck or at any layer of mild steel can be calculated using Equations 9-11. The assumptions made during this analysis are:

- Plane sections remain plane
- Sections are un-cracked
- Creep and shrinkage properties are assumed to represent the average behavior of the whole cross-sections, or components thereof, in drying conditions.
- Tensile creep is the same as compressive creep

Figure 4.6 (a) shows the stress distributions in Sections 1, 2 and 3 caused by differential shrinkage and shrinkage induced creep. In this paper tensile stresses are positive and compressive stresses are negative. The stress distribution shown for Section 1 applies at the portion of this section where the thickness of the precast inverted T-beam is 18 in. and the thickness of the cast-in-place topping is 7 in. The maximum tensile stresses at the bottom of the cast-in-place topping in Sections 1, 2 and 3 are 0.37 ksi, 0.496 ksi and 0.487 ksi respectively. The modulus of rupture ($f_r$) for the deck is 0.474 ksi (based on $7.5\sqrt{f'c}$ where $f'c= 4000$ psi). This highlights the potential of differential shrinkage to cause longitudinal cracking in the deck. The maximum tensile stresses at the bottom of the deck and at the bottom of precast inverted T-beam in Section 1 are lower than the ones in Section 2. As mentioned earlier this is due to the fact that the moment arm between the centroids of the cast-in-place topping and the precast beam in Section 1 is lower than in Section 2. This promotes the utilization of the inverted T-beam system as opposed to a voided slab system or adjacent box girder system, considering that Section 2 represents a similar section in the transverse direction in both of these systems, in addition to the longitudinal direction. The compressive stress at the top of the precast inverted T-beam is higher in Section 1 than in Section 2 due the higher volume of concrete. However, given that the weakness of the concrete is its tensile strength, this will not control design.
Section 1 - Transverse Section

Section 2 - Longitudinal Section through Precast Web

Section 3 - Longitudinal Section through Precast Flange

**Figure 4.4** Idealized locations of mild steel and prestressing steel used in time dependent analysis
Forces in composite section 2 due to differential shrinkage and creep

\[
\Delta \varepsilon_D = \frac{\Delta N_D}{E_D A_D} (1 + \mu \varphi_D) + \varepsilon_{SHD}
\]

(1)

\[
\Delta X = \frac{\Delta M_D}{E_D I_D} (1 + \mu \varphi_D)
\]

(2)

\[
\Delta \varepsilon_S = \frac{\Delta N_S}{E_S A_S}
\]

(3)

\[
\Delta \varepsilon_G = \frac{\Delta N_G}{E_G A_G} (1 + \mu \varphi_G) + \varepsilon_{SHG}
\]

(4)

\[
\Delta X = \frac{\Delta M_G}{E_G I_G} (1 + \mu \varphi_G)
\]

(5)

\[
\Delta N_D + \Delta N_G + \Delta N_{s1} + \Delta N_{s2} + \Delta N_{s3} + \Delta N_{s4} = 0
\]

(6)

\[
\Delta M_G + \Delta M_D - \Delta N_Da - \Delta N_{s1}a_{s1} - \Delta N_{s2}a_{s2} + \Delta N_{s3}a_{s3} + \Delta N_{s4}a_{s4} = 0
\]

(7)

\[
\Delta \varepsilon_D = \Delta \varepsilon_G - \Delta X (y_{Dbottom} - y_{Gbottom})
\]

(8)

\[
\Delta \sigma_D = \left( \frac{\Delta N_D}{A_D} + \frac{\Delta M_D}{I_D} \right) y
\]

(9)

\[
\Delta \sigma_G = \left( \frac{\Delta N_G}{A_G} + \frac{\Delta M_G}{I_G} \right) y
\]

(10)

\[
\Delta \sigma_S = \frac{\Delta N_S}{A_S}
\]

(11)

Figure 4.5 Forces in composite section 2 due to differential shrinkage and creep

Figure 4.6 (b), (c) and (d) show the sensitivity of the tensile stress at the bottom of the deck to shrinkage and creep properties of the deck for Sections 1, 2 and 3, respectively. The horizontal and the vertical lines represent the modulus of rupture and the ultimate shrinkage strain for the deck, respectively. For example in Section 1 for a creep coefficient \( \varphi = 2 \), there is a 78 psi decrease in the tensile stress for every 100 \( \mu \varepsilon \) decrease in the ultimate shrinkage strain of the topping mix \( (\varepsilon_{shdeck}) \). Similarly, for an ultimate shrinkage strain of \( \varepsilon_{shdeck} = -500 \times 10^{-6} \) there is
a 119 psi decrease in the tensile stress for every increase by 0.5 in the creep coefficient. Clearly a mix with lower free shrinkage and high creep will be ideal from the standpoint of reducing tensile stresses as a result of differential shrinkage. High creep properties are desired to relieve the stresses developed as a result of differential shrinkage. Low shrinkage in the deck is desired to minimize the amount of differential shrinkage, provided that most of the shrinkage in the precast beam has already taken place.

Figure 4.6 (a) Stress distribution due to differential shrinkage and shrinkage induced creep in all three cross-sections; (b), (c) and (d) Sensitivity of tensile stress at the bottom of the deck to shrinkage and creep properties of the deck for Sections 1, 2 and 3, respectively

The presence of mild steel in the deck restrains the free shrinkage of the deck and as a result creates additional tensile stresses. Figure 7 shows the sensitivity of the tensile stress at the bottom of the deck to the amount of mild steel. In Figure 7(a) $A_{smild1}$, $A_{smild2}$ and $A_{smild3}$ represent the variation in areas of mild steel in the deck in the longitudinal direction at different elevations. These are denoted as $A_s1$, $A_s2$ and $A_s3$ in Figure 4 - Section 1, respectively. In Figure 4.7 (b) and 6(c) $A_{smild1}$ and $A_{smild2}$ represent the variation in areas of mild steel in the deck in the transverse direction at different elevations. These are denoted as $A_{s1}$ and $A_{s2}$ in Figure 4.4, Section 2 and
Section 3, respectively. The vertical lines in Figure 4.6 represent the actual amounts of mild steel in the deck, which were based on the design of the U.S.360 Bridge per 2010 AASHTO LRFD Bridge Design Specifications\textsuperscript{11}. It can be seen that while the magnitude of the tensile stress at the bottom of the deck increases with an increase in the amount of mild steel, this increase is almost negligible. As a result, mild steel needs to be provided in the deck in both directions to control the width of potential cracks and it does not significantly increase the likelihood of cracking.

![Figure 4.7](image)

**Figure 4.7** Sensitivity of tensile stress at the bottom of the deck to the amount of mild steel in Sections 1, 2 and 3 respectively

4.3.1.2 Temperature Gradient

Temperature gradients create similar effects to the ones created by differential shrinkage. Because temperature can vary through the depth of the cross-section, some parts of the cross-section will tend to contract or expand more than the other parts. The temperature gradient used in this study was obtained from the 2010 AASHTO LRFD Bridge Design Specifications\textsuperscript{11} for the U.S.360 Bridge near Richmond, VA. The positive and negative temperature gradients have a bi-linear shape and are shown in Figure 4.8. Assuming plane sections remain plane, this bi-linear
variation in temperature will cause self-equilibrating stresses in the cross-section. These stresses can be calculated using the principles of equilibrium, compatibility and material constitutive relationships\textsuperscript{13}. A sensitivity analysis for the creep and aging coefficients was not done because it was assumed that the temperature gradient would develop over a period of 8 hours. As a result the changes in creep and aging coefficients over such a short period of time would be negligible.

![Diagram of temperature gradients](image)

**Figure 4.8** Positive and negative temperature gradients for the U.S.360 Bridge, near Richmond, VA

Some of the theoretical background presented in Gilbert\textsuperscript{13} for the calculation of self-equilibrating stresses due to thermal gradients is presented below for convenience. Figure 9 illustrates this approach by taking Section 2 as an example and the negative temperature gradient shown in Figure 4.8. If all the fibers in the composite cross-section were free to contract independently to accommodate the imposed negative temperature gradient, then the result would be the free strain diagram shown in Figure 4.9. The corresponding stress distribution can be calculated by simply multiplying the free strains with the moduli of elasticity of each component. However, because plane sections will tend to remain plane, the individual fibers will not be able to freely contract to accommodate the temperature gradient without violating the principle of compatibility. As a result, there will be some restrained stresses in the composite cross-section which are equal and opposite to the free stresses. The stress resultants of these restrained stresses (axial force and bending moment) can be calculated using Equations 12 and 13. Finally, the change in stress due to the imposed temperature gradient can be computed using Equations 14-16.
Figure 4.9 Approach for calculating self-equilibrating stresses due to thermal gradients (Section 2)

\[ \Delta N = \int \alpha T(y) E_b \, dy \quad (12) \]
\[ \Delta M = \int \alpha T(y) E_b y \, dy \quad (13) \]
\[ \Delta \sigma_D = -\Delta \sigma_{D\text{free}} + \left( \frac{\Delta N}{A_{tr}} + \frac{\Delta M y}{I_{\text{composite}}} \right) n_D \quad (14) \]
\[ \Delta \sigma_G = -\Delta \sigma_{G\text{free}} + \left( \frac{\Delta N}{A_{tr}} + \frac{\Delta M y}{I_{\text{composite}}} \right) \quad (15) \]
\[
\Delta \sigma_s = -\Delta \sigma_{s\text{free}} + \left( \frac{\Delta N}{A_{tr}} + \frac{\Delta M_y}{M_{\text{composite}}} \right) n_s
\]  

Figure 4.10 shows stress distributions in Sections 1, 2 and 3 due to negative and positive temperature gradients. The largest negative temperature gradient tensile stress is at the top of the deck and is slightly higher than the largest tensile stress created as a result of a positive temperature gradient (0.15 ksi versus 0.11 ksi). These stresses are lower than the modulus of rupture (0.474 ksi) for the concrete topping. Therefore, temperature gradients alone cannot create high enough tensile stresses to cause cracking. However, when the effects of differential shrinkage and temperature gradients are combined then these stresses exceed the rupture stress. Table 4.1 provides a summary of the stresses created at the top and bottom of the deck respectively. As a result the top of the deck is likely to experience longitudinal cracking above the web of the precast girder due to the combined effects of differential shrinkage and temperature gradient. The bottom of the deck will be subject to transverse and longitudinal cracking. It is important to note that the analysis performed at the cross-sectional level shows stress distributions that apply along the entire bridge superstructure. Therefore, if the tensile stresses in the deck are higher than its modulus of rupture, cracks could potentially develop along the entire bridge length and width.

**Figure 4.10** (a) Stress distribution - negative temperature gradient, (b) Stress distribution - positive temperature gradient
Table 4.1 Tensile stresses at the top and bottom of the deck due to differential shrinkage and temperature gradient

<table>
<thead>
<tr>
<th>Section</th>
<th>Differential Shrinkage (ksi)</th>
<th>Temperature Gradient (ksi)</th>
<th>Total (ksi)</th>
<th>$f_r$ (ksi)</th>
<th>Total/$f_r$</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>0.052</td>
<td>0.15</td>
<td>0.202</td>
<td>0.474</td>
<td>0.43</td>
</tr>
<tr>
<td>2</td>
<td>0.352</td>
<td>0.154</td>
<td>0.506</td>
<td>0.474</td>
<td>1.07</td>
</tr>
<tr>
<td>3</td>
<td>-0.264</td>
<td>0.15</td>
<td>-0.114</td>
<td>0.474</td>
<td>N/A</td>
</tr>
</tbody>
</table>

Tensile stresses at the bottom of the deck

<table>
<thead>
<tr>
<th>Section</th>
<th>Differential Shrinkage (ksi)</th>
<th>Temperature Gradient (ksi)</th>
<th>Total (ksi)</th>
<th>$f_r$ (ksi)</th>
<th>Total/$f_r$</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>0.373</td>
<td>0.1</td>
<td>0.473</td>
<td>0.474</td>
<td>1.0</td>
</tr>
<tr>
<td>2</td>
<td>0.496</td>
<td>0.09</td>
<td>0.586</td>
<td>0.474</td>
<td>1.24</td>
</tr>
<tr>
<td>3</td>
<td>0.487</td>
<td>0.02</td>
<td>0.507</td>
<td>0.474</td>
<td>1.07</td>
</tr>
</tbody>
</table>

4.3.2 Time Dependent and Temperature Analysis at the Structural Level

The indeterminacy of the superstructure plays an important role when it comes to evaluating the effects of differential shrinkage, creep and temperature at the structure level. For example, axial contraction as a result of a uniform decrease in temperature in the longitudinal direction of the two-span continuous bridge can cause significant tensile stresses in the topping and in the precast beam if not accommodated. In addition, the curvatures created as a result of differential shrinkage and temperature gradients cause axial contractions and expansions, which need to be allowed to take place to reduce the likelihood of developing additional stresses that might lead to excessive cracking. The following discussion illustrates some of the effects that these phenomena can have if the bearing details at the abutments do not allow axial movements.

Another type of restraint at the structure level in multi-span bridges is the moment restraint at the intermediate supports. The restraint moments develop because the curvatures created by the differential shrinkage and temperature gradients are not allowed to freely take place due to the continuity of the bridge at the interior supports. The assumptions made at the structural level to perform a time dependent analysis are as follows:

- The axial restraint provided by the abutments in the longitudinal direction was assumed to be rigid.
- Plane sections remain plane
- Sections are un-cracked
- Creep and shrinkage properties are assumed to represent the average behavior of the whole cross-sections, or components thereof, in drying conditions.
- Tensile creep is the same as compressive creep
4.3.2.1 Axial Restraint at the Abutments (Differential Shrinkage, Temperature Gradient, Uniform Temperature)

In a statically determinate structure the bridge superstructure will be free to contract and expand axially and therefore there will be only axial strains and no stresses. However, if this axial movement is restrained, the restraining axial force will create significant stresses in the superstructure. The calculation of these stresses can be performed by imposing the principle of compatibility that requires the total axial deformation to be zero at all bearings where this deformation is restrained. The restraining force can be calculated using the force method of structural analysis in which the axial deformation due to differential shrinkage, temperature gradient and uniform temperature changes must be equal to the axial deformation caused by the restraining force. The uniform temperature change used in this investigation was based on 2010 AASHTO LRFD Bridge Design Specifications\textsuperscript{11} and was calculated to be 70°F. Figure 4.11 (a) shows the stress distribution in the composite cross-section due to axial restraint at the abutments. The axial stresses due to axial restraint against negative temperature gradient in the topping and the precast beam are small (0.042 ksi and 0.06 ksi, respectively). However, the axial stresses due to axial restraint against differential shrinkage and a uniform decrease in temperature are at least 35% greater than the modulus of rupture (0.641 ksi and 0.906 ksi due to differential shrinkage and 0.965 ksi and 1.501 ksi due to a uniform decrease in temperature). The sensitivity of the stress in deck to the shrinkage and creep properties of the deck is illustrated in Figure 4.11(b). The horizontal and vertical lines represent the modulus of rupture and ultimate shrinkage strain for the deck, respectively. If the creep coefficient is assumed to be \( \phi = 2.0 \) then there is a 136 psi decrease in the tensile stress in the deck for every 100 \( \mu \varepsilon \) decrease in shrinkage strain.

\[ \text{(a) Stress distribution in Section 1 due to potential axial restraint at the abutments in the longitudinal direction; (b) Sensitivity of stress in deck to shrinkage and creep properties of the deck} \]

\[ \text{Figure } 4.11 \]
Similarly, if the free ultimate shrinkage strain of the deck is assumed to be \( \epsilon = -500 \times 10^{-6} \), then there is an 81 psi decrease in the tensile stress in the deck for every increase by 0.5 in the creep coefficient.

As a result, to reduce the likelihood of excessive transverse cracking, axial movement in the longitudinal direction should be accommodated at the abutments. If this movement is restrained, then a topping mix with low shrinkage and high creep will help reduce the tensile stresses. Tensile stresses developed as a result of axial restraints at the abutments in the longitudinal direction due to differential shrinkage, negative temperature gradient and a uniform decrease in temperature apply not only to the entire bridge superstructure but are also constant throughout the depth of the cross-sections. These high tensile stresses have the potential to develop full depth transverse cracks. In addition to the obvious serviceability and durability problems that these high tensile stresses can create, full depth cracks in regions of small moment can cause reductions in shear strength.

Figure 4.12(a) shows how the stress in the deck due to a uniform decrease in temperature is affected by creep and aging coefficients. The calculation of the restraining axial force at the abutments due to a uniform change in temperature was based on Equation (17). This equation was derived based on the principle of deformation compatibility and the fact that the deck concrete will creep and age whereas the precast girder has already aged and crept when continuity is established. For a fixed aging coefficient of 0.7, the higher the creep coefficient the lower the tensile stress. The tensile stress values in the deck vary from 1.5 ksi when the creep coefficient is zero to 0.97 ksi when the creep coefficient is 2.0. The corresponding values for the precast beam are 2.32 ksi and 1.5 ksi, respectively. Similarly, for a fixed creep coefficient in the deck equal to 2.0, the higher the aging coefficient the lower the tensile stress. This is illustrated in Figure 4.12 (b). The tensile stress values in the deck vary from 0.965 ksi when the aging coefficient is 0.7 to 0.89 ksi when the aging coefficient is 1.0. The corresponding values for the precast beam are 1.5 ksi and 1.384 ksi respectively. This highlights the advantage of a concrete mix that has high creep and does not age significantly. The influence of a higher creep coefficient is more pronounced in reducing the tensile stresses in the deck and the precast beam compared to the aging coefficient. Consequently priority should be given to a mix that has high creep.

\[
N_{\text{restraint}} = \frac{\alpha \Delta T_{\text{uniform}} E_G A_{\text{transformed}}}{1 + \frac{A_D \mu_D \varphi_D}{A_D + A_G (1 + \mu_D \varphi_D)}}
\]

where:

\( \alpha = \) coefficient of thermal expansion
\( \Delta T_{\text{uniform}} = \) uniform change in temperature
\( E_G = \) modulus of elasticity of the precast inverted T
\( A_{\text{transformed}} = \) transformed area of the composite section
\( A_D, A_G = \) area of the deck, area of precast inverted T
\( \mu_D = \) aging coefficient for deck concrete
\( \varphi_D = \) creep coefficient for deck concrete
It should be noted that in the analysis performed in this study the axial restraining stiffness of the abutments was taken equal to infinity. Additionally, the creep coefficient for the precast beams was taken equal to zero because, as stated earlier, it is believed that if the age of continuity is at least 90 days most of the creep in the precast beams has already taken place. In reality the axial restraining stiffness of the abutments will be smaller than infinity and the creep coefficient for the precast beam will be higher than zero. As a result, the stresses created in the deck and the precast beam may be slightly lower than the values presented in this paper.

4.3.2.2 Moment Restraint at the Intermediate Support

Restraint moments at the intermediate supports are another source for developing tensile stresses in the deck that can lead to excessive transverse cracking. These moments are developed as a result of the restraint to the curvatures induced by creep of concrete under sustained loads and prestressing, differential shrinkage and temperature gradients. The calculation of restraint moment (M_r) due to prestressing, sustained loads and differential shrinkage is based on Equation (18) (Peterman and Ramirez14):

\[
M_r = \left( \frac{3}{2} \alpha M_p - \alpha M_{dprecast} \right) [\Delta (1 - e^{-\varphi_1})] - \alpha M_{dcip} (1 - e^{-\varphi_2}) - \frac{3}{2} \alpha M_s \left( \frac{1-e^{-\varphi_2}}{\varphi_2} \right)
\]

(18)

where:

\( M_p \) = moment caused by prestressing force about centroid of composite member
\( M_s \) = differential shrinkage moment
\[ M_{\text{precast}} = \text{mid-span moment due to dead load of precast members} \]
\[ M_{\text{CIP}} = \text{mid-span moment due to dead load of CIP topping} \]
\[ \phi_1 = \text{creep coefficient for creep effects initiating when prestress force is transferred to the precast panels} \]
\[ \phi_2 = \text{creep coefficient for creep effects initiating when CIP topping is cast} \]
\[ \alpha = \text{factor that accounts for the relative flexural stiffnesses of the spans and diaphragm} \]
\[ \Delta (1-e^{-\phi_1}) = \text{change in expression (1-e^{-\phi_1}) occurring from time CIP topping is cast to time corresponding to restraint moment calculation.} \]

The first term represents the restraint moment due to creep of the precast member due to prestressing force and the weight of the precast member. The second term represents the restraint moment due to creep of the precast member due to the cast-in-place topping weight. The third term represents the restraint moment due to differential shrinkage. Peterman and Ramirez\textsuperscript{14} provide additional information for the calculation of some of the terms defined above including an equation for the calculation of differential shrinkage moment. However, this equation does not account for the restraint provided by steel in the precast member and shrinkage induced creep in precast and cast-in-place components. As a result the calculation of differential shrinkage moment was based on Menn’s method\textsuperscript{10}, which considers all the aforementioned effects. Table 4.2 provides a summary of differential shrinkage moments calculated using various methods. The method proposed by Peterman and Ramirez is simple to use and estimates a differential shrinkage moment which is only 11\% different from the one calculated using Menn’s Method. PCA method provides a much conservative estimate of the differential shrinkage moment. This is a result of the fact that PCA Method does not correctly account for the restraining effect that the precast girder and reinforcing steel has on the free shrinkage of the deck. In this paper only one time step was used to calculate the differential shrinkage moment using the CTL method. Additional information on the calculation of differential shrinkage moment based on the methods mentioned above is provided in Reference 10 and 14-17. The calculation of restraint moments due temperature gradients was based on Gilbert\textsuperscript{13}.

<table>
<thead>
<tr>
<th>Method</th>
<th>Differential shrinkage moment (ft-kips)</th>
<th>% difference with Menn’s Method</th>
</tr>
</thead>
<tbody>
<tr>
<td>Menn\textsuperscript{10}</td>
<td>646</td>
<td></td>
</tr>
<tr>
<td>PCA\textsuperscript{15,16}</td>
<td>1393</td>
<td>216%</td>
</tr>
<tr>
<td>CTL\textsuperscript{17*}</td>
<td>724</td>
<td>12%</td>
</tr>
<tr>
<td>Peterman and Ramirez\textsuperscript{14}</td>
<td>714</td>
<td>11%</td>
</tr>
</tbody>
</table>

Note: * Using only one time step

Tensile stresses developed as a result of moment restraint due to differential shrinkage and negative/positive temperature gradients are maximum at the intermediate support and reduce linearly towards the abutments (for a two-span continuous bridge). Because a positive temperature gradient causes a positive restraint moment at the intermediate support its effects
were not investigated because the focus of this paper was potential cracking on the top surface of the deck. As stated earlier the analysis performed in this study assumes and age of continuity equal to at least 90 days, which represents a best case scenario for reducing positive restraint moments, and a worst case scenario for developing negative restraint moments.

Figure 4.13(a) shows the stresses in the composite cross-section due to negative restraint moments caused by differential shrinkage and negative temperature gradients. The corresponding maximum tensile stresses in the deck are 1.291 ksi and 0.145 ksi, respectively. The stresses at the top of the precast inverted T-beam due to negative temperature gradient and differential shrinkage are 0.098 ksi and 0.87 ksi, respectively. Table 4.3 provides a summary of these values as well as the ratio of the total tensile stress due to negative temperature gradient and differential shrinkage to the modulus of rupture. The tensile stresses created as a result of negative temperature gradient are smaller than the modulus of rupture for the deck (0.474 ksi), whereas those created from differential shrinkage are more than 2.7 times. It can be seen that the sum of negative restraint moments creates tensile stresses in the deck and precast inverted T-beam that are well past the modulus of rupture.

Table 4.3 Stresses due to negative restraint moments

<table>
<thead>
<tr>
<th></th>
<th>Negative Temperature Gradient</th>
<th>Differential Shrinkage</th>
<th>Total</th>
<th>fr</th>
<th>Total/fr</th>
</tr>
</thead>
<tbody>
<tr>
<td>Stress at the top of the deck (ksi)</td>
<td>0.145</td>
<td>1.291</td>
<td>1.436</td>
<td>0.474</td>
<td>3.03</td>
</tr>
<tr>
<td>Stress at the top of precast (ksi)</td>
<td>0.098</td>
<td>0.87</td>
<td>0.968</td>
<td>0.671</td>
<td>1.44</td>
</tr>
</tbody>
</table>

Figure 4.13 (b) shows the sensitivity of the maximum tensile stress in deck to the shrinkage and creep properties of the deck. The horizontal and vertical lines represent the modulus of rupture and the ultimate shrinkage strain for the deck, respectively. It can be seen that while the maximum tensile stress in the deck is sensitive to the ultimate shrinkage strain, it is not that sensitive to the creep coefficient of the deck. If the creep coefficient is assumed to be 2.0 than there will be a 275 psi decrease in the tensile stress for every 100 με reduction in the free ultimate shrinkage strain of the deck.
Figure 4.13 (a) Stress distribution in Section 1 due to differential shrinkage/creep and negative temperature gradient; (b) Sensitivity of tensile stress at the top of the deck to shrinkage and creep properties of the deck

The negative moments due to superimposed dead and live loads at service for the U.S.360 Bridge are 107 kip-ft and 219 kip-ft, respectively. The restraint moment due to differential shrinkage and shrinkage induced creep is 909 kip-ft, which is nearly 2.8 times greater than the sum of the negative moments due to dead and live loads. The negative restraint moment due to negative temperature gradient is 102 kip-ft, which is slightly lower than the negative moment due to superimposed dead loads. Table 4.4 summarizes the magnitudes of negative moments at the interior support. This highlights the significance of negative restraint moments developed as a result of time dependent effects in terms of magnitude. Menn\textsuperscript{10} states in his book “Prestressed Concrete Bridges” that: “Theoretically no sectional forces are present at the ultimate limit state due to restrained deformations in ductile systems. In general, restrained deformations are significant only for the behavior of structures under service load conditions with regards to cracking and deflections”. This is due to the fact that a ductile system can accommodate imposed curvatures and axial strains by the formation of plastic hinges and yielding of the reinforcing steel. Consequently, while these high restraint moments do not present a safety concern they do need to be controlled to reduce the likelihood of excessive cracking. In this regard, specifying an optimized age of continuity in which the competing effects of negative and positive restraint moments would cancel each other as much as possible is essential. High positive restraint moments negate the effects of negative live load moments and may render a continuous design even more expensive than a design based on simply supported beams. High negative moments may create excessive cracking on the bridge decks and reduce the service life of bridges.
Table 4.4 Negative moments at interior support

<table>
<thead>
<tr>
<th>Load Cases</th>
<th>Negative Moments (ft-kips)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Superimposed Dead Load</td>
<td>107 ($M_{negsuperDL}$)</td>
</tr>
<tr>
<td>Superimposed Live Load</td>
<td>219 ($M_{negsuperLL}$)</td>
</tr>
<tr>
<td>Shrinkage + Creep</td>
<td>909 ($M_{negSH+CR}$)</td>
</tr>
<tr>
<td>Negative Temperature Gradient</td>
<td>102 ($M_{negTG}$)</td>
</tr>
</tbody>
</table>

Ratios

$$\frac{(M_{negSH+CR})}{(M_{negsuperDL} + M_{negsuperLL})} = 2.8$$

$$\frac{(M_{negTG})}{(M_{negsuperDL} + M_{negsuperLL})} = 0.3$$

### 4.4 Conclusions and Recommendations

This study has demonstrated that time dependent effects can cause significant stresses in composite concrete bridge superstructures with precast inverted T-beams. To reduce the likelihood of excessive cracking recommendations are made in the following five frameworks. While these recommendations were based on the analysis of a two-span continuous bridge with precast inverted T-beams, most of them apply to most types of composite bridges.

- **Mix Design** – It is recommended that the concrete mix for the CIP topping possesses low shrinkage and high creep, as these properties help relax tensile stresses built-up due to time dependent effects. While it may be onerous to the supplier to conduct creep tests for various mix designs, it is relatively simple to collect short term shrinkage data (up to 28 days) on a variety of mixes. This will help create a database of shrinkage values for various mixes that could be used in future projects if the specifications require a mix with certain shrinkage parameters. The engineer of record can use one of the shrinkage models available in AASHTO or ACI 209 “Guide for Modeling and Calculating Shrinkage and Creep in Hardened Concrete” to extrapolate the ultimate shrinkage strain based on strain data collected up to 28 days. A mix design for the cast-in-place concrete topping with low shrinkage and high creep is also recommended for composite bridges with steel girders. Because structural steel does not shrink or creep the effects of differential shrinkage and creep would be as pronounced as in the case considered in this study when the age of continuity for the precast beams is assumed to be 90 days.

- **Cross-Sectional Shape** - Inverted T-beam system reduces the tensile stresses in the CIP topping as a result of differential shrinkage compared to voided slabs and adjacent box girders by providing a smaller moment arm between the centroid of cast-in-place topping and that of the precast beam. In addition, the time dependent analysis performed at the cross-sectional level demonstrated that tensile stresses in the CIP topping were highest at section 2. Tapering the webs of the precast inverted T-beams helps reduce the area over which Section 2 applies and consequently reduces the likelihood of deck cracking. As a result it is recommended that precast inverted T-beams be used in the construction of short-to-medium-span bridges.

- **Mild steel** – While mild steel in the cast-in-place topping restrains its free shrinkage, tensile stresses in the CIP topping are not greatly influenced by the amount of mild steel
in the topping. Mild steel needs to be provided to control the cracks and help distribute live loads in the transverse direction and its presence does not significantly contribute towards increasing the likelihood of deck cracking

- **Boundary Conditions** – In two-span continuous bridges it is essential accommodate axial movement in the longitudinal direction at the abutments to avoid tensile stresses in the deck due to differential shrinkage, negative temperature gradients and uniform decreases in temperature. As a result, it is recommended that bearings at the abutments are detailed such that they emulate a roller support.

- **Age of Continuity** – It is recommended that the age of continuity is selected such that the competing effects of positive and negative restraint moment cancel each other as much as possible. High positive restraint moments negate the effects of negative live load moments and may render a continuous design even more expensive than a design based on simply supported beams. High negative moments may create excessive cracking on the bridge decks and reduce the service life of bridges.

4.4.1 Comments

- It is important to note that the analysis performed at the cross-sectional level shows stress distributions that apply along the entire bridge superstructure. Because the precast inverted T-beams serve as stay-in-place forms for the cast-in-place topping, there are no stresses in the cast-in-place topping due to its self-weight. Accordingly, any tensile stresses created in the topping due to time dependent effects could not be counterbalanced by any compressive stresses due to the self-weight of the topping as those stresses apply only to the precast beams. Therefore, if the tensile stresses in the deck are higher than its rupture stress, cracks could potentially develop in the entire top surface of the bridge in the longitudinal and transverse directions.

- Similarly, tensile stresses developed as a result of axial restraints at the abutments in the longitudinal direction due to differential shrinkage, negative temperature gradient and a uniform decrease in temperature apply not only to the entire bridge superstructure but are also constant throughout the depth of the composite cross-section of the bridge. These high tensile stresses have the potential to develop full depth transverse cracks. In addition to the obvious serviceability and durability problems that these high tensile stresses can create, full depth cracks in regions of small moment can cause reductions in shear strength.

- Tensile stresses developed as a result of moment restraint due to differential shrinkage and negative/positive temperature gradients are maximum at the intermediate support and reduce linearly towards the abutments (for a two-span continuous bridge).
In summary, time dependent effects can cause stresses that are higher than the ones created due to mechanical loads and need to be considered in the analysis and design of bridges to prolong their longevity.

**Notation:**

\( A_D \) = area of cast-in-place deck
\( A_G \) = area of precast girder
\( A_{ps} \) = area of prestressing strands
\( A_s \) = area of mild steel
\( a \) = distance between the centroid of cast-in-place deck and centroid of precast girder.
\( a_D \) = distance between the centroid of the cast-in-place deck and centroid of composite section
\( a_G \) = distance between the centroid of the girder and centroid of the composite section
\( c.g. \) = center of gravity (centroid)
\( E_D \) = modulus of elasticity of the cast-in-place deck
\( E_G \) = modulus of elasticity of the precast girder
\( E_s \) = modulus of elasticity of mild steel
\( e \) = eccentricity of the prestressing force with respect to the centroid of the precast girder
\( f'_c \) = specified compressive strength of concrete for use in design (ksi)
\( I_D \) = moment of inertia of the cast-in-place deck
\( I_G \) = moment of inertia of the precast girder
\( y \) = distance from centroid
\( y_{D bottom} \) = distance from the centroid of the deck to the bottom of the composite section
\( y_{G bottom} \) = distance from the centroid of the precast girder to the bottom of the composite section
\( \alpha \) = coefficient of thermal expansion
\( \Delta \varepsilon_D \) = change in strain at the centroid of deck due to time dependent effects
\( \Delta \varepsilon_G \) = change in strain at the centroid of girder due to time dependent effects
\( \Delta \varepsilon_s \) = change in strain in mild steel due to time dependent effects
\( \Delta X \) = change in curvature due to time dependent effects
\( \Delta N \) = change in axial force due to restrained stress as a result of temperature gradient
\( \Delta M \) = change in moment due to restrained stress as a result of temperature gradient
\( \Delta N_D \) = change in axial force in the deck due to time dependent effects
\( \Delta N_G \) = change in axial force in the girder due to time dependent effects
\( \Delta N_s \) = change in force in mild steel due to time dependent effects
\( \Delta M_D \) = change in moment in the deck due to time dependent effects
\( \Delta M_G \) = change in moment in the girder due to time dependent effects
\( \Delta \sigma_D \) = change in stress in deck due to time dependent effects
\( \Delta \sigma_G \) = change in stress in precast girder due to time dependent effects
\( \Delta \sigma_s \) = change in stress in mild steel due to time dependent effects
\( \Delta T_1 \) = change in temperature at location 1 due to temperature gradient
\( \Delta T_2 \) = change in temperature at location 2 due to temperature gradient
\[ \varepsilon_1 = \text{free strain at location 1 due to temperature gradient} \]
\[ \varepsilon_2 = \text{free strain at location 2 due to temperature gradient} \]
\[ \varepsilon_{\text{SHD}} = \text{ultimate shrinkage strain of the deck} \]
\[ \varepsilon_{\text{SHG}} = \text{ultimate shrinkage strain of the precast girder} \]
\[ \varphi_D = \text{creep coefficient for the deck} \]
\[ \varphi_G = \text{creep coefficient for the precast girder} \]
\[ \mu = \text{aging coefficient} \]
\[ \sigma_s = \text{free stress/restrained stress in steel due to temperature gradient} \]
\[ \sigma_{d1} = \text{free stress/restrained stress in deck at location 1 due to temperature gradient} \]
\[ \sigma_{d2} = \text{free stress/restrained stress in deck at location 2 due to temperature gradient} \]
\[ \sigma_g = \text{free stress/restrained stress at the top of precast girder due to temperature gradient} \]

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Chapter 5: Reducing Deck Cracking in Composite Bridges by Controlling Shrinkage and Creep Properties

(To be submitted for *ACI Special Publication* on the Sustainable Performance of Concrete Bridges – ACI 2014 Fall Convention)
Reducing Deck Cracking in Composite Bridges by Controlling Long Term Properties

Abstract

Composite concrete bridges are widely used because they combine the advantages of precast concrete with those of cast-in-place concrete. However, because of the difference in shrinkage properties between the girder and the deck and because of the sequence of construction, the deck is subject to differential shrinkage tensile stresses. These tensile stresses may lead to excessive cracking. This paper demonstrates how the likelihood of deck cracking due to differential shrinkage can be reduced and how consequently the resistance of composite concrete bridges against time dependent effects can be enhanced by choosing a deck mix with low shrinkage and high creep. An experimental study on the long term properties of seven deck mixes is presented to identify a deck mix with the aforementioned properties. A comparison of three composite concrete bridge systems used for short-to-medium-span bridges is performed to identify the bridge system that is most resistant against time dependent effects. The mix with saturated lightweight fine aggregates appears to best alleviate tensile stresses due to differential shrinkage and the bridge system with precast inverted T-beams and tapered webs appears to be the most resistant.

5.1 Introduction

State departments of transportation expend significant effort and resources on the construction of durable bridges. A durable bridge is defined as one with a long service life and low maintenance requirements, which lead directly to low life-cycle costs. Deterioration of concrete bridges is the result of harmful environmental conditions such as: water, deicing chemicals, freezing temperatures, mechanical abrasion etc. The most serious threats to durability are generally posed by combinations of two or more of these conditions. Cracks in concrete bridge decks provide easy access for water and deicing chemicals, which shorten the life of the deck, especially in bridges subject to aggressive environments. Both materials increase the effects of freeze-thaw damage, while the deicing chemicals lead to higher concentrations of chlorides, and subsequently, corrosion of reinforcing steel. While there is no general consensus on the influence of crack width on corrosion of reinforcing steel, there is agreement that thickness and porosity of concrete covering the reinforcement are important parameters influencing corrosion. There are generally two approaches that can be taken with regards to cracking.

The first approach is to design concrete members not to crack under service loads. To do this the sources of cracking must be identified and measures taken to reduce the potential for the development of cracks. These measures include designing not only for the mechanical loads but also for the structural effects of restrained deformations. Examples of restrained deformations include differential settlements, differential shrinkage, temperature gradients, and uniform changes in temperature. Measures that can be taken to reduce the potential for cracking include modifying...
concrete material properties and adjusting construction sequences and techniques. Article 3.3.2 of AASHTO\textsuperscript{4} classifies force effects due to shrinkage and creep as permanent loads. In addition, Table 3.4.1-1 in AASHTO\textsuperscript{4} stipulates the service and strength level load combinations which include the effects of shrinkage and creep.

The second approach is to design concrete members to crack under service loads and take measures to control crack width. For example code provisions provide guidelines on the amount of reinforcing steel needed to control crack width. A combination of both approaches is also a viable option, which includes taking measures to reduce the likelihood of cracking but also includes specifications for bar size, spacing and concrete cover to control crack width if cracks were to occur.

The discussion presented in this paper focuses on cracking that may be developed in the cast-in-place deck as a result of differential shrinkage and demonstrates how the likelihood of cracking in short-to-medium-span bridges can be reduced by controlling the long term properties of the deck concrete and by choosing a bridge system that is resistant against time dependent effects.

5.2 Scope of Study

The scope of this study is to demonstrate the potential for differential concrete shrinkage to induce tensile stresses in cast-in-place bridge deck in excess of the tensile strength of concrete and recommend a deck mix that reduces the likelihood of cracking due to differential shrinkage. In addition, one traditional and two relatively new bridge systems intended for short-to-medium-span bridges are investigated for their resistance to tensile stresses resulting from time dependent effects. A series of seven deck mixes developed by the researchers at Virginia Center for Transportation Innovation and Research (VCTIR) are investigated by measuring: compressive strength, modulus of elasticity, splitting tensile strength, shrinkage, and creep. A numeric investigation and several finite element analyses are conducted with the purposes of simulating and quantifying the effects of differential shrinkage in short-to-medium-span bridges and ranking these three bridge systems in order of resistance to this phenomenon.

5.3 Research Significance

Excessive deck cracking leads to deck deterioration and expensive repair and rehabilitation procedures. Cracks in concrete bridge decks provide easy access for water and deicing chemicals to reinforcement, which shorten the life of the deck, especially in bridges subject to aggressive environments. One cause of excessive deck cracking is differential shrinkage between the cast-in-place concrete deck and the precast concrete girder. The restraint of the shrinkage of the deck by the girder creates tensile stresses in the deck which may exceed the tensile strength of deck concrete. This paper demonstrates how the likelihood of deck cracking in short-to-medium-span bridges can be reduced by controlling the long term properties of the deck and by choosing a bridge
system that reduces the extent of critical tensile stresses in the deck resulting from differential shrinkage.

5.4 Structural Effects of Differential Shrinkage and Creep in Composite Systems

The inherent difference in shrinkage and creep properties between a cast-in-place deck and precast girder will cause self-equilibrating stresses at the cross-sectional level in a composite system. Even if the composite girder is used in a single span simply supported bridge, these self-equilibrating stresses will form along the entire span of the bridge. The difference in shrinkage properties is exacerbated by the difference in age between the two components. As a result, when the cast-in-place deck is placed, it will tend to shrink while the majority of the shrinkage in the precast girder has already taken place. The restraint provided by the precast girder to the free shrinkage of the deck will create a tensile force in the deck while the free shrinkage of the deck will exert a compressive force in the precast girder. In addition, because the centroids of the precast and cast-in-place components are at different locations, this differential shrinkage will cause a positive curvature. The curvature will result in a prestress gain in the bottom layer of prestressing in the precast girder, whereas the compression force from the shrinkage of the deck will cause a prestress loss.

Figure 5.1 shows the redistribution of forces in a composite system consisting of a precast prestressed concrete girder and a cast-in-place deck. The time dependent strain at any fiber in the precast girder or cast-in-place deck can be calculated by summing the elastic and creep strains due to initial stresses, elastic and creep strains due to changes in stress, and the shrinkage strain (Equation 1).

\[
\varepsilon_t = \frac{\sigma_0}{E_0} \left(1 + \varphi_{t,t_0}\right) + \int_{t_0}^{t} \frac{1}{E(t)} \left[ \frac{d\sigma(t)}{dt} \left(1 + \varphi(t,\tau)\right) \right] d\tau + \varepsilon_{sh,t} \tag{1}
\]

where
- \( \varepsilon_t \) = total strain at time \( t \)
- \( \sigma_0, \sigma(\tau) \) = stress at time \( t_0 \) and \( \tau \), respectively
\( E_0, E(t) \) = modulus of elasticity at times \( t_0 \) and \( t \), respectively  
\( \varphi_{t,t_0}, \varphi(t,\tau) \) = creep coefficient at time \( t \) due to load applied at time \( t_0 \) and \( \tau \), respectively  
\( \varepsilon_{sh,t} \) = shrinkage strain at time \( t \)

Term 1 represents the elastic and creep strains due to a stress applied at time \( t_0 \). Term 2 represents the elastic and creep strains due to changes in stress in the time interval \( t_0 \) to \( t \). Term 3 represents the shrinkage strain at time \( t \).

The integral term can be replaced by an algebraic expression if an aging coefficient \( \mu \) is introduced\(^5,6\). Equation 1 can then be reformulated as follows:

\[
\varepsilon_t = \frac{\sigma_0}{E_0} \left( 1 + \varphi_{t,t_0} \right) + \frac{\Delta \sigma}{E_0} \left( 1 + \mu \varphi_{t,t_0} \right) + \varepsilon_{sh,t}\tag{2}
\]

As defined earlier \( \sigma_0 \) represent initial stresses. These stresses can be created by forces and moments that are initially directly applied to the precast girder or the cast-in-place deck (\( M_{Ddirect}^0, N_{Ddirect}^0, M_{Gdirect}^0, N_{Gdirect}^0, N_{psdirect}^0 \)) or by forces and moments that are initially applied to the composite system (\( M^0, N^0 \)). For example an eccentric prestressing force in a pre-tensioned girder creates axial forces and bending moments that are applied directly to the girder in addition to the axial force applied directly to the prestressing strand. To utilize the free shrinkage and creep properties of the precast and cast-in-place concrete it is useful to decompose the internal forces and moment acting on the composite section, into forces and moments acting separately on the girder and deck (\( M_D^0, N_D^0, M_G^0, N_G^0, N_{ps}^0 \)). Examples of axial forces and bending moment that are applied initially to the composite system include post-tensioning forces applied after the system is made composite and bending moments created due to superimposed dead loads.

The term \( \Delta \sigma \) in Equation 2 represents the change in stress due to changes in axial forces and bending moments in the deck, girder or prestressing strands. These are denoted as \( \Delta M_D, \Delta N_D, \Delta M_G, \Delta N_G \) and \( \Delta N_{ps} \) in

Figure 5.1. The changes in strains and stresses due to time dependent effects can be calculated by using equations of material constitutive relationships, equilibrium and compatibility. For example the change in axial strain at the centroid of the deck and girder can be expressed by summing the creep strain due to initial axial forces, the elastic and creep strain due to the change in axial force over time and the free shrinkage strain (Equation 3.1 and 3.3). The change in curvature in the deck and girder can be expressed by summing the creep curvature due to initial moments and the elastic and creep curvatures due to changes in these moments over time (Equations 3.2 and 3.4). The change in prestressing strand strain over time can be simply calculated by dividing the change in stress in the strand by the modulus of elasticity of the strand (Equation 3.5). The change in stress in the strand over time represents either a prestress loss or prestress gain due to differential shrinkage and creep.

In a statically determinate system, because there are no externally applied axial forces over time, the sum of changes in the axial forces in each component must equal zero (Equation 3.6). In addition, in such a determinate system because there are no externally applied bending moments over time, the sum of moment about the centroid of the girder must equal zero (Equation 3.7).
Finally, assuming perfect bond between deck concrete, girder concrete and prestressing strands, the axial strains at the centroid of each of these components can be inter-related by using the change in curvature and the distances between the centroids (Equations 3.8 and 3.9).

Equations 3.1-3.9 form a set of linear equations which can be solved simultaneously to determine the 9 unknowns caused by the time dependent effects (\(\Delta \varepsilon_D, \Delta \varepsilon_G, \Delta \varepsilon_{ps}, \Delta X, \Delta N_D, \Delta N_G, \Delta N_{ps}, \Delta M_D, \Delta M_G\)). Initial and final stresses can then be calculated using equations 4.1-4.6. Because differential shrinkage can cause tensile stresses in the deck that may exceed the tensile strength of concrete, a deck mix with low shrinkage will reduce the likelihood of cracking. In addition, a deck mix with high creep will alleviate the tensile stresses created as a result of differential shrinkage. The following section presents an experimental investigation on seven deck mixes with the purpose of identifying the mix with lowest free shrinkage and highest creep.

5.5 Experimental Evaluation

Seven deck mixes developed by the researchers at VCTIR were put through a battery of tests to investigate their short term and long term properties. The deck mixes contained normal weight and light weight coarse and fine aggregates. The mixture proportions for each mix design are provided in Table 5.1. The cementitious materials that were used were fly ash and blast furnace slag. To increase the workability of the mixes without increasing the water content, superplasticizer was used as needed. In addition, because bridge decks are structural components that are exposed to weather, an air-entrained mix is typically used to improve durability when the deck is subjected to freeze-thaw cycles and deicing salts. This improvement is typically accomplished by adding an air entraining agent. The resulting even distribution of pores in the concrete prevents large air voids from forming and breaks down the capillary pathways from the surface to the reinforcement. Target material properties were as follows:

- Minimum compressive strength at 28 days = 4000 psi
- Maximum coarse aggregate size for:
  - normal weight mixes = No.57 stone (1 in.)
  - light weight mixes = \(\frac{3}{4}\) in.
- Minimum cementitious materials content = 635 lbs/yd\(^3\)
- Maximum water cementitious materials ratio for:
  - normal weight mixes = 0.45
  - light weight mixes = 0.43
- Slump = 4 in. to 7 in.
- Air Content = 6 ½ ± 1 ½ %

When a high-range water reducer (superplasticizer) was used, the upper limit on air content was increased by 1 %. One batch was placed for each concrete mix design.
Material Constitutive Relationships

\[ \Delta \varepsilon_D = \frac{N_{0D}^D}{E_D A_D} \varphi_D + \frac{\Delta N_D}{E_D A_D} (1 + \mu \varphi_D) + \varepsilon_{SHD} \]

(3.1)

where: \[ N_D^0 = \frac{n A_D}{A_c} N^0 - \frac{M^0 a_D E_D A_D}{I_c E_G} \]

\[ \Delta X = \frac{M_{0D}^D}{E_D I_D} \varphi_D + \frac{\Delta M_D}{E_D I_D} (1 + \mu \varphi_D) \]

(3.2)

where: \[ M_D^0 = \frac{M^0 E_D I_D}{E_G I_c} \]

\[ \Delta \varepsilon_G = \frac{N_{0G}^G}{E_G A_G} \varphi_G + \frac{\Delta N_G}{E_G A_G} (1 + \mu \varphi_G) + \varepsilon_{SHG} \]

(3.3)

where: \[ N_{Gdirect}^0 = -p_e + \frac{M^0 a_p E_p A_p}{I_c E_G}, \quad N_G^0 = \frac{A_G}{A_c} N^0 + \frac{M^0 a_D E_D A_D}{I_c E_G} \]

\[ \Delta X = \frac{M_{Gdirect}^0}{E_G I_G} \varphi_G + \frac{\Delta M_G}{E_G I_G} (1 + \mu \varphi_G) \]

(3.4)

where: \[ M_{Gdirect}^0 = -p_e + M_{Gself} + M_{Gslab}, \quad M_G^0 = \frac{M^0 I_G}{I_c} \]

\[ \Delta \varepsilon_{ps} = \frac{\Delta N_{ps}}{E_{ps} A_{ps}} \]

(3.5)

Equilibrium

\[ \Delta N_D + \Delta N_G + \Delta N_{ps} = 0 \]

(3.6)

\[ \Delta M_G + \Delta M_D - \Delta N_D a + \Delta N_{ps} e = 0 \]

(3.7)

Compatibility

\[ \Delta \varepsilon_D = \Delta \varepsilon_G - \Delta X (y_{Dbottom} - y_{Gbottom}) \]

(3.8)

\[ \Delta \varepsilon_{ps} = \Delta \varepsilon_G + \Delta X (y_{Gbottom} - y_{psbottom}) \]

(3.9)

Stresses

\[ \sigma_D = \left( \frac{N_{0D}^D + N_D^0}{A_D} + \frac{(M_{0D}^D + M_D^0)}{I_D} \right) \varphi_D \]

(4.1)

\[ \sigma_{Dfinal} = \left( \frac{N_{Ddirect}^0 + N_D^0 + \Delta N_D}{A_D} + \frac{(M_{Ddirect}^0 + M_D^0 + \Delta M_D)}{I_D} \right) \varphi_D \]

(4.2)

\[ \sigma_G = \left( \frac{N_{0G}^G}{A_G} + \frac{M_{Gdirect}^0}{I_G} \right) \varphi_G \]

(4.3)

\[ \sigma_{Gfinal} = \left( \frac{N_{Gdirect}^0 + N_G^0 + \Delta N_G}{A_G} + \frac{(M_{Gdirect}^0 + M_G^0 + \Delta M_G)}{I_G} \right) \varphi_G \]

(4.4)

\[ \sigma_{psinitial} = \left( \frac{N_{0ps}^0}{A_{ps}} \right) \]

(4.5)

\[ \sigma_{psfinal} = \left( \frac{N_{psdirect}^0 + N_G^0 + \Delta N_{ps}}{A_{ps}} \right) \]

(4.6)
### Table 5.1 Design Mixture Proportions for Topping Concrete

<table>
<thead>
<tr>
<th>Constituent</th>
<th>NWC-FA (lbs/yd³)</th>
<th>NWC-SL1 (lbs/yd³)</th>
<th>SLWC-FA (lbs/yd³)</th>
<th>SLWC-SL (lbs/yd³)</th>
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</thead>
<tbody>
<tr>
<td>Portland Cement</td>
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<td>476</td>
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<td>w/cm</td>
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</tbody>
</table>

<table>
<thead>
<tr>
<th>Constituent</th>
<th>NWC-SLFA-SL (lbs/yd³)</th>
<th>NWC-SL2 (lbs/yd³)</th>
<th>NWC-SLFA (lbs/yd³)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Portland Cement</td>
<td>382</td>
<td>382</td>
<td>635</td>
</tr>
<tr>
<td>Fly Ash</td>
<td>0</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>Slag Cement</td>
<td>254</td>
<td>254</td>
<td>0</td>
</tr>
<tr>
<td>Water</td>
<td>261</td>
<td>286</td>
<td>261</td>
</tr>
<tr>
<td>Coarse Aggregate</td>
<td>1733</td>
<td>1733</td>
<td>1733</td>
</tr>
<tr>
<td>NW Fine Aggregate</td>
<td>666</td>
<td>1285</td>
<td>666</td>
</tr>
<tr>
<td>LW Fine Aggregate</td>
<td>403</td>
<td>0</td>
<td>403</td>
</tr>
<tr>
<td>Total</td>
<td>3699</td>
<td>3940</td>
<td>3698</td>
</tr>
<tr>
<td>Unit Weight</td>
<td>137</td>
<td>146</td>
<td>137</td>
</tr>
<tr>
<td>w/cm</td>
<td>0.41</td>
<td>0.45</td>
<td>0.41</td>
</tr>
</tbody>
</table>

NWC-SL = normal weight coarse aggregate slag mix1, NWC-FA = normal weight coarse aggregate fly ash mix, SLWC-SL = saturated light-weight coarse aggregate slag mix, SLWC-FA = saturated light-weight coarse aggregate slag mix, NWC-SLFA-SL = normal weight coarse aggregate with saturated light weight fines aggregates and slag, NWC-SL2 = normal weight coarse aggregate slag mix2, NWC-SLFA = normal weight coarse aggregate with saturated light weight fines aggregates

5.5.1 Short Term Properties

Short term properties determined for the deck mixes include compressive strength, splitting tensile strength, and modulus of elasticity. These short term properties are useful in assessing the quality of concrete and the response to short-term loads such as vehicle live loads (Barker 2007). Sometimes these short term properties are modified to account for the long term effects. For example, the Age Adjusted Effective Modulus (AAEM) method accounts for the increase in strain due to creep of concrete under sustained loads by employing a reduced long term modulus of elasticity. The compressive strength, splitting tensile strength and modulus of elasticity were determined in accordance with ASTM C39, ASTM C496, ASTM C469 respectively. The short term properties for all seven mixes are provided in Table 5.2, Table 5.3 and Table 5.4.
Experimental data for tensile strength and modulus of elasticity are compared to the values obtained using the equations in AASHTO\textsuperscript{4}. These equations are provided in AASHTO\textsuperscript{4} Section 5.4.2.4 and Section 5.4.2.6-7, respectively. When cracking is caused by the effects of flexure, AASHTO\textsuperscript{4} provides a series of equations for the determination of modulus of rupture (\(f_r\)) for both normal-weight and light weight concrete. These values vary between 0.20\(\sqrt{f'c}\) to 0.24\(\sqrt{f'c}\) for normal weight concrete and between 0.17\(\sqrt{f'c}\) to 0.20\(\sqrt{f'c}\) for lightweight concrete. The commentary of Section C5.4.2.6 states that data show that most modulus of rupture values are between 0.24\(\sqrt{f'c}\) and 0.37\(\sqrt{f'c}\). In addition, the commentary of Section C5.4.2.7 states that the given values may be un-conservative for tensile cracking caused by restrained shrinkage, anchor zone splitting, and other similar tensile forces caused by effects other than flexure and that the direct tensile strength stress (\(f_t\)) should be used in these cases. Equation 5 is taken from the commentary of Section 5.4.2.7 –Tensile Strength and may be used for normal weight concrete with specified compressive strengths up to 10 ksi.

Because the focus of this paper is potential cracking due to restrained differential shrinkage, Equation 5 is used to compare calculated and tested tensile strength values. Equation 6 is used to calculate the tensile strength of mixes that contained light weight coarse aggregates and normal weight fine aggregates (sand-lightweight). There is no equation in AASHTO\textsuperscript{4} that is applicable to the mixes that contained normal weight coarse aggregates and a mixture of normal weight and light weight fines aggregates. Because the normal weight aggregates represented the majority of aggregates in these mixes, Equation 5 was used for comparison with tested values. However, as can be seen from the results in Table 5.3, Equation 5 overestimated the tensile strength of the two mixes that contained a mixture of normal weight and lightweight aggregates. In the calculation of concrete tensile strength using Equations 5 and 6, the tested values were used for the compression strength of concrete. Although in general, AASHTO’s\textsuperscript{4} equations overestimated the tensile strength of the investigated mixes, they provided reasonably good estimates for design purposes. The tensile strength of concrete is an important short term property because the likelihood of cracking is estimated by comparing the magnitude of tensile stresses created due to differential shrinkage with the tensile strength of the cast-in-place concrete deck. Table 5.3 provides also a summary of the tested tensile strength of the seven concrete mixes at 28 days. The mix with the lowest tensile strength was the one with normal weight coarse aggregates and saturated lightweight fines. The mix with the highest tensile strength was the mix with normal weight coarse aggregates and slag denoted (NWC-SL2).

\[ f_t = 0.23\sqrt{f'c} \text{ where } f'_c \text{ is in ksi}, \]  
\[ f_t = 0.20\sqrt{f'c} \text{ where } f'_c \text{ is in ksi}, \]  
\[ E_c = 33,000 \ K_1 w_c^{1.5} \sqrt{f'c} \text{ where } w_c \text{ is in k/ft}^3 \text{ and } f'_c \text{ is in ksi} \]  

\( \)
Table 5.2 Compressive strength test results

<table>
<thead>
<tr>
<th>Age (days)</th>
<th>NWC-FA</th>
<th>NWC-SL1</th>
<th>SLWC-FA</th>
<th>SLWC-SL</th>
<th>NWC-SLWF-SL</th>
<th>NWC-SL2</th>
<th>NWC-SLWF</th>
</tr>
</thead>
<tbody>
<tr>
<td>7</td>
<td>3100</td>
<td>3580</td>
<td>2600</td>
<td>4020</td>
<td>3660</td>
<td>4080</td>
<td>2650</td>
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<tr>
<td>14</td>
<td>3530</td>
<td>nm</td>
<td>3790</td>
<td>5270</td>
<td>4130</td>
<td>4830</td>
<td>3280</td>
</tr>
<tr>
<td>28</td>
<td>4260</td>
<td>5200</td>
<td>4600</td>
<td>5950</td>
<td>4560</td>
<td>5370</td>
<td>3540</td>
</tr>
<tr>
<td>56</td>
<td>4140</td>
<td>5250</td>
<td>4910</td>
<td>6420</td>
<td>nm</td>
<td>5410</td>
<td>3610</td>
</tr>
<tr>
<td>90</td>
<td>4060</td>
<td>5410</td>
<td>4880</td>
<td>6440</td>
<td>nm</td>
<td>nm</td>
<td>nm</td>
</tr>
</tbody>
</table>

nm=not measured, nc=not calculated

Table 5.3 Tensile strength test results

<table>
<thead>
<tr>
<th>Age (days)</th>
<th>NWC-FA</th>
<th>Tensile Strength, (psi)</th>
<th>NWC-SL1</th>
<th>Tensile Strength, (psi)</th>
<th>NWC-SL2</th>
<th>Tensile Strength, (psi)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Tested</td>
<td>Eqn. 5</td>
<td>Tested/(Eqn. 5)</td>
<td>Tested</td>
<td>Eqn. 5</td>
<td>Tested/(Eqn. 5)</td>
</tr>
<tr>
<td>7</td>
<td>317</td>
<td>405</td>
<td>0.78</td>
<td>391</td>
<td>435</td>
<td>0.90</td>
</tr>
<tr>
<td>28</td>
<td>418</td>
<td>475</td>
<td>0.88</td>
<td>455</td>
<td>524</td>
<td>0.87</td>
</tr>
<tr>
<td>90</td>
<td>412</td>
<td>463</td>
<td>0.89</td>
<td>541</td>
<td>535</td>
<td>1.01</td>
</tr>
<tr>
<td>Avg. = 0.85</td>
<td></td>
<td></td>
<td></td>
<td>Avg. = 0.93</td>
<td></td>
<td></td>
</tr>
<tr>
<td>SLWC-FA</td>
<td>Tested</td>
<td>Eqn. 6</td>
<td>Tested/(Eqn. 6)</td>
<td>Tested</td>
<td>Eqn. 6</td>
<td>Tested/(Eqn. 6)</td>
</tr>
<tr>
<td>7</td>
<td>274</td>
<td>322</td>
<td>0.85</td>
<td>374</td>
<td>401</td>
<td>0.93</td>
</tr>
<tr>
<td>28</td>
<td>370</td>
<td>429</td>
<td>0.86</td>
<td>391</td>
<td>488</td>
<td>0.80</td>
</tr>
<tr>
<td>90</td>
<td>435</td>
<td>442</td>
<td>0.98</td>
<td>503</td>
<td>508</td>
<td>0.99</td>
</tr>
<tr>
<td>Avg. = 0.90</td>
<td></td>
<td></td>
<td></td>
<td>Avg. = 0.91</td>
<td></td>
<td></td>
</tr>
<tr>
<td>NWC-SLWF-SL</td>
<td>Tested</td>
<td>Eqn. 5</td>
<td>Tested/(Eqn. 5)</td>
<td>Tested</td>
<td>Eqn. 5</td>
<td>Tested/(Eqn. 5)</td>
</tr>
<tr>
<td>7</td>
<td>377</td>
<td>440</td>
<td>0.86</td>
<td>417</td>
<td>470</td>
<td>0.89</td>
</tr>
<tr>
<td>28</td>
<td>370</td>
<td>470</td>
<td>0.79</td>
<td>483</td>
<td>510</td>
<td>0.95</td>
</tr>
<tr>
<td>90</td>
<td>nm</td>
<td>nc</td>
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<td>nm</td>
<td>nc</td>
<td>nc</td>
</tr>
<tr>
<td>Avg. = 0.83</td>
<td></td>
<td></td>
<td></td>
<td>Avg. =</td>
<td></td>
<td></td>
</tr>
<tr>
<td>NWC-SLWF</td>
<td>Tested</td>
<td>Eqn. 5</td>
<td>Tested/(Eqn. 5)</td>
<td>NWC-FA</td>
<td></td>
<td>418</td>
</tr>
<tr>
<td>7</td>
<td>287</td>
<td>370</td>
<td>0.78</td>
<td>NWC-SL1</td>
<td></td>
<td>455</td>
</tr>
<tr>
<td>28</td>
<td>340</td>
<td>420</td>
<td>0.81</td>
<td>SLWC-FA</td>
<td></td>
<td>370</td>
</tr>
<tr>
<td>90</td>
<td>nm</td>
<td>nc</td>
<td>nc</td>
<td>SLWC-SL</td>
<td></td>
<td>391</td>
</tr>
<tr>
<td>Avg. = 0.80</td>
<td></td>
<td></td>
<td></td>
<td>NWC-SLWF-SL</td>
<td></td>
<td>370</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>NWC-SL2</td>
<td></td>
<td>483</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>NWC-SLWF</td>
<td></td>
<td>340</td>
</tr>
</tbody>
</table>

nm=not measured, nc=not calculated

Tested modulus of elasticity values for the seven mixes were compared with those calculated using Equation 7 from AASHTO⁴. Tested unit weight and compressive strength values were used in the evaluation of Equation 7. In general, Equation 7 underestimated the modulus of elasticity of the seven investigated mixes, however, it provided reasonably fair estimates for design purposes. Modulus of elasticity is another short term property that plays an important role in the evaluation of composite bridge systems for the effects of differential shrinkage. Because in this study the quantification of stresses due to differential shrinkage is based on the age adjusted effective modulus method, a higher modulus of elasticity for the deck leads to higher tensile stresses in the deck. Conversely, a lower modulus of elasticity for the deck leads to a lower age adjusted effective modulus and represents a mix that can alleviate the tensile stresses in the deck.
created as a result of differential shrinkage. In addition, with time the modulus of elasticity of concrete increases, which is why in the age adjusted effective modulus method employed in this study uses an aging coefficient, which accounts for this effect. Such an increase in the modulus of elasticity works against the concept of alleviating tensile stresses as a result of differential shrinkage because it makes the mix stiffer and less accommodating towards restrained deformations.

Table 5.4 Modulus of elasticity test results

<table>
<thead>
<tr>
<th>Age (days)</th>
<th>NWC-FA</th>
<th>NWC-SL1</th>
<th>SLWC-FA</th>
<th>SLWC-SL</th>
<th>NWC-SLWF-SL</th>
<th>NWC-SL2</th>
<th>NWC-SLWF</th>
<th>Summary of Tested Values, E (ksi)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Tested</td>
<td>Eqn. 3</td>
<td>Tested/Eqn. 3</td>
<td>Tested</td>
<td>Eqn. 3</td>
<td>Tested/Eqn. 3</td>
<td>Tested</td>
<td>Eqn. 3</td>
</tr>
<tr>
<td>7</td>
<td>4530</td>
<td>3220</td>
<td>1.41</td>
<td>4760</td>
<td>3350</td>
<td>1.42</td>
<td></td>
<td></td>
</tr>
<tr>
<td>14</td>
<td>4280</td>
<td>3430</td>
<td>1.25</td>
<td>nm</td>
<td>nc</td>
<td>nc</td>
<td></td>
<td></td>
</tr>
<tr>
<td>28</td>
<td>4430</td>
<td>3770</td>
<td>1.18</td>
<td>5010</td>
<td>4040</td>
<td>1.24</td>
<td></td>
<td></td>
</tr>
<tr>
<td>56</td>
<td>4150</td>
<td>3720</td>
<td>1.12</td>
<td>5180</td>
<td>4060</td>
<td>1.28</td>
<td></td>
<td></td>
</tr>
<tr>
<td>90</td>
<td>4360</td>
<td>3680</td>
<td>1.18</td>
<td>4730</td>
<td>4120</td>
<td>1.15</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>Avg. = 1.23</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Age (days)</th>
<th>NWC-SLWF</th>
<th>Summary of Tested Values, E (ksi)</th>
</tr>
</thead>
<tbody>
<tr>
<td>7</td>
<td>3510</td>
<td>NWC-FA 4430</td>
</tr>
<tr>
<td>14</td>
<td>4295</td>
<td>NWC-SL1 5010</td>
</tr>
<tr>
<td>28</td>
<td>3990</td>
<td>SLWC-FA 3080</td>
</tr>
<tr>
<td>56</td>
<td>4670</td>
<td>SLWC-SL 3540</td>
</tr>
<tr>
<td>90</td>
<td>nm</td>
<td>NWC-SLWF-SL 4160</td>
</tr>
<tr>
<td></td>
<td></td>
<td>NWC-SL2 5460</td>
</tr>
<tr>
<td></td>
<td>Avg. = 1.36</td>
<td>NWC-SLWF 3990</td>
</tr>
</tbody>
</table>

nm = not measured, nc = not calculated

Table 5.4 provides a summary of the moduli of elasticity values for the seven mixes at 28 days. The mix with the lowest modulus of elasticity is the mix with saturated light weight coarse aggregates.
and fly ash. Whereas the mix with the highest modulus of elasticity is the one with normal weight coarse aggregates and slag denoted (NWC-SL2). The comparison of tensile strength and modulus of elasticity for the seven mixes illustrates how it is difficult to find a mix that embodies all the desired properties. For example the mix denoted NWC-SL2 had the highest tensile strength but also the highest modulus of elasticity.

5.5.2 Long Term Properties

Typically, compressive strength, tensile strength and modulus of elasticity of concrete increase with age, however, in this paper the phrase long term properties is used to describe shrinkage and creep properties of concrete. Shrinkage is considered to be a change in volume during hardening and drying under constant temperature, whereas creep is defined as increase in strain over time under constant stress. Shrinkage and creep tests were performed in accordance with ASTM C 157\textsuperscript{11} and ASTM C512\textsuperscript{12}, respectively. Creep specimens were loaded at 7 days and the applied load was maintained at approximately $0.4f'_c$ (where $f'_c$ is the average cylinder compressive strength at 7 days). The time dependent properties of concrete are influenced by the environmental conditions at the time of placement and throughout its service life\textsuperscript{7}. These properties are used in determining the structural effects of differential shrinkage and creep. ACI 209.2\textsuperscript{13} provides four models for calculating shrinkage and creep properties as a function of time. AASHTO\textsuperscript{4} has its own models for creep and shrinkage. ACI 209.2\textsuperscript{13} states that: “The variability of shrinkage and creep test measurements prevents models from closely matching experimental data. The within-batch coefficient of variation for laboratory-measured shrinkage on a single mixture of concrete was approximately 8%\textsuperscript{14}. Hence, it would be unrealistic to expect results from prediction models to be within plus or minus 20% of the test data for shrinkage. Even larger differences occur for creep predictions. For structures where shrinkage and creep are deemed critical, material testing should be undertaken and long-term behavior extrapolated from the resulting data.” It should be noted that the tests performed on the deck mixes described in this investigation were performed only on one batch for each mix. It was outside the scope of study to investigate the repeatability and consistency of the results obtained from each mix. One of the purposes of this study was to demonstrate the structural effects of differential shrinkage and creep and how they may influence the durability of composite bridges. The next section describes the results from the shrinkage and creep tests and identifies the mix with the desired long term properties.

5.5.3 Comparison of Shrinkage and Creep Properties for each Mix

Figure 5.2 shows the development of drying shrinkage strains with time for the seven mixes investigated. In this paper shortening strains are positive. Because the changes in shrinkage strains recorded after 70 days were generally negligible, shrinkage testing was stopped at 100 days. It is important to note that the measured shrinkage strains represent only drying shrinkage strains. Shrinkage specimens were moist cured for 7 days and were then exposed to drying. Table 5.5
provides a summary of the shrinkage strains at 100 days. The mix with the lowest shrinkage was the mix with normal weight coarse aggregates and saturated lightweight fine aggregates. The mix with the highest shrinkage was the mix saturated light weight coarse aggregates and slag. Both mixes that contained saturated lightweight coarse aggregates exhibited the highest shrinkage strains.

Table 5.5 Summary of experimental shrinkage strains at 100 days

<table>
<thead>
<tr>
<th>Mix</th>
<th>Shrinkage strains at 100 days (ue)</th>
</tr>
</thead>
<tbody>
<tr>
<td>NWC-FA</td>
<td>466</td>
</tr>
<tr>
<td>NWC-SL1</td>
<td>483</td>
</tr>
<tr>
<td>SLWC-FA</td>
<td>603</td>
</tr>
<tr>
<td>SLWC-SL</td>
<td>606</td>
</tr>
<tr>
<td>NWC-SLWF-SL</td>
<td>310</td>
</tr>
<tr>
<td>NWC-SL2</td>
<td>264</td>
</tr>
<tr>
<td>NWC-SLWF</td>
<td>215</td>
</tr>
</tbody>
</table>

Figure 5.3 shows the development of measured strains with time during the creep test. Figure 5.3(a) shows the total strain which is defined in Equation 8. Total strain is equal to the summation of elastic strain, shrinkage strain and creep strain. Elastic strain is the strain measured immediately after the creep specimens are loaded. Shrinkage strain is the strain due to the shrinkage of the creep specimens during the creep tests and is measured from unloaded companion cylinders. Creep strain is the increase in strain in the creep specimens over time as a result of the applied load.

\[
\text{total strain} = \text{elastic strain} + \text{shrinkage strain} + \text{creep strain} \tag{8}
\]

Because the change in total strains recorded from the creep test after 80 days were negligible, creep tests were terminated at a 106 days. Figure 5.3(b) shows the shrinkage strain measured in the creep specimens, which was deducted from the total strain to obtain the stress induced strain. Figure 5.3(c) shows the stress induced strain which is defined in Equation 9. Stress induced strain is the strain caused only by the sustained load (or sustained stress) over time and
can be expressed as the summation of the elastic strain and creep strain. Alternatively, stress induced strain can be expressed as the total strain measured in the creep specimens minus the shrinkage strain measured in unloaded companion cylinders.

\[
\text{stress induced strain} = \text{elastic strain} + \text{creep strain} \quad \text{or} \quad \text{stress induced strain} = \text{Total strain} - \text{shrinkage strain}
\]

Figure 5.3(d) shows the creep strain. Creep strain was obtained from deducting the elastic strain from the stress induced strain. One of the parameters used in the analysis for time dependent effects is the creep coefficient. Creep coefficient is the ratio of creep strain to the initial elastic strain (Equation 10).

\[
\text{creep coefficient} = \frac{\text{drying creep strain}}{\text{initial elastic strain}}
\]

Table 5.6 provides a summary of the experimental data from creep tests. Elastic strains for the two mixes containing saturated lightweight coarse aggregates are higher than the rest of the mixes. This is expected because lightweight mixes typically have lower moduli of elasticity compared to normal weight mixes (as shown in Table 5.4). There were several differences between the shrinkage strains measured during the shrinkage tests and those measured from the unloaded companion cylinders during the creep tests. For example the NWC-SLWF, NWC-SL2 and NWC-SLWC-SL mixes exhibited the lowest shrinkage strains compared to the other mixes. The lowest shrinkage strain however in the creep tests was measured in the NWC-SL2 mixes as opposed to the NWC-SLFW mix. Shrinkage strains measured during the creep test in the NWC-FA, NWC-SL1, SLWC-FA and SLWC-SL were also not entirely consistent with those measured from the shrinkage prisms. The highest disparity was observed in the shrinkage measurements for the SLWC-SL mix. During the creep tests this mix exhibited the lowest shrinkage strain compared to NWC-FA, NWC-SL1 and SLWC-FA mixes, whereas the data from the shrinkage prisms revealed the opposite.

Because creep strain is calculated by subtracting elastic and shrinkage strains from the total strain the magnitude of the elastic and shrinkage strains plays an important role in this determination. In this study creep coefficient is used as the metric for comparing creep properties of the seven mixes, because it is this coefficient that is employed in the age adjusted effective modulus method.

Table 5.7 provides a summary of the long terms properties of the seven investigated mixes. The NWC-SLWF mix exhibited the lowest shrinkage, whereas the SLWC-SL mix exhibited the highest one. The NWC-SLWF mix also exhibited the highest creep coefficient, whereas the SLWC-SL mix exhibited the lowest creep coefficient. While it may be typically difficult to find a mix that exhibits both low shrinkage and high creep properties, in this study the NWC-SLWF mix possessed both of these characteristics and is considered to be the most desirable mix, whereas the SLWC-SL mix is considered to be the least desirable one. For other situations where the
combination of low shrinkage and high creep may not be possible, priority should be given the
mix with the lowest shrinkage because it is the free shrinkage of the deck that serves as a catalyst
for the creation of tensile stresses in the cast-in-place topping and potentially excessive cracking.
In addition, sensitivity studies can be performed for a given structure to determine the influence of
the short and long term properties of the concrete materials on the structural effects of shrinkage
and creep.

Figure 5.3 Creep test – (a) total strain, (b) shrinkage strain, (c) stress induced strain, (d) creep
strain

Figure 5.4. Creep coefficient vs time
Table 5.6 Summary of experimental data from the creep tests

<table>
<thead>
<tr>
<th></th>
<th>NWC-FA</th>
<th>NWC-SL1</th>
<th>SLWC-FA</th>
<th>SLWC-SL</th>
<th>NWC-SLWF-SL</th>
<th>NWC-SL2</th>
<th>NWC-SLWF</th>
</tr>
</thead>
<tbody>
<tr>
<td>Elastic strain (ue)</td>
<td>439</td>
<td>442</td>
<td>711</td>
<td>720</td>
<td>470</td>
<td>498</td>
<td>372</td>
</tr>
<tr>
<td>Shrinkage strain (ue)</td>
<td>402</td>
<td>358</td>
<td>527</td>
<td>321</td>
<td>276</td>
<td>214</td>
<td>260</td>
</tr>
<tr>
<td>Creep strain (ue)</td>
<td>819</td>
<td>548</td>
<td>868</td>
<td>502</td>
<td>416</td>
<td>511</td>
<td>719</td>
</tr>
<tr>
<td>Total strain (ue)</td>
<td>1660</td>
<td>1348</td>
<td>2106</td>
<td>1543</td>
<td>1162</td>
<td>1223</td>
<td>1351</td>
</tr>
<tr>
<td>Creep Coefficient</td>
<td>1.87</td>
<td>1.24</td>
<td>1.22</td>
<td>0.70</td>
<td>0.89</td>
<td>1.03</td>
<td>1.93</td>
</tr>
</tbody>
</table>

Table 5.7 Summary of experimental data on shrinkage and creep properties (100 days)

<table>
<thead>
<tr>
<th>Mix</th>
<th>Drying shrinkage strain (ue)</th>
<th>Creep coefficient</th>
</tr>
</thead>
<tbody>
<tr>
<td>NWC-FA</td>
<td>466</td>
<td>1.87</td>
</tr>
<tr>
<td>NWC-SL1</td>
<td>483</td>
<td>1.24</td>
</tr>
<tr>
<td>SLWC-FA</td>
<td>603</td>
<td>1.22</td>
</tr>
<tr>
<td>SLWC-SL</td>
<td>606</td>
<td>0.70</td>
</tr>
<tr>
<td>NWC-SLWF-SL</td>
<td>310</td>
<td>0.89</td>
</tr>
<tr>
<td>NWC-SL2</td>
<td>264</td>
<td>1.03</td>
</tr>
<tr>
<td>NWC-SLWF</td>
<td>215</td>
<td>1.93</td>
</tr>
</tbody>
</table>

5.6 Numeric Example

To illustrate the advantages of a deck mix with low shrinkage and high creep a composite bridge system with adjacent voided slabs is considered. This type of system is typically used for short-to-medium-span bridges. Because the adjacent precast voided slabs serve as stay-in-place formwork, this system offers accelerated bridge construction and replacement by eliminating the need for formwork. Figure 5.5 shows a typical transverse cross-section for this bridge system. The cross-sectional dimensions shown in Figure 5.5 represent those used in a two-span continuous bridge constructed during the spring of 2014 near Richmond, VA. Prestressing and mild steel reinforcing details are not shown for clarity. A time dependent analysis using the age adjusted effective modulus method described earlier in this paper was conducted with the purpose of quantifying the distribution of initial and final stresses in such a composite bridge system. A total of seven analyses were conducted with the purpose of exploring the influence of the long term properties of the seven deck mixes on the final distribution of stresses in this composite bridge system. The variables in each analysis consisted of the tested short and long term properties of each deck mix. Because the contract documents specified an age of continuity of 90 days for the bridge in question, the ultimate shrinkage strain and the ultimate creep coefficient for the precast voided slab were taken equal to zero. The age of continuity refers to the time when the cast-in-place deck is placed over the adjacent precast voided slabs, making the two-span bridge continuous. This time starts after the fabrication of the individual precast voided slabs. 90 days is considered a long enough time to allow the majority of the shrinkage and creep to take place in the precast girder. The aging coefficient was taken equal to 0.7. This investigation was also conducted with the purpose of revealing which investigated deck mix offers the best combination of short term and long term properties with regards to minimizing the tensile stresses in the deck as a result of differential shrinkage.
Figure 5.6(a) shows the distribution of the initial longitudinal normal stresses in the precast voided slab due to its self-weight, the prestressing force and the weight of the cast-in-place topping. Because the adjacent precast voided slabs serve as stay-in-place formwork for the cast-in-place topping, they support the wet weight of the topping and initial stresses in the topping are zero. In addition, the differences in the initial stress distribution along the depth of the girder among the deck mixes is because of the different units weights of each topping mix. With time, the composite section will be subject to additional stresses due to differential shrinkage as the cast-in-place topping will try to shrink and the precast girder will tend to restrain this shrinkage. These additional stresses due to time dependent effects are illustrated in Figure 5.6(b). The combination of the initial stresses and those created due to time dependent effects is shown in Figure 5.6(c), which highlights the variation of final stresses depending on which topping mix is selected. For example if the NWC-SLWF mix is selected the maximum final tensile stress at the bottom of the cast-in-place topping is 0.214 ksi. Whereas, if the SLWC-SL mix is selected then the maximum tensile stress at the bottom of the cast-in-place topping can be as high as 0.756 ksi, which is greater than the modulus of rupture for the corresponding mix. Table 5.8 provides a summary of the maximum tensile stresses created at the bottom of cast-in-place topping and for each of the investigated mixes. It also provides a comparison between the maximum tensile stress at the bottom of the deck and the corresponding tensile strength for the mix used in that analysis. As can be seen, the lowest ratio is provided by the mix that has normal weight coarse aggregates and saturated lightweight fine aggregates, whereas the highest ratio is provided by the mix that contains saturated lightweight coarse aggregates and slag. Only two of the mixes provided ratios that were smaller than one, which means that if the other mixes are used for the cast-in-place topping than the bridge may exhibit transverse cracks along both spans. It should be noted that in the time dependent analysis presented herein, the short and long term properties were based on the tested values for each mix and ultimate shrinkage strains and creep coefficient were not adjusted to account for the relative humidity at the bridge site and the volume to surface ratio of the structural components. The purpose of this analysis is to demonstrate how the long term properties of the deck mix can influence the resistance of the composite bridge system against time dependent effects. As a result, it is essential to specify a topping mix with low shrinkage and high creep to reduce the likelihood of excessive cracking and consequently increase the longevity of composite concrete bridges.

Figure 5.5 Cross-section of composite voided slab system
Figure 5.6 Stresses in composite voided slab system (a) initial stresses due to mechanical loads, (b) changes due to time dependent effects, (c) final stresses

Table 5.8 Comparison of tensile stresses in the deck and the tensile strength of the deck

<table>
<thead>
<tr>
<th>Mix</th>
<th>$f_{\text{tmax}}$ (ksi)</th>
<th>$f_{\text{ttested}}$ (ksi)</th>
<th>Ratio = $f_{\text{tmax}}/f_{\text{ttested}}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>NWC-FA</td>
<td>0.504</td>
<td>0.418</td>
<td>1.21</td>
</tr>
<tr>
<td>NWC-SL1</td>
<td>0.653</td>
<td>0.455</td>
<td>1.44</td>
</tr>
<tr>
<td>SLWC-FA</td>
<td>0.591</td>
<td>0.370</td>
<td>1.60</td>
</tr>
<tr>
<td>SLWC-SL</td>
<td>0.756</td>
<td>0.390</td>
<td>1.94</td>
</tr>
<tr>
<td>NWC-SLWF-SL</td>
<td>0.407</td>
<td>0.370</td>
<td>1.10</td>
</tr>
<tr>
<td>NWC-SL2</td>
<td>0.399</td>
<td>0.483</td>
<td>0.83</td>
</tr>
<tr>
<td>NWC-SLWF</td>
<td>0.214</td>
<td>0.340</td>
<td>0.63</td>
</tr>
</tbody>
</table>

5.7 Finite Element Analyses

The method illustrated in Equations 3.1-4.6 using the age adjusted effective modulus is a sectional analysis method and can provide the distribution of longitudinal and transverse normal stresses due to time dependent effects for any given composite cross-section. Along the longitudinal axis of the bridge the voided slab composite section is prismatic and the results of time dependent analysis based on Equations 3.1 to 4.6 will apply at any given transverse cross-
However, because of the presence of the voids, the voided slab system is not prismatic in the transverse direction and the results of the time dependent analysis performed on a section by section basis will vary. To account for the interaction of these adjacent longitudinal sections a finite element model representing a typical transverse composite cross-section of the voided slab system was created. The material properties for the deck mix with normal weight coarse aggregates and saturated light weight fines were selected for this investigation because this mix had the best long term properties. To simulate the effects of differential shrinkage, the cast-in-place topping was subject to a uniform decrease in temperature. The magnitude of the decrease in temperature was chosen such that when combined with the coefficient of thermal expansion for concrete it resulted in a strain equal to the tested shrinkage strain at 100 days for the selected mix. No temperature effect was applied to the girder, to be consistent with the earlier assumption that when continuity is established the majority of the shrinkage and creep in the girder have already taken place. Creep effects were simulated using an age adjusted effective modulus. Poisson’s ratio for both mixes was taken as 0.2. The finite element model for the typical composite voided slab cross-section was for a simply supported beam with the purpose of investigating only the effects of differential shrinkage at a cross-sectional level and not those due to restraint moments that are present in a multi-span bridge application. The length of the span was 41.5 ft. A 2 in. mesh was used for both the cast-in-place topping and the precast voided slab.

Figure 5.7(a) shows a comparison of the magnitude and distribution of longitudinal normal stresses caused by differential shrinkage obtained based on Equations 3.1 to 4.6 with those obtained from the finite element model. The longitudinal stresses in the finite element model were obtained at mid-width of the composite cross-section. There are some small differences in the magnitude of these stresses because the values obtained from Equations 3.1 to 4.6 are based on a sectional analysis using a one-dimensional stress state and represent the average behavior of the typical composite cross-section. Whereas those obtained from the finite element model account for the interaction between sections above the voids with those above the precast webs and also account for the three-dimensional stress-state. The results obtained from Equations 3.1-4.6 provide a reasonable estimation of the stresses due to time dependent effects for design purposes and are a viable alternative to finite element analysis. Figure 5.7(b) illustrates that the location where the tensile stresses in the cast-in-place topping are highest is the interface between the girder and the deck. In addition, because the model represents a simply supported beam application, the magnitude of these stresses is relatively uniform along the span of the beam. Accordingly, if the highest tensile stress in the cast-in-place topping exceeds its tensile strength, transverse cracks will likely develop along the entire span of the beam. The spacing of these cracks will depend on the bond strength between the reinforcing steel and the surrounding concrete.

To compare the relative resistance of the voided slab system (Figure 5.8 (a)) against time dependent effects two other bridge systems were investigated. The second one is shown in Figure 5.8(b) and consists of adjacent precast Inverted T-beams with straight webs and a cast-in-place topping. This bridge system was identified in 2004 in France during a scanning tour funded by FWHA and AASHTO and has been used on several bridges in the state of Minnesota. The third
system is similar to the second one with the exception that the precast webs are tapered rather than straight (Figure 5.8(c)). This bridge system was implemented for the first time in Virginia for the replacement of a two-span continuous bridge on US 360 near Richmond, VA. The overall depth of the composite cross-section in the two inverted T-beam models was the same with the voided slab composite system. The width of the precast inverted T-beams was 6 ft. The width of the flange was 12 in., whereas the thickness of the flange was 4 in. The angle for the tapered webs in the system implemented in Virginia was 45°. The thickness of thinnest portion of the topping in both inverted T-beam systems matched that used in the voided slab system and was 7.5 in. The investigation of differential shrinkage induced stresses in the longitudinal and transverse directions of the bridge was of interest to explore the likelihood of transverse and longitudinal cracking, respectively.

![Graph showing longitudinal stress contours](image)

**Figure 5.7**  (a) Distribution of longitudinal normal stresses in the composite voided slab system due to differential shrinkage and shrinkage induced creep (b) Longitudinal normal stress contours in the composite voided slab system

Figure 5.8 shows transverse and longitudinal normal stress contours caused by differential shrinkage and shrinkage induced creep. In the voided slab system, longitudinal tensile stresses are highest at the precast-CIP interface (Figure 5.8(a)), which is consistent with the results presented in Figure 5.6 and Figure 5.7. Similarly, transverse tensile stresses are also highest at the interface between the precast and cast-in-place components. The same applies for the inverted T-beam systems, except that in these systems the interfaces between the precast and cast-in-place components are at two different elevations, with the interface between the precast web and cast-in-place topping being the most critical. Hence, reducing the length of this interface will help reduce the extent of potential cracking due to time dependent effects. Tensile stresses also develop in the cast-in-place topping at the interface between the precast flange and cast-in-place topping, however these stresses reduce towards the top of the composite cross-section and are compressive in the upper portions of the cast-in-place topping. Because it is the top of the cast-in-place topping that is subject to water and deicing salts these locations are not critical compared to the portion of the topping above the precast web where the entire cast-in-place section is in tension. Figure 5.8(c)
suggests that the inverted T-beam system with tapered webs features the narrowest cast-in-place topping precast web interface and therefore minimizes the extent of potential longitudinal and transverse cracking.

*Figure 5.8* Transverse (S11) and longitudinal (S33) normal stresses for three composite bridge systems due to differential shrinkage and shrinkage induced creep

Figure 5.9 shows the magnitude and distribution of longitudinal and transverse normal stresses along the depth of the three composite bridge cross-sections investigated. This distribution was obtained from the finite element models created for each bridge system. Transverse and longitudinal stress values were obtained from a column of finite elements located at mid-width of the composite cross-sections. Figure 5.9(a) shows that the magnitude of transverse normal stresses caused by time dependent effects in the three investigated bridge systems is similar. The voided slab system appears to be experiencing slightly smaller tensile stresses at the top of the cast-in-place topping due to the slightly more flexible restraint provided by the precast component as a result of the holes. However, this reduction is small compared to the advantage of a shorter critical interface offered by the inverted T-beam systems.

Figure 5.9(b) shows that the magnitude of longitudinal normal tensile stresses in the CIP topping is smaller in the precast inverted T-beam systems with tapered webs compared to the other two systems. The combination of smaller tensile stresses in the CIP topping and a shorter interface at the top of the precast web reduces not only the likelihood of transverse cracking but also the
extent of it. Table 5.9 shows a comparison of the vulnerability to cracking of the three bridge systems in terms of the ratio between the maximum tensile stress at the bottom of the cast-in-place concrete deck and the tensile strength of the deck. The tensile strength of the concrete deck was taken equal to the tensile strength of the concrete mix with normal weight coarse aggregates and a portion of saturated lightweight fine aggregates. Table 5.9 shows that the bridge system with precast inverted T-beams features the lowest average ratio between the maximum tensile stress at the bottom of the deck and the tensile strength of the deck. Tapering the precast webs helps reduce the extent of the critical interface between the top of the precast web and the cast-in-place deck in both the transverse and longitudinal directions. Any tensile stresses developed at the interface between the precast flange and cast-in-place topping, or those created between the tapered precast webs and cast-in-place topping are less likely to cause cracking in the top surface of the bridge deck.

The selection of the inverted T-beam system with tapered webs for short-to-medium-span bridges combined with the specification of a cast-in-place concrete topping mix that exhibits low shrinkage and high creep will offer a bridge system that accelerates construction and is less likely to exhibit cracking due to time dependent effects.

![Figure 5.9](image)

**Figure 5.9** Comparison of normal stresses in three composite bridge systems caused by differential shrinkage and shrinkage induced creep, (a) Transverse normal stresses, (b) Longitudinal normal stresses

<table>
<thead>
<tr>
<th>Bridge system</th>
<th>( f_{\text{max}} ) (ksi)</th>
<th>( f_{\text{tested}} ) (ksi)</th>
<th>Ratio = ( f_{\text{max}} / f_{\text{tested}} )</th>
</tr>
</thead>
<tbody>
<tr>
<td>Voided Slab</td>
<td>S11 0.240</td>
<td>0.340</td>
<td>0.71 Average</td>
</tr>
<tr>
<td></td>
<td>S33 0.236</td>
<td>0.340</td>
<td>0.69</td>
</tr>
<tr>
<td>Precast inverted T-beam with straight webs</td>
<td>S11 0.250</td>
<td>0.340</td>
<td>0.74 Average</td>
</tr>
<tr>
<td></td>
<td>S33 0.210</td>
<td>0.340</td>
<td>0.62</td>
</tr>
<tr>
<td>Precast inverted T-beam with tapered webs</td>
<td>S11 0.255</td>
<td>0.340</td>
<td>0.75 Average</td>
</tr>
<tr>
<td></td>
<td>S33 0.156</td>
<td>0.340</td>
<td>0.46</td>
</tr>
</tbody>
</table>

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5.8 Conclusions

Reducing the likelihood of bridge deck cracking is essential for increasing the longevity of bridges. This study has demonstrated that the effects of differential shrinkage and creep in composite bridges can cause tensile stresses in the deck that exceed its tensile strength. To reduce the likelihood of excessive cracking it is recommended that the concrete mix for the cast-in-place deck possesses low shrinkage and high creep properties. Low free shrinkage reduces the tensile stresses developed due to restrained differential shrinkage and high creep helps relax any tensile stresses that may develop. In addition, the short terms properties that will help reduce the extent of cracking due to differential shrinkage include a mix with high tensile strength and low modulus of elasticity. Because it is difficult to find a concrete mix that embodies all the aforementioned long term and short term properties, priority should be given the mix with the lowest shrinkage because it is the free shrinkage of the deck that serves as a catalyst for the creation of tensile stresses in the cast-in-place topping. In addition, sensitivity studies can be performed for a given structure to determine the influence of the short and long term properties of the concrete materials on the structural effects of shrinkage and creep. While a concrete mix with high creep is advocated for the cast-in-place deck to help alleviate tensile stresses due to differential shrinkage, long term deflections should be checked to ensure that they are within the allowable limits. Although the investigation described in this study was focused on short-to-medium-span composite concrete bridges, the benefits of a cast-in-place deck with low shrinkage and high creep apply also to composite steel bridges in which the free shrinkage of the deck is restrained by the steel girder.

Seven deck mixes developed by the researchers at VCTIR were investigated with the goal of identifying a mix with low shrinkage and high creep. Drying shrinkage strains recorded in the shrinkage prisms showed that the mix with normal weight coarse aggregates and saturated light weight fine aggregates exhibited the lowest shrinkage strain. In addition, data obtained from the creep test showed that the mix with normal weight coarse aggregates and saturated light weight fine aggregates also exhibited the highest creep coefficient. It should be noted that the tests performed on the deck mixes described in this investigation were performed only on one batch for each mix. It was outside the scope of study to investigate the repeatability and consistency of the results obtained from each mix.

Three composite bridge systems intended for short-to-medium-span bridges were investigated with the purpose of identifying the system that is most resistant against time dependent effects. The voided slab system is the traditional system and the inverted T-beam system with straight and tapered precast webs are relatively new bridge systems used in Minnesota and Virginia, respectively. The precast inverted T-beam system with tapered webs minimizes the width of the interface between the precast web and the cast-in-place topping, which is the most vulnerable interface to longitudinal and transverse cracking as a result of differential shrinkage. While the magnitude of transverse normal stresses was similar in all three bridge systems, the inverted T-beam with the tapered webs exhibited lower longitudinal tensile stresses at the interface between the precast web and cast-in-place topping, which is desirable for reducing the likelihood of transverse cracking.
The results presented in this investigation suggest that the combination of the precast inverted T-beam system with tapered webs and the implementation of a cast-in-place topping mix with normal weight coarse aggregates and saturated light weight fine aggregates should help increase the resistance of short-to-medium-span bridges against time dependent effects.

**Notation**

- $A_D$ = area of cast-in-place deck
- $A_G$ = area of precast girder
- $A_{ps}$ = area of prestressing strands
- $a$ = distance between the centroid of cast-in-place deck and centroid of precast girder.
- $a_D$ = distance between the centroid of the cast-in-place deck and centroid of composite section
- $a_G$ = distance between the centroid of the girder and centroid of the composite section
- $E_D$ = modulus of elasticity of the cast-in-place deck
- $E_G$ = modulus of elasticity of the precast girder
- $E_{ps}$ = modulus of elasticity of prestressing strands
- $e$ = eccentricity of the prestressing force with respect to the centroid of the precast girder
- $f'_c$ = specified compressive strength of concrete for use in design (ksi)
- $I_D$ = moment of inertia of the cast-in-place deck
- $I_G$ = moment of inertia of the precast girder
- $K_1$ = correction factor for source of aggregate
- $M_{D0}$ = initial moment in the deck due to forces applied to the composite system
- $M_{D\text{direct}0}$ = initial moment applied directly to the deck
- $M_{G0}$ = initial moment in the girder due to forces applied to the composite system
- $M_{G\text{direct}0}$ = initial moment applied directly to the girder
- $N_{D0}$ = initial axial force in the deck due to forces applied to the composite system
- $N_{D\text{direct}0}$ = initial axial force applied directly to the deck
- $N_{G0}$ = initial axial force in girder due to forces applied to the composite system
- $N_{G\text{direct}0}$ = initial axial force applied directly to the girder
- $N_{ps0}$ = initial prestressing force in the strand due to forces applied to the composite system
- $N_{ps\text{direct}0}$ = initial prestressing force in the strand
- $\Delta \varepsilon_D$ = change in strain at the centroid of deck due to time dependent effects
- $\Delta \varepsilon_G$ = change in strain at the centroid of girder due to time dependent effects
- $\Delta \varepsilon_{ps}$ = change in strain in the prestressing strand due to time dependent effects
- $\Delta X$ = change in curvature due to time dependent effects
- $\Delta N_D$ = change in axial force in the deck due to time dependent effects
- $\Delta N_G$ = change in axial force in the girder due to time dependent effects
- $\Delta N_{ps}$ = change in prestressing force due to time dependent effects
- $\Delta M_D$ = change in moment in the deck due to time dependent effects
- $\Delta M_G$ = change in moment in the girder due to time dependent effects
- $w_c$ = unit weight of concrete (kcf)
References


13. ACI Committee 209 (2008), “Guide for modeling and calculating shrinkage and creep in hardened concrete”, American Concrete Institute, Farmington Hills, MI.

Chapter 6: Investigation of Stresses in the End Zones of Precast Inverted T-Beams with Tapered Webs

(Submitted to 2014 PCI Convention and National Bridge Conference)
Investigation of Stresses in the End Zones of Precast Inverted T-beams with Tapered Webs

Abstract

Short-to-medium-span composite bridges constructed with adjacent precast inverted T-beams and cast-in-place topping are intended to provide a higher degree of resistance against reflective cracking and time dependent effects compared to voided slab and adjacent box girder systems. This paper investigates the stresses in the end zones of such a uniquely shaped precast element. The transfer of prestressing force creates vertical and horizontal tensile stresses in the end zones of the girder. A series of 3-D finite element analyses were performed to investigate the magnitude of these tensile stresses. Various methods of modeling the prestressing force including the modeling of the transfer length are examined and the effect of notches at the ends of the precast beams is explored. Existing design methods are evaluated and strut-and-tie models, calibrated to match the results of 3-D finite element analyses are proposed as alternatives to existing methods to aid designers in sizing reinforcing in the end zones. It is shown that the magnitude of tensile stresses in the pre-tensioned anchorage zones depends on the eccentricity of the prestressing force. Recommendations for how to apply existing provisions and recommendations to such a uniquely shaped precast member are presented.

6.1 Introduction

End regions of prestressed members are subject to high concentrated loads during the transfer of the prestressing force. Accordingly, the state of stress in these regions is complicated and cannot be predicted by the Euler-Bernoulli beam theory, in which plane sections are assumed to remain plane. According to Saint Venant’s principle\(^1\), the disturbance caused by the concentrated forces at the ends of the member diminishes after a distance \(h\) from the end of the member, where \(h\) is the overall depth of the member. In pre-tensioned concrete members, the transfer of the prestressing force into the surrounding concrete creates tensile stresses in the end zones. These stresses are characterized as spalling, splitting and bursting stresses. Spalling stresses are vertical tensile stresses that occur near the end face at the centroid of the member. Splitting stresses are circumferential tensile stresses that occur near the end face at the centroid of the member. Bursting stresses are vertical tensile stresses that occur along the line of the prestressing force, beginning a few inches into the member and extending through the transfer length. When these tensile stresses exceed the modulus of rupture of concrete, cracks form, which may compromise the shear and flexural strength of the member near that region as well as its durability.

AASHTO LRFD Specifications\(^2\) require that reinforcing be provided in pre-tensioned anchorage zones to resist 4% of the total prestressing force. The Specifications also require that
this reinforcing be placed within a distance that is equal to \( h/4 \) from the end of the beam, where \( h \) is the overall dimension of the precast member in the direction in which “splitting” resistance is evaluated. These provisions, incorrectly labeled as splitting provisions, are intended to resist spalling forces. The value of \( h \) and the direction in which the reinforcing required to resist the spalling forces is oriented, depends on the shape of the member. For example, for pre-tensioned I-girders or bulb tees, \( h \) represents the overall depth of the member and the end zone reinforcing is placed vertically within a distance equal to \( h/4 \) from the end of the member. For pre-tensioned solid or voided slabs, \( h \) represents the overall width of the section and the end zone reinforcing is placed horizontally within \( h/4 \). For pre-tensioned box or tub girders with prestressing strands located in both the bottom flange and the webs, end zone reinforcing is placed both horizontally and vertically within \( h/4 \), where “\( h \)” is the lesser of the overall width or height of the member. Although not specifically addressed in AASHTO\(^2\), the confinement requirements of AASHTO\(^2\) 5.10.10.2 should help control the bursting and splitting stresses that develop in the transfer length region (French et al.\(^3\)). It should be noted that the Specifications\(^2\) require that end zone reinforcing be provided in the vertical plane, horizontal plane or both planes regardless of the geometry of the pre-tensioned member, the strand pattern or the eccentricity in the plane under consideration.

The research presented in this paper investigates stresses in the end zones of precast inverted T-beams with tapered webs. This unique precast shape is intended for the construction of short-to-medium-span bridges. The inverted T-beam bridge system provides an accelerated bridge construction alternative and consists of adjacent precast inverted T-beams finished with a cast-in-place concrete topping. The adjacent precast inverted T-beams serve as stay-in-place formwork for the cast-in-place concrete topping and eliminate the need for site-installed formwork. This bridge system is intended to address reflective cracking problems present in composite bridges built with the traditional adjacent voided slab or adjacent box beam systems. The tapered precast webs help emulate monolithic construction by providing enhanced resistance against transverse tensile stresses induced because of transverse bending\(^4\). In addition, the tapered precast webs increase the resistance of the bridge system against longitudinal and transverse cracking caused by differential shrinkage\(^5\). Virginia Department of Transportation is implementing the system for the first time in a bridge replacement project near Richmond, VA.

Because the inverted T-beam system featuring adjacent precast inverted T-beams with tapered webs and cast-in-place topping is a new bridge system, there is a need to evaluate the applicability of the current Specification\(^2\) provisions for pre-tensioned anchorage zones. Figure 6.1(a) shows the elevation of the first application of the inverted T-beam system in the US 360 Bridge over the Chickahominy River and Figure 6.1(b) shows the transverse cross-section of the bridge. The US 360 Bridge is a two-span continuous bridge. The design span for the precast inverted T-beams is 41.5 feet. The design concrete compressive strength at transfer is \( f'_{ci} = 5 \) ksi. Figure 6.2(a) shows an isometric view of the end of the precast beam featuring recessed precast flanges at bearing locations to avoid high flexural stresses at the precast web-flange intersection. The recession of precast flanges allows the precast web to resist the reaction at the support and
prevents the transverse bending of a 4 in. flange, which would take place if the flanges are not recessed. The length of precast flange recession is 12 in. Three 6 in. by 9 in. by ½ in. elastomeric bearing pads (70 durometer hardness) were provided at the ends of each precast inverted T-beam and were located within the width of the precast web. The rest of the bearing area was covered with ½ in. preformed asphalt joint filler.

Figure 6.2(b) and 2(c) show the end zone reinforcing at Sections 1 and 2, respectively. End zone mild steel reinforcing consists of AASHTO\textsuperscript{2} required confinement steel, and features No.4 stirrups. The first four rows of confinement steel are placed at 3 in. on center with the first row at 2 in. from the end face. The rest of the confinement steel is placed at 6 in. on center. In addition, four legs of No.4 extended stirrups are provided at the same spacing as the confinement steel. Beyond a distance equal to 1.5\(d\), where \(d\) is the effective depth of the member, the spacing of closed and extended stirrups is 12 in. Past the flange cuts, horizontal transverse steel consisting of No.4 at 8 in. on center is provided to resist the wet weight of cast-in-place concrete topping and transverse bending moments due to live loads. All prestressing steel is concentrated within the footprint of the precast web. The bottom two layers of prestressing consist of 24 0.6 in. diameter strands (twelve strands in each layer). The top layer consists of two 0.6 in. diameter strands. The jacking force for each Grade 270 strand was 44 kips. The eccentricity of the strand group is 2.99 in. In addition to the 26 fully stressed strands described above, four additional strands stressed only to 1 kip were provided between the two fully stressed top strands to facilitate the placement of extended stirrups. Longitudinal normal stresses during transfer were kept below AASHTO\textsuperscript{2} allowable stresses without the need to resort to strand debonding.

![Figure 6.1 (a) Elevation of US 360 Bridge, (b) Transverse cross-section of US 360 Bridge](image-url)
Because of the unique shape of the cross-section of the precast beam, the diffusion of the prestressing force will occur in both the vertical and horizontal planes. The purpose of this paper is to quantify normal tensile stresses at the end zones in both planes and determine whether these stresses are high enough to cause cracking. A series of 3-D finite element analyses were performed to investigate the magnitude of these tensile stresses. Various methods of modeling the prestressing force including the modeling of the transfer length are examined and the effect of notches at the end of the precast beams is explored. Existing design methods are evaluated and strut-and-tie models, calibrated to match the results of 3-D finite element analyses, are proposed as alternatives to existing methods to aid engineers in sizing reinforcing in the end zones.

6. 2 Related Studies

Gergerly et al.\textsuperscript{6} state that the horizontal cracks that frequently form in the end region of prestressed concrete members when the prestressing strand is released and the prestressing force is transferred to the concrete section are defined as “spalling” cracks, though often incorrectly labeled as “bursting” or “splitting” cracks. If unrestrained, these cracks can extend into the precast member and negatively affect the flexural and shear strength and durability of the member. Studies performed by Fountain\textsuperscript{7} suggest that these cracks cannot be eliminated, however vertically oriented reinforcing steel can limit crack width and propagation.

Gergerly et al.\textsuperscript{6} showed that the distribution of the tensile stresses in the end region depends on the eccentricity of the prestressing force in the member. For example, in a concentrically loaded member forces distribute symmetrically through the vertical member height until a uniform stress distribution is established at a distance $h$ from the end of the member (Saint Venant’s principle\textsuperscript{1}). In such a member, the spalling forces developed at the end face are smaller than the bursting forces that develop at a distance $h/2$ from the end of the member (Figure 6.3 (a)). Conversely, in an eccentrically loaded member the spalling forces developed near the end face are higher than the bursting forces developed a certain distance away from the end of the member (Figure 6.3 (b)). Hawkins\textsuperscript{8} corroborated Gergerly’s\textsuperscript{6} findings and found that as eccentricity increased so did the magnitude of maximum tensile stress in the spalling zone.

Eriksson\textsuperscript{9} performed an evaluation of the stresses in the end zones of precast inverted T-beams with straight webs to determine the applicability of the AASHTO provisions\textsuperscript{2} on pre-tensioned anchorage zones. Because the overall depth of precast inverted T-beams is relatively shallow compared to I-girders, the requirement to place the vertical steel in the end zone within a distance equal to $h/4$ from the end of the member results in congestion problems. However, as stated earlier, the placement of vertical steel in the end zones of wide and shallow members (solid or voided slabs) is relaxed by allowing the designer to spread this steel within a distance $h/4$ where $h$ is the width of the member rather than its depth. According to French et al.\textsuperscript{3} such a relaxation may not be appropriate when trying to control spalling stresses, because in eccentrically loaded members, the magnitude of spalling stresses diminishes quickly away from the end of the member.
The evaluation that Eriksson\textsuperscript{9} and French et al.\textsuperscript{3} performed included experimental and numerical studies. The experimental study was performed on laboratory bridge specimens, constructed with precast inverted T-beams, which featured various configurations of end zone reinforcing (Table 6.1). The experimental results revealed that the 12 in. deep precast sections had sufficient strength to resist the tensile stresses created in the end zone even in cases where no vertical steel was present. These findings were corroborated with the results of numerical studies that showed certain inverted-T members did not require spalling reinforcement, specifically.
those members with depths less than 22 in. for which the expected concrete strength was higher than the expected tensile stresses due to the development of prestress (French et al.\textsuperscript{3}).

In contrast, for deep inverted T-beams, it was numerically determined that larger amounts of spalling reinforcement than specified by AASHTO’s provisions\textsuperscript{2} for splitting resistance is required. It was also concluded that the reinforcement should be placed as close to the end of the beam as possible (i.e., within $h/4$ of the end of the member, where $h$ represents the depth of the member). For the numerical study, finite element modeling was used to determine the magnitude and location of spalling and bursting stresses by employing several simplifications to reduce the complexity and computational requirements of the model. The flanges were neglected to allow the system to be modeled as a two-dimensional rectangular slab. As a result, spalling and bursting stresses were only investigated in the vertical plane.

**Table 6.1 Vertical reinforcement in configurations 1-4 of the precast members utilized in experimental study (French et al.\textsuperscript{3})**

<table>
<thead>
<tr>
<th>Configuration</th>
<th>Description of Vertical End Zone Reinforcement</th>
<th>Cross Section View of Stirrup</th>
<th>Elevation View of Reinforcement Spacing</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>#3 stirrup at 2 and 4 in. total area = 0.44 in$^2$</td>
<td>![Cross Section View of Stirrup]</td>
<td>2 spaces @ 2 in.</td>
</tr>
<tr>
<td>2</td>
<td>#4 stirrup at 2 in. total area = 0.40 in$^3$</td>
<td>![Cross Section View of Stirrup]</td>
<td>1 space @ 2 in.</td>
</tr>
<tr>
<td>3</td>
<td>#5 four legged stirrup at 2 and 4 in. total area = 2.5 in$^2$</td>
<td>![Cross Section View of Stirrup]</td>
<td>2 spaces @ 2 in.</td>
</tr>
<tr>
<td>4</td>
<td>#5 stirrup at 2 and 4 in. total area = 1.2 in$^2$</td>
<td>![Cross Section View of Stirrup]</td>
<td>2 spaces @ 2 in.</td>
</tr>
</tbody>
</table>

Some of the suggested modifications to AASHTO\textsuperscript{2} Article 5.10.10.1 that resulted from this study are presented below:

- For all sections other than rectangular slabs and shallow inverted-T sections with heights less than 22 in, the spalling resistance of pretensioned anchorage zones provided by reinforcement in the ends of pretensioned beams shall be taken as:

  $$P_r = f_s A_s$$

  \textit{where:}

  $$f_s = \text{stress in steel not to exceed 20 ksi}$$

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\[ A_s = \text{total area of reinforcement located within the distance } h/4 \text{ from the end of the beam (in.}^2) \]

\[ h = \text{overall dimension of precast member in the direction in which spalling resistance is being evaluated (in.)} \]

The resistance shall not be less than four percent of the total prestressing force at transfer.

- In pretensioned anchorage zones of rectangular slabs and shallow inverted-T sections with heights less than 22 in., vertical reinforcement in the end zones is not required if:

\[ \sigma_s < f_r \quad (2) \]

where:

\[ \sigma_s = \frac{P}{A} \left( 0.1206 \frac{e^2}{h \ d_b} - 0.0256 \right) \geq 0 \quad (3) \]

\[ f_r = 0.23 \sqrt{f_{ct}'} \quad (4) \]

\[ \sigma_s = \text{maximum spalling stress on the end face (ksi)} \]

\[ f_r = \text{direct tensile strength as defined by Article C5.4.2.7 (ksi)} \]

\[ P = \text{prestressing force at transfer (kips)} \]

\[ A = \text{gross cross-sectional area of concrete (in}^2) \]

\[ e = \text{strand eccentricity (in.)} \]

\[ h = \text{overall depth of precast member (in.)} \]

\[ d_b = \text{prestressing strand diameter (in.)} \]

\[ f_{ct}' = \text{concrete compressive strength at transfer (ksi)} \]

Where end zone vertical reinforcement is required, it shall be located within the horizontal distance \( h/4 \) from the end of the beam, and shall be determined as:

\[ A_s = \frac{P \ (0.02 \frac{e^2}{h \ d_b} - 0.01)}{f_s} \quad (5) \]

The resistance shall not be less than four percent of the total prestressing force at transfer. In all cases, the reinforcement shall be as close to the end of the beam as practicable. Reinforcement used to satisfy this requirement can also be used to satisfy other design requirements.

In the suggested modifications presented above, the modulus of rupture is taken equal to 0.23 \( \sqrt{f_{ct}'} \). The commentary of Article C5.4.2.6 in AASHTO\(^2\) states that: “Most modulus of rupture test data on normal weight concrete is between 0.24\( \sqrt{f_{ct}'} \) and 0.37\( \sqrt{f_{ct}'} \) ……” The given
values may be unconservative for tensile cracking caused by restrained shrinkage, anchor zone splitting, and other tensile forces caused by effects other than flexure. The direct tensile strength stress should be used for these cases.” In addition, the commentary of Article C5.4.2.7 in AASHTO states:” For normal weight concrete with specified compressive strengths up to 10 ksi, the direct tensile strength may be estimated as $f'_{t} = 0.23\sqrt{f'_{c'}}$. Accordingly, the estimation of the tensile strength based on $0.23 \sqrt{f'_{c'}}$ to determine the likelihood of cracking at the end zones because of the diffusion of the prestressing force is consistent with AASHTO’s commentary.

As stated earlier, because the precast inverted T-beam with tapered webs features a unique shape, there was a need to evaluate the applicability of the current provisions given in the AASHTO LRFD Specifications, as well as the recommendations made by Erisksson and French et al. for the vertical plane.

The numerical study performed by Eriksson was based on 2D finite element models using shell elements and by modeling only the portion of the precast web. The presence of precast flanges was ignored to make possible such an idealization in 2D. In this study, the precast beams are modeled as 3D components using 3D continuum elements for concrete and 3D embedded truss elements for prestressing strands. As a result, tensile stresses in the end zones are investigated in the vertical plane as well as in the horizontal plane. Such 3D modeling was essential for the precast inverted T-beams with the tapered webs, because, in this case a 2D idealization would not be justified.

### 6.3 Investigation Using Finite Element Analysis

The precast inverted T-beam section used in the construction of the US 360 Bridge was modeled using 3D continuum elements using the commercially available finite element software Abaqus. Initially, stresses and deflections due to the self-weight of the member were computed using a 2 in. mesh with the purpose of comparing them with those calculated using the Euler-Bernoulli beam theory. Figure 6.4 shows the longitudinal normal stress contours due to the self-weight of the member and Figure 6.5 shows vertical displacement contours. Table 6.2 shows a comparison between stresses and deflections computed using finite element analysis and those based on “hand calculations” using the Euler-Bernoulli beam theory. This comparison was carried out for the top and bottom fibers at mid-span of the beam. The difference in the results is very small, which demonstrates that a 2 in. mesh can properly capture the effects of the self-weight of the member. Mid-span deflections were identical whereas the small differences in top and bottom stresses can be attributed to the 3D state of stress in the finite element model compared to the 1D stress state employed in the beam line theory used in “hand calculations”.

### 6.4 US 360 Bridge Girder (41.5 Foot Long)

The implementation of the inverted T-beam system in the US 360 Bridge provided a good opportunity to observe the performance of a unique precast shape immediately after
prestress transfer. The modulus of elasticity for the precast beam at transfer was calculated based on the formula provided in Article 5.4.2.4 of AASHTO LRFD Specifications as a function of the design compressive strength at transfer and was 4287 ksi. Poisson’s ratio was used as 0.2 (based on Article 5.4.2.5 of AASHTO LRFD Specifications). Linear elastic finite element analyses, which are appropriate up to the initiation of cracking, were performed to investigate normal stresses at the end zones in the vertical and horizontal planes. Various methods of modeling the prestressing force were considered with the purpose of identifying the most accurate modeling technique. In all the modeling techniques presented in the following sections, only the effect of the fully stressed 26 strands was considered. The effect of the four additional top strands used for constructability and stressed only to 1 kip was considered negligible.

6.4.1 Vertical Plane - Case 1

The prestressing force in Case 1 was modeled as a series of concentrated loads at the ends of the precast beam simulating a condition similar to a post-tensioned beam (Figure 6.6). As stated earlier, concrete in the precast beam was modeled using 3D continuum elements. The advantage of this modeling technique is simplicity. The strands are not modeled and the entire prestressing force is assumed to be applied at the ends of the precast beam. This modeling technique does not take into consideration the transfer length for the prestressing force. The magnitude of the prestressing force in each strand was taken as the jacking force. The magnitude of normal longitudinal stresses away from the end zones was similar to that calculated using “hand calculations” based on the principles of linear elastic mechanics of materials. However, in the end zones the magnitude of spalling stresses created because of the application of the prestressing force was unrealistically high. This was because the concentrated loads representing the force in the strands were applied entirely at the nodes of the elements at the end faces of the precast beam. These concentrated forces created high stress concentrations in the vicinity where they were applied as well as along the depth the precast beam at the ends. A distribution of normal stresses along the depth of the precast beams is shown in Figure 6.7 (a). Figure 6.7 (b) also shows a longitudinal cut and illustrates how the magnitude of the spalling stresses diminishes away from the ends of the precast beam. The maximum tensile stress estimated at the nodes of the elements along the depth of the precast beam was 2.44 ksi, which is much higher than the modulus of rupture of the precast beam when the strands were de-tensioned. The modulus of rupture was taken equal to 0.23\(\sqrt{f'c}\), where \(f'c\) is in ksi. For a design compressive strength at transfer equal to \(f'ct = 5\) ksi the modulus of rupture is approximately 0.51 ksi. Because a visual inspection of the 37 precast beams used in the construction of the US 360 Bridge (36 production beams + 1 trial), showed no signs of cracking at the end zones, such a modeling technique was deemed unrealistically conservative for designing the pre-tensioned anchorage zones. This conclusion is corroborated by previous studies, which report that tensile stresses in the end zone are affected by the transfer length (Base). In addition, Uijl\(^1\)\(^2\) concludes that longer transfer lengths in pre-tensioned systems result in smaller bursting and spalling stresses. Shorter
transfer lengths concentrate the transfer of forces, which result in larger bursting and spalling stresses, more similar to the case of post-tensioned systems (Uijl9). Many theories developed from post-tensioned experiments can provide conservative estimates of the spalling and bursting stresses in pre-tensioned members, because they simulate the case of a very short transfer length (French et al.3).

6.4.2 Vertical Plane - Case 2

In this case, the prestressing strands were modeled as embedded truss elements in perfect bond with 3D continuum elements used for concrete. The prestressing force in the strands was modeled as an initial condition, which simulates the tensile stress in the pre-tensioned strands. This modeling capability is available in Abaqus10. A uniform tensile stress was applied along the length of the strands and the cross-sectional area of the strands was kept constant along the span of the precast beam. This modeling technique while more realistic than the previous one, still does not take into consideration the transfer length because it assumes that the prestressing force is constant along the length of the precast beam starting at the face of the beam. Figure 6.8(a) shows the normal stress contours along the depth of the precast beam. Figure 6.8(b) shows a longitudinal cut highlighting how the magnitude of the vertical tensile stresses diminishes away from the end of the precast beam highlighting once again that spalling stresses are the dominating type of tensile stresses at the end zones. The maximum spalling stress in this case is approximately 2.0 ksi, which is lower compared to the previous case but still unrealistic because no cracking was observed during the visual inspection of the 37 precast beams.

6.4.3 Vertical Plane - Case 3

The modeling technique utilized in this case is similar to that used in Case 2 with the exception that the transfer length was modeled by incrementally varying the cross-sectional area of the prestressing strands along the transfer length. The transfer length was taken equal to 60 strand diameters as given in Article 5.11.4.1 of AASHTO LRFD Specifications2. By keeping the magnitude of the prestress constant and by incrementally varying the cross-sectional area of the strands within the transfer length the amount of prestressing force transferred to the surrounding concrete varies linearly within the transfer length. This modeling technique is more realistic compared to the previous two techniques. The computed maximum vertical tensile stress between the top and bottom layers of strands is approximately equal to 0.4 ksi. This is smaller than the modulus of rupture (0.51 ksi) for the precast beam when the strands were de-tensioned and corroborates the fact that no cracks were observed during the visual inspection of the 37 precast beams.
Figure 6.4 Longitudinal normal stress due to self-weight

Figure 6.5 Deflection due to self-weight
Table 6.2 Comparison of stress and deflections due to self-weight

<table>
<thead>
<tr>
<th>Max. longitudinal stress (ksi)</th>
<th>FEA*</th>
<th>Euler-Bernoulli</th>
<th>% Difference</th>
</tr>
</thead>
<tbody>
<tr>
<td>Mid-span - Top</td>
<td>1.16</td>
<td>1.17</td>
<td>0.9</td>
</tr>
<tr>
<td>Mid-span - Bottom</td>
<td>0.72</td>
<td>0.74</td>
<td>3.0</td>
</tr>
<tr>
<td>Deflection (in.)</td>
<td>Mid-span</td>
<td>0.64</td>
<td>0.64</td>
</tr>
</tbody>
</table>

*FEA = Finite Element Analysis

Figure 6.6 Prestressing applied as point loads at the ends

Figure 6.7 Normal stress contours along the depth of the precast beam – Case 1 (a) full beam, (b) longitudinal cut
Figure 6.8 Normal stress contours along the depth of the precast beam – Case 2, (a) full beam, (b) longitudinal cut

Figure 6.9 (a) and (b) show the vertical normal stress contours at the ends of the precast beam and a longitudinal cut at mid-width of the beam. The predominance of spalling stresses in precast beams in which the prestressing force is applied eccentrically towards the bottom of the beam, occurs because there is a greater concrete area above the prestressing force through which the stresses distribute. This allows the prestressing force to spread over a larger vertical distance, making the curvature of the flow of stresses greater, creating a larger spalling force near the end region (Figure 3(b))(French et. al.3). Hawkins8 and Gergerly6 corroborate this phenomenon and report that as eccentricity increases so does the magnitude of the maximum tensile stress in the spalling zone.

There are two isolated locations at the bottom corners of the precast beam where the tensile stress is around 0.9 ksi, however this higher concentration of stress is isolated only at the corner node of the corresponding element and diminishes quickly. These isolated higher concentrations of tensile stress at the bottom corners of the precast beam are believed to be a result of stress concentrations at these corners. Because the visual inspection of the 37 precast beams did not show any signs of cracking at these areas, these isolated stress concentrations are not believed to be detrimental to the structural integrity of the precast beam and its performance. In addition, the provision of AASHTO2 required confinement steel should help control the width of any potential cracks at these locations.
6.4.4 Vertical Plane - Case 4

The flanges of the precast beam were cut by approximately one foot at the ends to avoid high flexural stresses at the intersection of the precast flange and web at the bearing points. A finite element model without this cut was created to determine whether the presence of the cut has an adverse effect on the stresses at the end zones. Figure 6.10 shows the normal stress contours along the depth of the precast beam. With the flange cut eliminated the stress concentration at the bottom of the intersection between the precast flange and the precast web still exists. The magnitude of vertical tensile tresses at this location is approximately 1.34 ksi, which is higher compared to Case 3. As a result, cutting the precast flanges at the end zones reduces the vulnerability of cracking at the intersection between the precast flange and the precast web.

Figure 6.9 Normal stress contours along the depth of the precast beam – Case 3, (a) full beam, (b) longitudinal cut
6.4.5 Horizontal Plane

The diffusion of the prestressing force was also investigated in the horizontal plane. Because the prestressing force introduced at the top layer consisted of only two 0.6 in. diameter strands and because these strands were located near the top corners of the precast web, there was limited space for the prestressing force to diffuse. Accordingly, normal tensile stresses in the horizontal plane at the top portion of the beam were negligible. However, the distribution of the prestressing force introduced at the bottom two layers (24 0.6 in. diameter strands) caused normal tensile stresses in the horizontal plane that were higher in magnitude. This is because the strands at these two layers were located within the footprint of the precast web and the prestressing force at this location could diffuse horizontally outwards towards the precast flanges. In addition, the magnitude of the prestressing force at the bottom two layers was the majority of the prestressing force introduced in the entire section. Nonetheless, the maximum normal tensile stress in the horizontal plane towards the bottom of the precast beam was only approximately 0.2 ksi, which is lower than the modulus of rupture at transfer (0.51 ksi). As a result, tensile stresses created because of the diffusion of the prestressing force in the horizontal plane were lower than the ones created in the vertical plane. Figure 6.11 shows horizontal normal stress contours towards the bottom of the precast beam. It can be seen that the distribution of these normal tensile stresses is fairly uniform past 12 to18 inches from the end of the beam. Because the prestressing force at the bottom two layers was symmetric about the vertical axis, there was no eccentricity in the horizontal plane. Accordingly, tensile stresses created because of
the diffusion of the prestressing force in the horizontal plane were predominantly bursting stresses.

![Bursting stresses](image)

**Figure 6.11** Normal stress contours in the horizontal plane

### 6.5 Other Cases

Because the precast inverted T-beam bridge system can be used for short-to-medium-span bridges with spans ranging from 20 feet to approximately 60 feet, two additional cases that represent the extreme spans in this range were investigated.

#### 6.5.1 20 Foot Long Precast Beam

A composite bridge featuring 20-foot long spans was designed based on AASHTO LRFD Specifications with the purpose of determining the number of prestressing strands required to resist the effects of the design loads. The cross-sectional dimensions for the precast and cast-in-place components, as well as the number and position of prestressing strands are shown in Figure 6.12. Material properties for the precast beam, cast-in-place concrete and prestressing strands were the same ones used for the US 360 Bridge. The prestressing force was modeled as described in Case 3 for the 41.5 foot span because that was determined to be the most accurate modeling technique. The eccentricity of the prestressing force is 1.47 in.

The magnitude of the vertical normal tensile stresses at the end zones was negligible with the exception of two isolated locations at the bottom corners of the precast web where the tensile stress was 1.3 ksi. However, as discussed previously for the precast beams used in the US 360 Bridge, these higher tensile stresses isolated only at the bottom corners of the precast web are not considered detrimental to the structural integrity and serviceability of the precast beam. In the horizontal plane, the maximum tensile stress was equal to approximately 0.21 ksi, which is still lower than the modulus of rupture of the precast beam at transfer (0.51 ksi). The creation of bursting stresses in the horizontal plane in the case of precast inverted T-beams with tapered
webs is due to the diffusion of the prestressing force towards the flanges of the precast beam. This confirms the approach presented in AASHTO LRFD Specifications\(^2\), which suggests that for pretensioned solid or voided slabs end zone reinforcing should be placed in the horizontal plane. However, for rectangular solid or voided slabs, in which the strand layout is uniform along the width of the section, the diffusion of the prestressing force in the horizontal plane will not be applicable. The negligible magnitude of spalling stresses in the vertical plane also confirms the findings from previous research that the magnitude of spalling stresses is directly proportional to the eccentricity of the prestressing force.

**Figure 6.12** Typical composite bridge cross-section for a 20-foot long span (mild reinforcing not shown).

![](image)

**Figure 6.13** Normal stress contours (a) vertical plane (b) horizontal plane

6.5.2 60 Foot Long Precast Beam

A composite bridge featuring a 60-foot long span was designed based on AASHTO LRFD Specifications\(^2\) to represent a long span for the inverted T-beam system. The cross-sectional dimensions for the precast beam and the cast-in-place topping are shown in Figure 6.14. The eccentricity of the prestressing force is 3.94 in. The material properties for the precast
beam, cast-in-place topping and prestressing strands were identical to the ones used for the US 360 Bridge. In this case, the magnitude of spalling stresses near the end of the beam exceeded the modulus of rupture of the precast beam at transfer (0.51 ksi). The maximum tensile stress in the vertical direction was 0.83 ksi. Consequently, spalling stresses at the end zones of precast beams used for similar spans present a potential for cracking at the end zones. The magnitude of bursting stresses in the horizontal plane was lower than the modulus of rupture of the precast beam at transfer with the maximum tensile stress equal to 0.27 ksi. Accordingly, bursting stresses in the horizontal plane did not present a potential for cracking in the end zones.

![Diagram](image)

**Figure 6.14** Typical composite bridge cross-section for a 60-foot long span (mild reinforcing not shown)

![Graphs](image)

**Figure 6.15** Normal stress contours (a) vertical plane (b) horizontal plane
6.6 Evaluation of Existing Design Methodologies

6.6.1 AASHTO LFRD Specifications

Because the shape of the precast inverted T-beams with tapered webs is unique, engineering judgment will be used in implementing the AASHTO provisions for the pre-tensioned anchorage zones. The following questions need to be addressed:

1) Should the end zone reinforcing be provided in the vertical plane, horizontal plane or both?
2) Where should the end zone reinforcing be located?

AASHTO LRFD Specifications require end zone reinforcing in pre-tensioned anchorage zones, regardless of the span length, strand pattern, geometry of the precast member, eccentricity or magnitude of the prestressing force. Following is a comparison of end zone reinforcement designed based on the present AASHTO provisions, the finite element model results previously discussed, and the recommendations of a recently completed NCHRP project. The three span lengths previously discussed will be evaluated.

6.6.1.1 41.5 foot span

The total prestressing force for the 18 in. deep precast beam used in the 41.5 foot span US 360 bridge is 1144 kips. 4% of this force equals 45.76 kips. If an allowable steel stress of 20 ksi is used, then the required area of vertical steel in the end zones is 2.29 in\(^2\). In addition, according to AASHTO provisions, this amount of steel is distributed over a distance of \(h/4\) from the end of the member. The area of vertical end zone reinforcing provided in the first row in the precast beams used in the US 360 Bridge is 1.08 in\(^2\) (four legs of No.4 extended stirrups and the vertical component of the two inclined legs of the No.4 confinement stirrups (Figure 2)). In addition, the first row of vertical steel is located at 2 in. from the end of the precast beam. The second row of vertical steel provides the same area of steel and is located at 5 in. from the end of the beam, which is past the prescribed \(h/4\) distance. The total area of vertical steel provided in the first two rows is 2.16 in\(^2\), which is smaller than the AASHTO required 2.29 in\(^2\). However, because the results of finite element analyses indicated that spalling stresses in the vertical plane were smaller than the modulus of rupture of the precast beam at transfer, using a slightly smaller area was deemed acceptable. In addition, to comply with the AASHTO placement requirement the position of the second row can be changed to 4 in. from the end of the member rather than 5 in. (Figure 6.16(b)).

Bursting stresses in the horizontal plane were approximately half of the spalling stresses in the vertical plane (0.2 ksi versus 0.4 ksi). Accordingly, it would be conservative to apply the 4% rule for sizing reinforcing in the horizontal plane. In addition, because the distribution of bursting stresses was relatively uniform within the disturbed region \(h\), horizontal reinforcing can be distributed throughout a distance \(h\) from the end of the precast flange rather than \(h/4\). For the US 360 bridge, the 2.29 in\(^2\) of horizontal reinforcing determined using the 4% rule can be...
distributed over a distance of 6 feet past the precast flange. This leads to approximately 0.38 in²/ft. The closed stirrups in the US 360 Bridge consisted of No.4 at 6 in. on center, for up to 1.5d from the end of the precast member (confinement steel) and No.4 at 12 in. on center for the rest of the span. In addition, No.4 at 8 in. on center transverse straight reinforcing steel was provided in the precast flanges. Accordingly, as a minimum, the provided amount of horizontal steel at the end zones was equal to 0.5 in²/ft (Figure 6.16(b)).

In summary, it is conservative to size the vertical and horizontal steel at the end zones based on the 4% rule stipulated in AASHTO². The distribution of such reinforcing should be such that the vertical steel is located within a distance equal to \( h/4 \), where \( h \) is the depth of the member, and the horizontal steel is located within a distance equal to \( h \) from the end of the precast flange, where \( h \) is the width of the section.

6.6.1.2 20 foot span

Similar to the 41.5 foot span, spalling and bursting stresses for the 20-foot span were lower than the modulus of rupture of the precast beam at transfer. Accordingly, end zone reinforcing is not required and the implementation of AASHTO provisions² for pre-tensioned anchorage zones in the vertical and horizontal planes would be conservative. The total prestressing force for the 8 in. deep precast beam is 434 kips. 4% of this force equals 17.36 kips. If an allowable steel stress of 20 ksi is used, then the required area of the steel in the end zones is 0.87 in². The vertical steel can be provided in one row of No.4 confinement steel and four legs of No.4 extended stirrups. The horizontal steel can be provided by the horizontal leg of the No.4 confinement reinforcing at 6 in. on center (Figure 6.16(a)).

6.6.1.3 60 foot span

Because spalling stresses exceeded the modulus of rupture for the precast beam at transfer, vertical reinforcing at the end zones is required to control the widths of potential cracks. The vertical tensile force at the end zone can be calculated from the tension stress in the finite elements in the end zone. The tension stress above the modulus of rupture multiplied by the area of the elements is equal to 28.5 kips, whereas the force based on the 4 % rule is equal to 78.72 kips. Therefore, the amount of vertical steel can be conservatively calculated based on AASHTO² provisions. The required area of vertical reinforcing in the end zones based on AASHTO² provisions in this case is 3.94 in². This area of reinforcing can be provided by placing three rows of No.4 confinement steel and 4-legs of No.5 extended stirrups at 2 in. on center. The total area of provided vertical steel in this case will be 4.57 in² compared to the required 3.94 in² (Figure 6.16(c)).

Because the magnitude of the bursting stresses in the horizontal plane did not exceed the modulus of rupture for the precast beam at transfer, reinforcing steel in the horizontal plane in the end zones is not required. Accordingly, the AASHTO provisions² for pre-tensioned anchorage zones in the horizontal plane would yield a conservative design. The required area of
horizontal reinforcing based on the 4% rule (3.94 in$^2$) can be partially provided by three rows of No.4 confinement reinforcing at 2 in. on center and the rest of the confinement steel at 6 in. on center. This steel area combined with No.4 transverse straight bars at 6 in. on center yields a total area of bottom transverse steel of approximately 4.8 in$^2$, which is larger than the required 3.94 in$^2$ (Figure 6.16 (c)).

**Figure 6.16** Summary of end zone reinforcing details calculated based on current AASHTO provisions$^2$, (a) 20 foot span, (b) 41.5 foot span, (c) 60 foot span
6.6.2 NCHRP Web-Only Document 173

NCHRP Web-Only Document 173 provides recommended equations for sizing end zone reinforcing in the vertical plane. Table 3 provides the input parameters required to evaluate the recommendations of NCHRP Web-Only Document 173 for the three bridge spans and the associated results.

6.6.2.1 41.5-foot span

The magnitude of spalling stresses predicted by the NCHRP method for the 41.5-foot span is equal to 0.106 ksi. This is lower than the magnitude of spalling stresses computed from the finite element models, which is 0.4 ksi. The NCHRP method yields a smaller spalling stress for this case, however, the conclusion that no vertical end zone reinforcing is needed is consistent with the one based on finite element analyses.

### Table 6.3 NCHRP Web-Only Document 173 recommendations

<table>
<thead>
<tr>
<th></th>
<th>20 foot span</th>
<th>41.5 foot span</th>
<th>60 foot span (Same as AASHTO)</th>
</tr>
</thead>
<tbody>
<tr>
<td>h (in.)</td>
<td>8</td>
<td>18</td>
<td>24</td>
</tr>
<tr>
<td>P_r (kips)</td>
<td>417</td>
<td>1078</td>
<td>1968</td>
</tr>
<tr>
<td>A (in²)</td>
<td>460</td>
<td>757</td>
<td>1044</td>
</tr>
<tr>
<td>e (in.)</td>
<td>1.47</td>
<td>2.99</td>
<td>3.94</td>
</tr>
<tr>
<td>d_b (in.)</td>
<td>0.5</td>
<td>0.6</td>
<td>0.6</td>
</tr>
<tr>
<td>f_c (ksi)</td>
<td>5</td>
<td>5</td>
<td>5</td>
</tr>
<tr>
<td>f_s (ksi)</td>
<td>20</td>
<td>20</td>
<td>20</td>
</tr>
<tr>
<td>σ_s (ksi)</td>
<td>0.036</td>
<td>0.106</td>
<td>NA</td>
</tr>
<tr>
<td>f_r (ksi)</td>
<td>0.51</td>
<td>0.51</td>
<td>0.51</td>
</tr>
<tr>
<td>P_r (kips)</td>
<td>NA</td>
<td>NA</td>
<td>78.72</td>
</tr>
<tr>
<td>A_s (in²)</td>
<td>Not Required</td>
<td>Not required</td>
<td>3.94</td>
</tr>
</tbody>
</table>

6.6.2.2 20-foot span

For the 20-foot span, the NCHRP approach predicts negligible spalling stresses at the end face (0.036 ksi). The results from the NCHRP equations are in close agreement with the results from finite element analyses for the 8 in. deep precast beam, which exhibited negligible spalling stresses. In addition, the conclusion that no vertical reinforcing is required is supported by the results from finite element analyses.

6.6.2.3 60-foot span

For the 60-foot span, NCHRP recommendations are identical with the AASHTO provisions for pre-tensioned anchorage zones because the depth of the precast member for this span was 24 in.
6.7 Alternative Approach Using Strut-and-Tie Modeling

An alternative approach for pretensioned anchorage zone design is to use strut-and-tie modeling to determine spalling forces in the vertical plane and bursting forces in the horizontal plane. Several strut-and-tie models were investigated in the vertical and horizontal planes with the purpose of identifying the models that most closely replicated the results obtained from finite element analyses. One property of strut-and-tie models is that they ignore the contribution of concrete in tension and if chosen properly usually lead to conservative designs. Only the 41.5 ft span girder will be evaluated using strut-and-tie modeling.

6.7.1 Vertical Plane

Figure 6.17 shows the distribution of longitudinal normal stresses caused by the prestressing force at a distance \( h \) from the end of the precast member for the precast beams used in the US 360 Bridge. The majority of the prestressing force was concentrated at the bottom two layers and consisted of 24 0.6 in. diameter strands, each stressed to approximately 44 kips (43.94 kips). This resulted in a prestressing force of 1055 kips 3 in. above the bottom of the beam. The remaining two strands were located 2 in. from the top of the precast beam. These two strands created a prestressing force of 88 kips. Figure 6.18 shows the distribution of the prestressing force in the vertical plane and the orientation of principal stress vectors. The maximum principal tensile stresses in the vertical plane are located at the end face of the precast beam (yellow vectors). Also shown in this Figure is one of the strut-and-tie models that was used to estimate the magnitude of the spalling stresses at the end face of the precast beam.

The longitudinal stress diagram at a distance \( h \) from the end of the beam was integrated to produce top and bottom horizontal forces that matched the magnitude of those applied at the end of the beam. The location of these forces is shown in Figure 6.19 for the models evaluated.

Three different strut-and-tie models were investigated as shown in Figure 6.19. The strut-and-tie Model V1 consists of only one tension tie and is the model that matched most closely the distribution of spalling stresses at the end face of the precast beam. The disadvantage of this model is that all the vertical steel intended to resist spalling stresses must be placed within 4.5 in. (\( h/4 \)) from the end of the beam. The tension force in the tie was 28.2 kips (as opposed to 45.7 kips determined using the 4% rule of AASHTO Provisions). If a 20 ksi allowable stress is used to determine the area of vertical steel then the required area is 1.41 in\(^2\). The total vertical area of steel in the first row, used in the precast beams for the US 360 Bridge, was 1.08 in\(^2\), which is approximately 77% of the required steel area based on strut-and-tie model V1. The second row of extended stirrups and confinement steel is the same as the first row and is located 5 in. from the end of the member, which is past the prescribed distance of \( h/4 \) (4.5 in.). However, because the results of finite element analyses for the 41.5-foot span revealed that spalling stresses at the end of the beam were smaller than the modulus of rupture of concrete at transfer, such a distribution of steel at the end zones was deemed acceptable. In addition, the visual inspection of all fabricated precast beams confirmed that no cracking was observed at the end zones.
Compared to the 4% AASHTO rule, strut-and-tie model V1 leads to designs that are more economical and less congestion in the end zones. However, experimental testing is required to validate the suitability of this model for sizing vertical reinforcing in the end zones, especially for cases when spalling stresses exceed the modulus of rupture of concrete at transfer.

Figure 6.17 Distribution of longitudinal normal stresses at the ends of the precast beam

Figure 6.18 Principal stress vectors for 41.5 foot span Case 3 – vertical plane

Strut-and-tie models V2 and V3 were attractive alternatives, because they allow the distribution of vertical steel at the end zone to be uniform throughout the disturbed region \( h \), which is helpful in avoiding congestion. The sum of tension forces in the ties of model V2 is equal to 43 kips, which is close to 45.72 kips estimated based on the 4% AASHTO rule. Similarly, the sum of tension forces in the ties of model V3, is also equal to 43 kips, and allows and even more uniform distribution of vertical steel in the end zone. However, these two models were not favored because the distribution of spalling stresses at the end zones obtained from
finite elements analyses were highest at the end face of the member, and diminished quickly away from the end of the member.

Figure 6.19 Strut-and-tie models for the vertical plane
6.7.2 Horizontal Plane

Figure 6.20 illustrates the diffusion of the prestressing force introduced in the bottom two strand layers in the horizontal plane using principal stress vectors. Because the prestressing force at the bottom two strand layers was introduced within the footprint of the precast web, it will tend to distribute outwards towards the flanges as it is being transferred to the surrounding concrete. Also shown in this Figure is one of the strut-and-tie models used to determine the magnitude of bursting stresses within the disturbed region.

![Image of principal stress vectors](image)

**Figure 6.20** Principal stress vectors for 41.5 foot span Case 3 – horizontal plane

Three strut-and-tie models were investigated (Figure 6.21 (a)-(c)). Model H1 is the simplest of the three and consist of only one tension tie. The tension force in the tie is 92 kips, which is approximately 8.7% of the total prestressing force in the bottom two strand layers. Model H2 consist of two tension ties. The sum of tension forces in the ties of this model is 59 kips, which is 5.6% of the total prestressing force in the bottom two strand layers. Models H1 and H2 are attractive because of their simplicity, however because the distribution of horizontal bursting stresses observed in the finite element models was relatively uniform in the disturbed region they were not considered for adoption in design. Model H3 was the one that most closely matched the distribution of bursting stresses. This model consists of three tension ties throughout the disturbed region. The sum of tension forces in the ties is 83 kips, which is 7.87% of the total prestressing force in the bottom two strand layers. The utilization of this model in design presents an even more conservative approach compared to the 4% AASHTO\(^2\) rule. If this model is selected, then the horizontal reinforcing can be distributed uniformly throughout the disturbed region.
Figure 6.21 Strut-and-tie models for the horizontal plane
6.8 Summary

Table 6.4 provides a summary of end zone reinforcing determined using the various methods described in this paper. With the exception of the vertical plane in the 24 in. deep precast beam used in the 60-foot span, the results of finite element analyses suggest that no end zone reinforcing is required for the other cases. As stated earlier, AASHTO LRFD Specifications require end zone reinforcing in pre-tensioned anchorage zones, regardless of the span length, strand pattern, geometry of the precast member, eccentricity or magnitude of the prestressing force. Table 6.4 provides the end zone reinforcing for the vertical and horizontal planes based on AASHTO. The result of the method proposed in the NCHRP report are consistent with the results of finite element analyses. For the 24 in. deep precast beam used in the 60-foot span the NCHRP method predicts a higher amount of vertical reinforcing and can therefore be used conservatively in design. Only the 18 in. deep precast beam used in the US 360 Bridge (41.5-foot span) was evaluated using the strut-and-tie method. Compared to the 4% AASHTO rule, strut-and-tie model V1 leads to designs that are more economical and creates less congestion in the end zones. However, experimental testing is required to validate the suitability of this model for sizing vertical reinforcing in the end zones, especially for cases when spalling stresses exceed the modulus of rupture of concrete at transfer. In the horizontal plane, strut-and-tie model H3 presents an even more conservative approach compared to the 4% AASHTO rule. If this model is selected, then the horizontal reinforcing can be distributed uniformly throughout the disturbed region.

<table>
<thead>
<tr>
<th></th>
<th>Area of end zone reinforcing (in.²)</th>
<th>20 foot span</th>
<th>41.5 foot span</th>
<th>60 foot span</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>Vertical</td>
<td>Horizontal</td>
<td>Vertical</td>
</tr>
<tr>
<td></td>
<td>FEA*</td>
<td>Not required</td>
<td>Not required</td>
<td>Not required</td>
</tr>
<tr>
<td></td>
<td>AASHTO²</td>
<td>0.87</td>
<td>0.87</td>
<td>2.29</td>
</tr>
<tr>
<td></td>
<td>NCHRP³</td>
<td>Not required</td>
<td>Not addressed</td>
<td>Not required</td>
</tr>
<tr>
<td></td>
<td>Strut-and-Tie</td>
<td>Not evaluated</td>
<td>Not evaluated</td>
<td>1.41</td>
</tr>
</tbody>
</table>

*FEA = Finite Element Analysis

6.9 Conclusions and Recommendations

Precast inverted T-beams with tapered webs present a unique shape that is being implemented for the first time in Virginia in the construction of the US 360 Bridge near Richmond. Properly accounting for stresses created in the end zones as a result of the diffusion on the prestressing force from the strands into the surrounding concrete is essential to preclude excessive cracking that may lead to strength and serviceability concerns. While 3D linear elastic
finite element analyses were employed in this study to gain an understanding of the stresses that develop at the end zones of precast inverted T-beam in the vertical and horizontal planes, such analysis may not always be a viable option in a design office. Accordingly, the following conclusions and recommendations are intended to aid engineers when sizing reinforcing in the pre-tensioned anchorage zones of precast inverted T-beams with tapered webs.

Vertical Plane:

- Although this study did not include an exhaustive array of various precast beam depths, it can be concluded that precast inverted T-beams 18 in. deep or less experience spalling and bursting stresses that are lower than the modulus of rupture of concrete at transfer. As a result, theoretically no vertical reinforcing is required to resist these stresses. The recommendations provided in NCHRP Report 173\(^3\) corroborate this conclusion and may be used to evaluate the need for such reinforcing. The application of AASHTO Provisions\(^2\) for pre-tensioned anchorage zones in the vertical plane of precast inverted T-beams with tapered webs that are 18 in. deep or less provides a conservative alternative. If such vertical reinforcing is provided, it should be placed with a distance equal to \(h/4\) from the end of the beam, where \(h\) is the depth of the precast member, or as close to end face as practically possible, because spalling stresses at the end face were the dominating type of tensile stresses in terms of magnitude.

- While the 18 in. depth for precast inverted T-beams with tapered webs does not represent the dividing line at which spalling stresses at the end faces exceed the modulus of rupture of concrete, it can be conservatively stated that the application of AASHTO provisions\(^2\) for beams that are 18 in. deep or greater is also conservative. Similarly, for the beams in this bracket, the vertical reinforcing at the end zones should be placed with a distance equal to \(h/4\) from the end of the beam, where \(h\) is the depth of the precast member, or as close to end face as practically possible, because the magnitude of vertical tensile stresses at the end zones diminishes quickly past the first few inches from the end face. Vertical steel at the end zones can consist of stirrups as well as the vertical component of the AASHTO\(^2\) required confinement steel.

- As an alternative to AASHTO provisions\(^2\) and NCHRP recommendations\(^3\), vertical reinforcing in the end zones can be calculated based on strut-and-tie model V1. Compared to the 4% AASHTO\(^2\) rule, strut-and-tie model V1 leads to designs that are more economical and creates less congestion in the end zones. However, experimental testing is required to validate the suitability of this model for sizing vertical reinforcing in the end zones, especially for cases when spalling stresses exceed the modulus of rupture of concrete at transfer.

Horizontal Plane:

- In none of the cases considered in this study did the bursting stresses exceed the modulus of rupture of concrete at transfer. Accordingly, no reinforcing is required in the horizontal
plane to resist these stresses. However, the application of the 4% rule presented in AASHTO\textsuperscript{2} for sizing reinforcing in the horizontal plane is a conservative alternative. If such reinforcing is provided, it should be placed within a distance $h$ from the end of the precast flange. The AASHTO\textsuperscript{2} required confinement steel can be used for this purpose given that it needs to be provided for a distance up to $1.5d$ from the end of the member. In addition, the straight transverse bars in the precast flanges provided to resist the weight of wet concrete and transverse bending moments due to live loads can be used to resist the bursting force based on the 4% rule.

- Alternatively, horizontal reinforcing at the end zones can be sized based on strut-and-tie model H3. The utilization of this model in design presents an even more conservative approach compared to the 4% AASHTO\textsuperscript{2} rule. If this model is selected, then the horizontal reinforcing can be distributed uniformly throughout the disturbed region.

References

7. Fountain, R.S., “A Field Inspection of Prestressed Concrete Bridges”, Portland Cement Association (PCA),1963
Chapter 7: Investigation of Composite Action in Bridges Built with Adjacent Precast Inverted T-Beams and Cast-in-Place Topping

(Submitted to 2014 PCI Convention and National Bridge Conference)
Investigation of Composite Action in Bridges Built with Adjacent Precast Inverted T-beams and Cast-in-Place Topping

Abstract

Short-to-medium-span composite bridges constructed with adjacent precast inverted T-beams and cast-in-place topping are intended to provide a higher degree of resistance against reflective cracking and time dependent effects compared to voided slab and adjacent box girder systems. This paper investigates the composite action between the unique precast and cast-in-place element shapes. A full-scale composite beam has been tested under different loading arrangements with the purpose of simulating the service level design moment, strength level design shear, strength level design moment and nominal moment capacity. To investigate the necessity of extended stirrups one half of the span featured extended stirrups whereas the other half featured no extended stirrups. It is shown that the system behaved compositely at all loading levels and that no slip occurred at the interface. In addition to measuring slip at various interface locations full composite action has been verified by comparing load displacement curves obtained analytically and experimentally. It is concluded that because of the large contact surface between the precast and cast-in-place elements, cohesion alone appears to provide the necessary horizontal shear strength to ensure full composite action.

7.1 Introduction

Most bridge systems that consist of prefabricated elements and feature a jointless riding surface rely on some type of composite action. Typically, for concrete bridges, the term composite construction refers to the combination of precast girders with a cast-in-place deck or topping. The cast-in-place deck meets functional requirements by providing a smooth, useful surface and, in addition, substantially stiffens and strengthens the precast unit (Nilson\(^1\)). When superimposed loads are applied to a composite system there is a tendency for the cast-in-place slab to slip horizontally, the bottom face of the slab tending to move outward with respect to the top face of the precast girder, which tends to displace inward (Nilson\(^1\)). Preventing this slip is essential in ensuring full composite action and to do that there must be a means for transferring shear forces across the interface between the two components of the composite member. Resistance against interface shear forces can be provided by the natural adhesion and friction between the cast-in-place and precast components. Deliberately roughening the top surface of the precast girder enhances the contribution of adhesion and friction to the horizontal shear strength of the composite member. In addition, for composite systems that feature a broad interface, no other provisions need to be made to transfer the horizontal shear stresses. When the contribution of
adhesion and friction are not sufficient, extended stirrups are typically provided to enhance slip resistance through dowel action and by holding the two components in intimate contact.

To ensure full composite action, the interface shear force must be smaller than the horizontal shear strength of the interface. There are various ways to calculate the interface shear force, or horizontal shear demand. When a beam is un-cracked and its behavior is linear elastic, horizontal shear stresses can be estimated using the following equation:

\[ v_h = \frac{VQ}{Ib_v} \]  \hspace{1cm} (1)

where:
- \( V \) = vertical shear force at location under consideration
- \( Q \) = first moment of area of portion above interface with respect to neutral axis
- \( I \) = moment of inertia of composite cross-section
- \( b_v \) = width of the interface

Loov\(^2\) states that this equation can be used to evaluate the horizontal shear stress for cracked beams if \( Q \) and \( I \) are based on the cracked section. Because it provides a common basis for comparison, this equation was adopted in previous studies even though Hanson\(^3\) and Saeman and Sasha\(^4\) recognized that it does not give an exact representation of the horizontal shear stress at failure (Loov\(^2\)). As an alternative to the classical elastic strength of materials approach, a reasonable approximation of the factored interface shear force at the strength or extreme event limit state for either elastic or inelastic behavior and cracked or uncracked sections can be provided by Equations 2 and 3\(^5\).

\[ V_{ui} = v_{ui}A_{cv} \]  \hspace{1cm} (2)

where:
- \( V_{ui} \) = factored interface shear force on area \( A_{cv} \) (kips)
- \( v_{ui} \) = factored interface shear stress (ksi)
- \( A_{cv} \) = area of concrete considered to be engaged in horizontal shear transfer (in.\(^2\))

\[ v_{ui} = \frac{V_u}{b_{vi}d_v} \]  \hspace{1cm} (3)

- \( V_u \) = factored vertical shear force at section under consideration (kips)
- \( b_{vi} \) = interface width considered to be engaged in shear transfer (in.)
- \( d_v \) = the distance between the centroid of the tension steel and the mid-thickness of the slab to compute a factored interface shear stress (in.)

The interface shear force can also be calculated based on equilibrium conditions by computing the actual change in compressive or tensile force in any segment\(^6\). For example if
the change in the compressive force over a segment of length \( l_v \) is \( C \), and if the width of the interface is \( b_v \), then the horizontal shear stress can be computed by Equation 4, which implies that the entire length of the shear span can be used to transfer the horizontal shear force:

\[
v_{ul} = \frac{C}{b_v l_v}
\]

(4)

where:

\[ C = \text{change in the compressive force over a segment of length } l_v \]

Loov\(^2\) explains how Equations 1, 3 and 4 are closely related although they appear different. For example in Equation 1 the term \( VQ/I \) represents the rate of change of force in the flange. Equation 4 represents the average rate of change of force in the flange in segment whose lengths is \( l_v \). For beams subject to points loads, Equations 1 and 4 would yield the same result because the vertical shear diagram between the points loads will be constant. For beams subject to uniformly distributed loads Equation 4 misses the locations with the highest horizontal shear stress because it reports only the average shear stress in the segment under consideration. Also, Equation 3 is similar to Equation 4 because \( V = dM/dx \) is the rate of change of moment. Loov states that if the compression block is entirely within the flange, and the small variation in the depth of the stress block is ignored, then the compression force \( C \) will be equal to \( M/(d-a/2) \) and the rate of change of force in the flange will be \( V/(d-a/2) \). Therefore \( V/d_v \) is simply a non-conservative simplification of Eq.1\(^2\).

After the horizontal shear demand has been determined a method for estimating the horizontal shear capacity is required to design for composite action. According to AASHTO LRFD Specifications\(^5\) the nominal shear resistance of the interface plane shall be taken as:

\[
V_{ni} = c A_{cv} + \mu \left( A_{vf} f_y + P_c \right) \leq \min \left[ \frac{K_1 f'_c A_{cv}}{K_2 A_{cv}} \right]
\]

(5)

in which:

\[ A_{cv} = b_v l_{vi} \]

where:

\[ A_{vf} = \text{area of interface shear reinforcement crossing the shear plane within the area } A_{cv} \text{ (in.}^2\text{)} \]

\[ L_{vi} = \text{interface length considered to be engaged in shear transfer (in.)} \]

\[ c = \text{cohesion factor specified in Article 5.8.4.3 (ksi)} \]

\[ \mu = \text{friction factor specified in Article 5.8.4.3} \]

\[ f_y = \text{yield stress of reinforcement but design value not to exceed 60 ksi.} \]

\[ P_c = \text{permanent net compressive force normal to the shear plane; if force is tensile, } P_c = 0.0 \text{ kip. (kips)} \]
\( f_c^* = \) specified 28-day compressive strength of the weaker concrete on either side of the interface (ksi)

\( K_1 = \) fraction of concrete strength available to resist interface shear, as specified in Article 5.8.4.3 of AASHTO LRFD Specifications\(^5\)

\( K_2 = \) limiting interface shear resistance specified in Article 5.8.4.3 (ksi)

Equation 5 is a modified shear friction model accounting for a contribution, evident in the experimental data, from cohesion and/or aggregate interlock depending on the nature of the interface under consideration given by the first term\(^5\). This equation is similar to the one used to estimate the vertical shear capacity of a concrete section, \( V_c + V_s \), where \( V_c \) represents the shear strength provided by concrete and \( V_s \) the shear strength provided by transverse reinforcing steel.

Article 5.8.4.4 of AASHTO LRFD Specifications\(^5\) requires a minimum area of interface shear reinforcement, which can be estimated by Equation 6.

\[
A_{vf} \geq \frac{0.05 A_{cv}}{f_y}
\]  

(6)

Prior to 2006 AASHTO LRFD Specifications\(^7\) and AASHTO Standard Specifications\(^8\) have required a minimum area of reinforcement based on the full interface area; similar to Equation 6, irrespective of the need to mobilize the strength of the full interface area to resist the applied factored interface shear\(^5\). In 2006, additional minimum area provisions, applicable only to girder slab interfaces were introduced with the purpose of eliminating the need for additional interface shear reinforcement due simply to a beam with a wider top flange being utilized in place of a narrower flanged beam\(^5\). These additional provisions are provided below for convenience:

- The minimum interface shear reinforcement, \( A_{vf} \), need not exceed the lesser of the amount determined using Equation 6 and the amount needed to resist 1.33\( V_{ui}/\phi \) as determined using Equation 2.

- The minimum reinforcement provisions specified herein shall be waived for girder/slab interfaces with surface roughened to an amplitude of 0.25 in. where the factored interface shear stress, \( V_{ui} \) of Equation 3, is less than 0.210 ksi, and all vertical (transverse) shear reinforcement required by the provisions of Article 5.8.1.1 is extended across the interface and adequately anchored in the slab.

The first bulleted item establishes a rational upper bound for the area of interface shear reinforcement required based on the interface shear demand rather than the interface area as stipulated by Equation 6\(^5\). This treatment is analogous to minimum reinforcement
provisions for flexural capacity where a minimum additional overstrength factor of 1.33 is required beyond the factored demand\textsuperscript{5}. The second bulleted item suggests that an intentionally roughened surface can be expected to achieve 210 psi of horizontal shear resistance, but still requires the vertical shear reinforcing to be extended into the slab.

The inverted T-beam system is a new bridge system that consists of adjacent precast inverted T-beams with tapered webs, covered with a cast-in-place topping. This bridge system is intended to provide a higher degree of resistance against reflective cracking and time dependent effects compared to voided slab and adjacent box girder systems\textsuperscript{9,10}. This system is being implemented for the first time in Virginia, on US 360, near Richmond. Figure 1(a) shows the elevation of the US 360 Bridge and Figure 1(b) shows the transverse cross-section of the bridge. US 360 Bridge is a two-span continuous bridge. The clear span for the precast inverted T-beams is approximately 41 feet. Because of the unique shape of the precast beam, the composite action behavior of this bridge system was of interest and was investigated by testing a full-scale typical composite cross-section to failure. The purpose of the research presented in this paper is to:

- Investigate whether full composite action can be maintained at service and strength level design loads as well as under loads that simulate the nominal moment capacity of the composite section,
- Investigate the necessity of extended stirrups to ensure full composite action and determine whether cohesion alone can provide the necessary horizontal shear strength to achieve full composite action,
- Investigate the applicability of cohesion and friction factors stipulated in AASHTO\textsuperscript{5} and those recommended by other researchers to the uniquely shaped composite section described herein,
- Investigate the necessity of minimum horizontal shear reinforcing provisions stipulated in AASHTO\textsuperscript{5} for the interface condition described herein

7.2 Previous Studies

French et al.\textsuperscript{11} investigated composite action behavior in composite bridges built with adjacent precast inverted T-beams with straight webs and covered with a cast-in-place topping. This investigation was carried out by testing two laboratory bridge specimens named Concept 1 and Concept 2. Concept 1 laboratory bridge specimen consisted of two spans whereas Concept 2 laboratory bridge specimen consisted of a single span. The horizontal shear reinforcing used in Span 2 of the Concept 1 bridge was based on AASHTO LRFD Specifications\textsuperscript{5}, whereas Span 1 was constructed with fewer horizontal shear reinforcing bars and did not satisfy the minimum horizontal shear reinforcing requirements of 2005 AASHTO LRFD Specifications\textsuperscript{5}. The Concept 2 laboratory bridge was constructed
with no horizontal shear reinforcing. Both bridge specimens had a standard raked finish (1/4 in. rake) on the top horizontal surface of the precast web. Furthermore, each specimen had a roughened diamond pattern with approximately 1/8 in. to 1/4 in. perturbations on the vertical web surfaces of the precast panels. Likewise, East span of the Concept 1 bridge, which was constructed with the 5 1/4 in. thick precast flange, also had the tops of the precast flanges roughened with the same diamond pattern. It should be noted that the inverted T-beam system investigated by French et al. featured transverse hooked bars in the precast elements that protruded from the precast webs into the cast-in-place topping.

![Figure 7.1](image)

**Figure 7.1** (a) Elevation of US 360 Bridge, (b) Transverse cross-section of US 360 Bridge

In the tests on both spans of Concept 1 laboratory bridge and on Concept 2 laboratory bridge, the sections were observed to remain composite well beyond service load levels, through the full range of loading to the maximum capacity of the loading system, which was in excess of the predicted nominal capacity of the Concept 1 and 2 bridges.

Longitudinal strains measured throughout the depth of the composite cross-sections indicated linear distributions, which evince full composite action. The Kent and Park model was used to determine the corresponding compressive stress distribution in the CIP section assuming unconfined concrete models. Tested values were used for the maximum compressive concrete strength and a corresponding concrete strain assumed to be 0.002 at the
maximum compressive stress. Integrating the nonlinear stress distribution, resulted in an estimate of the maximum compression force achieved in the slab during loading to the ultimate capacity\textsuperscript{11}. The horizontal shear stress estimated in the system at the precast-CIP interface was subsequently calculated by dividing the total compression force by half of the center-to-center of bearing span length and the total width of the bridge structure, and was determined to be 135 psi\textsuperscript{11}. This method of calculating the horizontal shear stress is based on the approach presented by Equation 4 and gives an average horizontal shear stress. In addition, this method assumes that the failure mode in horizontal shear consists of a horizontal shear plane.

French et al.\textsuperscript{11} concluded that AASHTO LRFD Specifications\textsuperscript{5} should allow for the design of composite bridges with adjacent precast inverted T-beams with straight webs without horizontal shear ties, and allow the development of a maximum factored horizontal shear stress of 135 psi in sections with intentionally roughened surfaces (i.e., ¼ in. rake) unreinforced for horizontal shear. The proposed friction factor was based on AASHTO LRFD Specifications\textsuperscript{5} for surfaces intentionally roughened to an amplitude of 0.25 in. The $K_1$ and $K_2$ values, which provide upper bound estimates of the horizontal shear capacity of a given section, selected to be used in the proposed specification modifications are simply the smallest, or most conservative of the existing $K_1$ and $K_2$ values\textsuperscript{11}. The proposed specification recommendations by French et al.\textsuperscript{11} are presented below. The recommendations are in italics and the current AASHTO\textsuperscript{5} provisions are provided in non-italicized font to illustrate the context in which they are proposed.

5.8.4.3 Cohesion and Friction Factors

The following values shall be taken for cohesion, c, and friction factor, $\mu$:

For concrete placed against a clean concrete surface, free of laitance, but not intentionally roughened:

\[ c = 0.075 \text{ ksi} \]
\[ \mu = 0.6 \text{ ksi} \]
\[ K_1 = 0.2 \]
\[ K_2 = 0.8 \text{ ksi} \]

For normal weight concrete placed against a clean concrete surface, free of laitance, with surface intentionally roughened to an amplitude of 0.25 in. and no interface shear reinforcement provided crossing the shear plane up to the minimum required $A_{sf}$ in Eq. 5.8.4.4-1:
\[ c = 0.135 \text{ ksi} \]
\[ \mu = 1.0 \]
\[ K_1 = 0.2 \]
\[ K_2 = 0.8 \text{ ksi} \]

5.8.4.4 Minimum Area of Interface Shear Reinforcement

- The minimum reinforcement provisions specified herein shall be waived for girder/slab interfaces with surface roughened to an amplitude of 0.25 in. where the factored interface shear stress, \( v_{ui} \) of Equation 3, is less than 0.210 ksi, and all vertical (transverse) shear reinforcement required by the provisions of Article 5.8.1.1 is extended across the interface and adequately anchored in the slab.
- For the cast-in-place concrete of precast-composite slab-span systems that is cast on clean precast inverted-T surfaces free of laitance, with a surface intentionally roughened to an amplitude of ¼ in., the minimum reinforcement provisions specified herein shall be waived.

C5.8.4.4

With respect to a girder/slab interface, the intent is that the portion of the reinforcement required to resist vertical shear which is extended into the slab also serves as interface shear reinforcement.

In the case of precast-composite slab-span systems, research (French et al. 2010) has shown that transverse reinforcement was not required across the CIP-precast interface in order to achieve composite action. Similar results were obtained in studies by Naito et al. (2008).

Because composite bridges constructed with the precast inverted T-beams with tapered webs and cast-in-place topping represent a unique composite cross-sectional shape, the applicability of existing provisions in AASHTO LRFD Design Specifications\(^5\), and the recommendations proposed by French et al. was investigated.

7.3 Experimental Investigation

A full scale composite beam representing a typical transverse section was tested with the purpose of investigating its performance under design service level and strength level moments and shears. To investigate the necessity of extended stirrups half of the composite beam span featured extended stirrups whereas the other half did not. Initially, extended stirrups were provided along the entire span of the precast beam, however, prior to the placement of the cast-in-place topping half of them were cut off.
Figure 7.2 (a) shows the cross-sectional dimensions of the composite section. Figure 7.2 (b) shows the reinforcing details for half of the span that featured extended stirrups, whereas Figure 7.2 (c) features the reinforcing details for the other half. All precast surfaces in contact with the cast-in-place topping were roughened. The tapered precast webs and the tops of the precast flanges were roughened in the longitudinal direction to enhance composite action in the transverse direction of the bridge. Full composite action in the transverse direction is desired to avoid delamination at the precast beam cast-in-place topping interface because of transverse bending caused by wheel loads. Figure 3 illustrates the roughened precast surfaces. The roughened surface on the tapered webs was created by using steel forms, the inside of which featured the pattern shown in Figure 4. The top of the precast flanges was roughened in the longitudinal direction by using a traditional ¼ in. rake finish. The top of the precast web was roughened in the transverse direction by performing a ¼ in. rake finish to enhance composite action in the longitudinal direction.

![Figure 7.2](image)

**Figure 7.2** (a) Composite beam cross-section, (b) half of the span with extended stirrups, (c) the other half of the span without extended stirrups
Figure 7.3 Roughened precast surfaces

Figure 7.4 Roughened surface pattern in the longitudinal direction

Figure 7.5 shows the elevation of the composite beam and some of the instrumentation used to verify composite action. A displacement sensor (denoted WP-7) was used at mid-span with the purpose of comparing the load versus mid-span deflection curve obtained experimentally with that obtained analytically assuming full composite action. Displacements sensors were also used at quarter points (denoted WP-8 and WP-6) with the purpose of comparing the load versus quarter span deflection curves, provided that half of the span contained extended stirrups whereas the other half had no extended stirrups. Identical load versus quarter span deflections curves serve as evidence that the presence of extended stirrups is not required to enhance composite action.

A photograph of the test setup is provided in Figure 7.6 featuring the loading frame near mid-span. A 220 kip closed-loop servo controlled hydraulic actuator powered by a 30 gallons per minute hydraulic pump was used to load the composite system monotonically. A pin support was provided at one end of the beam and a roller support was provided at the other end to accommodate any potential longitudinal translation during testing. The pin support was provided by a solid circular steel section which rested on an assembly of a semicircular steel pipe section and a channel welded together to receive the solid steel section and create an assembly that allowed rotation but not longitudinal translation. Similarly, the roller support was provided by welding individual quarter circle steel pipe sections to a steel channel to create an assembly that allowed rotation and longitudinal translation at the same time (Figure 7.5 and Figure 7.7). The precast flanges at the ends of the precast beam were terminated one foot short from the end of the beam to prevent high flexural stresses in the
precast flanges at the bearing points (such as abutments and intermediate supports). The cast-in-place topping for the tested full scale beam followed the outline of the precast beam at the ends.

In addition to the displacement sensors, ten linear variable differential transformers (LVDT’s) were used to ensure that there was no slip during the various loading stages. Loss of composite action would be manifested as a relative slip between the precast and the cast-in-place components. Five LVDT’s were used at each end (Figure 7.7) to capture any potential slip. The LVDT’s at each end consisted of one installed at the interface between the top of the precast web and the cast-in-place topping, two installed at the interface between the precast flanges and the cast-in-place topping and two others installed near the ends of the composite beam but on the sides, at the interface between the precast flanges and the cast-in-place topping.

Table 7.1 provides a summary of the moments and shears that each individual precast inverted T-beam in the US 360 Bridge was expected to be subject to. The moments and shears due to each load case are tabulated, and that information was used to calculate design moments and shears using Service I and Strength I load combinations. Three tests were performed with the purpose of simulating the maximum service level positive moment, the maximum strength level shear, the maximum strength level positive moment and the nominal moment capacity of the composite section. During the first test the simply supported beam was subject to two point loads symmetrical about mid-span (Figure 7.8 (a)). The two point loading was applied by attaching a spreader beam to the actuator and by supporting the spreader beam on two tire prints 4 ft apart. The 4 ft spacing was intended to represent tandem axle spacing. The actuator load required to simulate the maximum service level positive moment was estimated to be 40 kips (20 kips on each tire print). During this test the composite beam was expected to remain un-cracked and behave elastically.

![Figure 7.5 Drawing of Test Setup](image)
Figure 7.6 Photograph of Test Setup

Figure 7.7 Location of LVDT’s at the ends of the composite beam to measure slip

Table 7.1 Design moments and shear for each composite beam at service and at ultimate

<table>
<thead>
<tr>
<th>Service (Service I)</th>
<th>Shears (kips)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Moments (ft-kips)</td>
<td></td>
</tr>
<tr>
<td>$+M_{\text{invT}}$</td>
<td>173</td>
</tr>
<tr>
<td>$+M_{\text{deck}}$</td>
<td>231</td>
</tr>
<tr>
<td>$+M_{\text{live}}$</td>
<td>297</td>
</tr>
<tr>
<td>$-M_{\text{live}}$</td>
<td>219</td>
</tr>
<tr>
<td>$+M_{\text{superD}}$</td>
<td>60</td>
</tr>
<tr>
<td>$-M_{\text{superD}}$</td>
<td>107</td>
</tr>
<tr>
<td>$+M_{\text{service}}$ = 761</td>
<td></td>
</tr>
<tr>
<td>Ultimate (Strength I)</td>
<td>Shears (kips)</td>
</tr>
<tr>
<td>$+M_a$</td>
<td>1100</td>
</tr>
<tr>
<td>$-M_a$</td>
<td>516</td>
</tr>
</tbody>
</table>

LVDT’s at beam ends to measure slip (5 at each end)
The purpose of the second test was to simulate the strength level design shear. The loading frame was moved from mid-span to the position described in Figure 7.8 (b). The actuator load required to simulate strength level design shear was estimated to be 118 kips (59 kips on each tire print). The strength level design vertical shear was simulated on the portion of the composite beam without any extended stirrups with the purpose of subjecting the most critical half of the span to the design vertical shear force. The underlying logic in this approach was that if the half of the span without any extended stirrups could resist the design vertical shear force without incurring any slip, then the other half should be able to at least offer a comparable performance. Even though the actuator load in this test simulated strength level design shear forces, the behavior of the composite beam was expected to be linear elastic when tested material properties were considered.

During the third and the final test, the loading frame was moved back to the mid-span of the composite beam and the load was increased monotonically to simulate strength level design positive moment and the nominal positive moment capacity (Figure 7.8 (c)).

![Figure 7.8 Summary of loading arrangements for the three tests, (a) simulation of service level design positive moment, (b) simulation of strength level design vertical shear, (c) simulation of strength level design positive moment and nominal moment capacity](image-url)
7.4 Analytical Investigation

Before the three tests were conducted, an estimation of the vertical and horizontal shear capacity of the composite beam was performed based on AASHTO LRFD Design Specifications\textsuperscript{5} using several assumptions. These estimations were conducted to ensure that the composite beam had adequate vertical and horizontal shear strength to resist the loads induced during the three tests. In addition, an estimation of the actuator load versus mid-span deflection curve was conducted assuming full composite action, with the purpose of comparing this curve with the one obtained experimentally.

7.4.1 Estimation of Vertical Shear Capacity

The estimation of the vertical shear capacity was performed in accordance with Article 5.8.3.3 of AASHTO LRFD Design Specifications\textsuperscript{5} based on Equations 7, 8 and 9. In addition, the vertical shear strength provided by concrete was calculated using the entire composite cross-section and the lower concrete compressive strength ($f'_c = 4$ ksi). Furthermore, this estimation was conservatively based on the simplified procedure for non-prestressed sections. Vertical stirrups were considered to provide shear strength only if they are extended in the cast-in-place topping. The presence of the bent transverse bars in the cast-in-place topping and the closed stirrups that encompass the prestressing strands in the precast beam were considered to contribute towards the vertical shear resistance of the composite section. Vertical shear demand was calculated at the critical section and was based on the loads simulated during Test 2. This information is provided in Table 7.2. The last column in Table 7.2 gives the ratio of the vertical shear demand to the vertical shear capacity. It can be observed that even when the contribution of the extended stirrups is ignored the demand to capacity ratio is still considerably lower than one.

\[
V_n = \min \left[ V_c + V_s + V_p, 0.25 f'_c b_v d_v + V_p \right] \tag{7}
\]

\[
V_c = 0.0316 \beta f'_c b_v d_v \tag{8}
\]

\[
V_s = \frac{A_v f_y d_v (\cot \theta + \cot \alpha) \sin \alpha}{s} \tag{9}
\]

<table>
<thead>
<tr>
<th>Table 7.2 Calculated vertical shear demand and vertical shear strength</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Vertical Shear Strength (kips)</strong></td>
</tr>
<tr>
<td>$\phi V_c$</td>
</tr>
<tr>
<td>$\phi V_{\text{extended}}$</td>
</tr>
<tr>
<td>Portion without extended stirrups</td>
</tr>
<tr>
<td>Portion with extended stirrups</td>
</tr>
</tbody>
</table>
7.4.2. Estimation of Horizontal Shear Capacity

To determine whether slip could be prevented a comparison of the horizontal shear demand and capacity was performed. Horizontal shear demand was based on the loads simulated during Test 2 and was determined using Equations 1, 3 and 4 (Table 7.3). Because the composite beam remained un-cracked during Test 2 the utilization of Equation 1 using transformed un-cracked section properties was appropriate. Equation 1 yields higher horizontal shear stresses in Plane 1 compared to Plane 2 because Plane 1 is closer to the neutral axis, which is where horizontal shear stresses are highest in an un-cracked section. A single horizontal shear stress value for Plane 3 could not be calculated using Equation 1 because Plane 3 consists of sub-planes whose distances to the neutral axis vary. Equation 3, yielded a similar horizontal shear stress value with that calculated for Plane 1 using Equation 1, which confirms that it provides a reasonable approximation of the horizontal shear stress. An examination of the derivation of Equation 3 reveals that this equations does not differentiate between horizontal shear stresses in any horizontal plane between the internal compression and tension forces. Also, because Equation 3 is a reasonable approximation for calculating horizontal shear stresses in horizontal planes, it does not apply to Plane 3. For the loading arrangement illustrated in Test 2, Equation 4 yields the average horizontal shear stress between the point of maximum moment to the point of zero moment. Because the shear diagram between these two points is not constant the horizontal shear stress calculated using Equation 4 is lower than that calculated using either Equation 1 or 3, which capture the maximum horizontal shear stress or an approximation of it.

Horizontal shear capacity was calculated based on AASHTO LRFD Specifications\(^5\) (Equation 10). In the estimation of the horizontal shear capacity, three potential slip planes were considered (Figure 7.9). Plane 1 consists of the interface between the top of the precast web and the cast-in-place topping plus the rest of the width of the composite section. Plane 1 includes an intentionally roughened interface in the transverse direction and monolithic planes. Plane 2 consists of the interfaces between the precast flanges and cast-in-place topping and the bottom width of the precast beam web. Plane 2 includes intentionally roughened interfaces in the longitudinal direction and a monolithic plane. Plane 3 consists of the entire interface between the precast and cast-in-place components and includes roughened interfaces in the transverse and longitudinal directions. The horizontal shear capacity for each plane was calculated by using the appropriate cohesion and friction factors for the type of interfaces that each plane consisted of. The cohesion and friction factors for the assumed interface conditions are provided in Table 7.4.
Table 7.3 Horizontal shear stress (Test 2 – Simulation of strength level design shear)

<table>
<thead>
<tr>
<th>Equation</th>
<th>Plane 1</th>
<th>Plane 2</th>
<th>Plane 3</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>99</td>
<td>65</td>
<td>Varies</td>
</tr>
<tr>
<td>3</td>
<td>96</td>
<td>96</td>
<td>NA</td>
</tr>
<tr>
<td>4</td>
<td>46</td>
<td>46</td>
<td>40</td>
</tr>
</tbody>
</table>

NA = not applicable

\[
V_{ni} = cA_{cv} + \mu \left( A_{vf} f_y + P_c \right) \leq \min \left\{ \frac{K_1 f' c A_{cv}}{K_2 A_{cv}} \right\}
\] (10)

<table>
<thead>
<tr>
<th>Plane 1</th>
<th>Plane 2</th>
<th>Plane 3</th>
</tr>
</thead>
<tbody>
<tr>
<td>$A_{cv} = 864 \text{ in}^2$</td>
<td>$A_{cv} = 864 \text{ in}^2$</td>
<td>$A_{cv} = 1000 \text{ in}^2$</td>
</tr>
</tbody>
</table>

Figure 7.9 Potential failure planes due to horizontal shear

Table 7.4 AASHTO LRFD Specification\(^5\) cohesion and friction factors

<table>
<thead>
<tr>
<th></th>
<th>Interface A (Intentionally roughened)</th>
<th>Interface B (Not intentionally roughened)</th>
<th>Interface C (monolithic)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Cohesion (c)</td>
<td>0.28</td>
<td>0.075</td>
<td>0.40</td>
</tr>
<tr>
<td>Friction ((\mu))</td>
<td>1</td>
<td>0.6</td>
<td>1.4</td>
</tr>
<tr>
<td>(K_1)</td>
<td>0.3</td>
<td>0.2</td>
<td>0.25</td>
</tr>
<tr>
<td>(K_2)</td>
<td>1.8</td>
<td>0.8</td>
<td>1.5</td>
</tr>
</tbody>
</table>

In addition, the estimation of the horizontal shear capacity was performed by both accounting for the presence of the extended stirrups and ignoring them. The results of this estimation are provided in Table 7.5. The horizontal shear demand and capacity values provided in Table 7.5 were calculated for one foot of length. The horizontal shear demand in terms of force was calculated by multiplying the horizontal shear stress values in Table 7.3 by the corresponding interface areas. The last six columns shows the ratio between the horizontal shear demand and capacity and suggest that a horizontal shear failure should not occur. As stated earlier, one of the goals of this study was to investigate experimentally whether adequate horizontal shear strength in such a uniquely shaped composite member can be provided solely by the natural cohesion between the two components.
Table 7.5 Comparison of estimated design horizontal shear force and horizontal shear capacity

<table>
<thead>
<tr>
<th>Eq.</th>
<th>Demand (kips) (per foot of length)</th>
<th>Capacity (kips) (per foot of length)</th>
<th>Ratio</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Plane 1</td>
<td>Plane 2</td>
<td>Plane 3</td>
</tr>
<tr>
<td>1</td>
<td>St.</td>
<td>No St.</td>
<td>St.</td>
</tr>
<tr>
<td>2</td>
<td>St.</td>
<td>No St.</td>
<td>St.</td>
</tr>
<tr>
<td>3</td>
<td>St.</td>
<td>No St.</td>
<td>St.</td>
</tr>
</tbody>
</table>

St. = with stirrups, No St. = without stirrups, NA = not applicable

7.4.3 Estimation of Full Load versus Mid-Span Displacement Curve

To verify full composite action behavior of the system under various stages of loading, the full anticipated load versus mid-span deflection curve of the simply supported beam system was estimated analytically for comparison with the load versus mid-span deflection curve obtained experimentally. To do this, material models defining the stress strain relationships for the two types of concrete and the prestressing steel present in the composite system had to be adopted.

7.4.3.1 Stress-strain relationship

For the precast and CIP concrete materials the Hognestad model was adopted and calibrated to match the tested compressive strength at 28 days. The design compressive strengths for the precast and cast-in-place components were $f_c' = 6$ ksi and $f_c' = 4$ ksi, respectively. The tested compressive strengths for the precast and cast-in-place components were 10.2 ksi and 8.5 ksi, respectively. The model consists of a second degree parabola with apex at a strain $\varepsilon_0$, which is the strain when $f_c$ reaches $f_c'$. In this case $\varepsilon_0$ was taken equal to 0.0025. This model is described mathematically in Eq.11 and graphically in Figure 7.10. The maximum usable concrete strain was taken equal to 0.004. This model is convenient for use in analytical studies involving concrete because the entire stress-strain curve is given by one continuous function. The material model for the prestressing steel consisted of a tri-linear curve, which is mathematically described by the piecewise functions in Eq.12 and illustrated in Figure 11.

$$f_c = f_c' \left[ \frac{2\varepsilon_c}{\varepsilon_0} - \left( \frac{\varepsilon_c}{\varepsilon_0} \right)^2 \right]$$ (11)
Figure 7.10 Stress strain relationship for concrete

\[
    f_{ps} = \begin{cases} 
        E_{ps} \epsilon_{ps} & \text{if } \epsilon_{ps} \leq 0.0084 \\
        240 + 1515(\epsilon_{ps} - 0.0084) & \text{if } 0.0084 \leq \epsilon_{ps} \leq 0.015 \\
        250 + 444(\epsilon_{ps} - 0.015) & \text{otherwise}
    \end{cases} 
\]  \hspace{1cm} (12)

Figure 7.11 Stress strain relationship for prestressing strand

7.4.3.2 Moment Curvature Relationship

To obtain the anticipated full load versus mid-span deflection curve for the simply supported composite beam a moment curvature relationship had to be developed for any given cross-section of the beam. Because the system consists of a pre-tensioned precast beam with straight strands and a cast-in-place topping the moment-curvature relationship was constant throughout the span. After the moment curvature relationship is defined, this information can be used to relate the moment diagram in the simply supported beam to a curvature diagram, which can then be used to calculate deflections at desired locations along the span.
Up to first crack

The moment curvature relationship up until the first crack was calculated using principles from linear elastic mechanics of materials. For the non-composite section strain profiles along the depth of the section were obtained by first calculating the stresses at the extreme fibers (Eq. 13) and then dividing them by the modulus of elasticity of the precast beam (Eq.14). Curvatures were calculated based on the slope of the strain diagram (Eq.15) and moments were calculated using statics (Eq. 16). For the composite section additional moments and curvatures up to first crack were calculated by using the section properties of the composite section (Eq. 18-20). The cracking moment due to actuator load was calculated by assuming a modulus of rupture equal to $7.5\sqrt{f'_c}$ (Eq. 17). Total curvatures in the composite system up until the first crack were calculated simply by adding the additional curvatures due to loads in the composite system to the already calculated ones on the precast beam. Total moments were calculated using statics.

The slope of the moment curvature curve defines the flexural stiffness of the precast before it was made composite and that of the composite system after the cast-in-place topping was placed. The difference in these slopes is illustrated in Figure 7.14 (a).

Non-composite section

\[
\sigma = \mp \frac{P_e}{A_{pc}} \mp \frac{P_e e_y}{I_{pc}} \mp \frac{(M_{invT} + M_{clip})}{I_{pc}} 12 y
\]  
\[\varepsilon = \frac{\sigma}{E}\]  
\[\phi_{noncomposite} = \frac{\varepsilon_{bottom} - \varepsilon_{top}}{h_{pc}}\]  
\[M_{noncomposite} = M_{invT} + M_{clip}\]

Composite section

\[
M_{cracking} = \frac{1}{12} \left[ f_r + \frac{P_e}{A_{pc}} + \frac{P_e e_y_{botcomp}}{I_{pc}} - \frac{(M_{invT} + M_{clip})}{I_{pc}} 12 y_{botcomp} \right] I_c
\]
\[
\Delta \varepsilon = \mp \frac{M_{cracking}}{I_c E_{pc}} 12 y
\]
\[
\Delta \phi = \frac{\Delta \varepsilon_{bot} - \Delta \varepsilon_{top}}{h_{pc}}
\]
\[M_{composite} = M_{noncomposite} + M_{cracking}\]
\[\phi = \phi_{noncomposite} + \Delta \phi\]
After cracking

An algorithm was used to obtain the moment curvature relationship in the composite section after cracking. This algorithm is described in Figure 7.12 and Figure 7.13 and consists of incrementally increasing the strain in the top of the cast-in-place concrete and finding the corresponding depth of the neutral axis. The strain in the top of the cast-in-place concrete and the depth to the neutral axis are used to calculate strain and stress profiles in the composite section. Compressive stress profiles in concrete are integrated to calculate internal compressive forces and the tensile stress in steel is used to calculate the internal tension force. After internal equilibrium is satisfied, the internal moment, curvatures and the depth to the neutral axis are reported. Nilson\(^1\) states that the relatively small strain discontinuity at the interface between precast and cast-in-place concrete, resulting from prior bending of the non-composite precast section, can been ignored without serious error at the overload stage. Because the strain range covered in this algorithm is relatively large, the strain discontinuity at the interface was ignored. However, the discontinuity of the concrete stress profiles at the interface of the two components was taken into account for cases when the neutral axis falls below the thinnest portion of the cast-in-place concrete topping.

Because the data from the test will include the superimposed load (actuator load) versus the corresponding mid-span deflection the full moment curvature relationship (Figure 7.14 (a)) is adjusted to reflect just the superimposed moment and the corresponding curvature (Figure 14(b)). This information is then used to construct a curvature diagram based on the moment diagram in the composite beam caused by the actuator load. Deflection at mid-span of the beam is then calculated by multiplying the individual areas in the curvature diagram by the distance between their centroids and the support (Eq.22).

7.5 Results

7.5.1 Test 1 – Simulation of Service Level Design Moment

The purpose of the first test was to load the composite beam to simulate the service level design positive moment. The actuator load required to cause this moment was estimated to be 40 kips (P\(_{Ms}\)). No cracking was observed in the precast beam during this test, which was consistent with the design requirements for a fully prestressed member. Figure 7.16 shows a comparison of the estimated and tested load versus mid-span deflection curves for the first test. As can be seen, the curves are almost identical which provides evidence that full composite action was maintained up until the service level moment. Also, a comparison of the load versus quarter-span deflection curves is illustrated in Figure 7.17. These curves are also almost identical despite the fact that one half of the span contained extended stirrups whereas the other half did not. This shows that the presence of extended stirrups is not required to ensure composite action up until the service level design positive moment. In addition, an examination of the typical load versus slip relationship at both ends of the beam, with and without extended stirrups suggests that there
is no slip at either end (Figure 7.18 and Figure 7.19), and confirms the assumption for full composite action.

7.5.2 Test 2 – Simulation of Strength Level Design Shear (V_d)

The purpose of the second test was to simulate strength level design vertical shear on the portion of the beam without the extended stirrups. The actuator load required to simulate this condition was estimated to be 118 kips (P_{Vd}). Figure 7.20 and Figure 7.21 reveal that there was no slip at either end of the beam under this load arrangement, which confirmed the hypothesis that the composite beam can resist the strength level design shear force without incurring any slip even with no extended stirrups. The maximum horizontal shear stress computed using Equation 1 was 99 psi. This observation leads to the conclusion that the design for horizontal shear of composite bridge systems consisting of adjacent precast inverted T-beams with tapered webs and cast-in-place topping can be confidently based on a cohesion factor equal to at least 99 psi.

7.5.3 Test 3 – Simulation of Nominal Moment Capacity (M_n)

The purpose of the third test was to simulate moments in the composite section that were equal to the strength level design positive moment and the nominal moment capacity of the composite section. The actuator load required to simulate the strength level design positive moment and nominal moment capacity was 76 kips (P_{Mu}) and 200 kips (P_{Mn}), respectively. The capacity of the actuator was 220 kips. The composite beam was loaded until the capacity of the actuator was met. Figure 7.22 shows a comparison between the estimated actuator load versus mid-span deflection curve to the experimentally obtained curve. It can be seen that the two curves are similar with the experimental curve exhibiting slightly higher strength and stiffness. A part of the small difference between the experimental and predicted curve can be attributed to the fact that tension stiffening was ignored in the prediction method used herein. Figure 7.23 shows a comparison of the actuator load versus quarter span deflection relationship. As can be seen, the two curves are identical, which suggests that the behavior of the half of the span without extended stirrups is identical to that of the other half of the span, which features extended stirrups. This observation confirms the hypothesis that the presence of extended stirrups is not required to maintain full composite action up to the development of the nominal moment capacity of the composite beam.
Figure 7.12 Partial algorithm for calculating Moment Curvature Relationship

Continues on Figure 7.13
Figure 7.13 Partial algorithm for calculating Moment Curvature Relationship
Figure 7.14 (a) Full moment-curvature relationship, (b) Moment-curvature relationship for superimposed loads
Figure 7.15 Moment and curvature diagram

\[ \Delta_{\text{midspan}} = \sum_{i=1}^{n} A_i y_i \]  

(22)

Figure 7.16 Comparison of predicted and experimental load vs mid-span deflection curves (up to \( P_{Ms} \))
Figure 7.17 Comparison of load quarter span deflection curves (up to $P_{Ms}$)

Figure 7.18 Typical load vs slip relationship – without extended stirrups (up to $P_{Ms}$)

Figure 7.19 Typical load vs slip relationship – with extended stirrups (up to $P_{Ms}$)

Figure 7.20 Typical load vs slip relationship – without extended stirrups (up to $P_{Vu}$)
Figure 7.21 Typical load vs slip relationship – with extended stirrups (up to $P_{Vu}$)

Figure 7.24 and Figure 7.25 show that there is no slip at either end of the composite beam, an observation that provides additional evidence about the ability of the composite beam to develop its nominal moment capacity without incurring any slip. The maximum vertical shear force at the critical section when the actuator load reached 220 kips was equal to 147 kips, which was larger than the strength level design vertical shear forces at the critical section (138 kips). Because the failure mode of the composite beam under the loading arrangement illustrated in Test 3 was of interest, the 220 kip actuator was replaced with a 400 kip actuator, and the composite beam was loaded to failure. The composite beam failed in flexure at an actuator load of 272 kips. The corresponding vertical shear force at the critical section was 173 kips including the self-weight of the composite beam. The horizontal shear stresses computed using Equation 1, 2 and 3 in the previously investigated planes are provided in Table 6. The maximum computed horizontal shear stress was 124 psi in Plane 1 and was based on Equation 1. Although the composite beam at failure exhibited significant flexural cracking, the regions near the support, with the highest vertical shear did not exhibit cracking. Accordingly, the utilization of Equation 1 for these regions is valid. In addition, the horizontal shear stresses computed using Equation 3 in Planes 1 and 2 were 120 psi. As expected, horizontal shear stresses computed using Equation 4 were lower and were equal to 110 psi for Planes 1 and 2 and 95 psi for Plane 3. Because Equation 3 is provided in AASHTO as a reasonable approximation of the horizontal shear stress, these results suggest that the design for horizontal shear of adjacent precast inverted T-beams with tapered webs and cast-in-place topping can be confidently based on the following cohesion and friction factors:

$$c = 120 \quad \mu = 1.0 \quad K_1 = 0.2 \quad K_2 = 0.8$$

Because of the flexural failure mode, the 120 psi horizontal shear stress representing the recommended cohesion factor does not constitute the maximum horizontal shear stress that can be developed in the composite inverted T-beam system described herein. The flexural failure of the composite beam prevented it from achieving higher horizontal shear stresses at the interfaces such as those achieved by French et al.\(^\text{11}\) (135 psi) in their experiments.
Figure 7.22 Comparison of predicted and experimental load vs mid-span deflection curves (Full Curve)

Figure 7.23 Comparison of load quarter span deflection curves (up to $P_{Mn}$)

Figure 7.24 Typical load vs slip relationship – without extended stirrups (up to $P_{Mn}$)

Figure 7.25 Typical load vs slip relationship – with extended stirrups (up to $P_{Mn}$)
Table 7.6 Horizontal shear stress (based on actuator load that caused failure)

<table>
<thead>
<tr>
<th>Equation</th>
<th>Horizontal shear stress (psi)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Plane 1</td>
</tr>
<tr>
<td>1</td>
<td>124</td>
</tr>
<tr>
<td>3</td>
<td>120</td>
</tr>
<tr>
<td>4</td>
<td>110</td>
</tr>
</tbody>
</table>

NA = not applicable

7.6 Conclusions and Recommendations

The analytical and experimental results presented in this paper lead to the following conclusions and recommendations:

- The full scale test described in this paper has demonstrated that full composite behavior is assured not only at service and strength level design loads, but also up to flexural failure in such a uniquely shaped composite system.
- The presence of extended stirrups in one half of the span did not result in any differences in behavior between the two halves of the span. Because the composite inverted T-beam bridge system with tapered webs and cast-in-place topping features a broad contact surface between the precast and cast-in-place components, adequate horizontal shear resistance along the interface to develop the nominal moment capacity of the composite section can be provided solely by the natural adhesion and friction between the two components.
- The composite bridge system described in this paper and used in the construction of the US 360 Bridge was able to develop at least a horizontal shear stress equal to 120 psi without the presence of extended stirrups. The failure mode of the composite section was a flexural failure. Because Equation 3 is provided in AASHTO as a reasonable approximation of the horizontal shear stress, these results suggest that the design for horizontal shear of adjacent precast inverted T-beams with tapered webs and cast-in-place topping can be confidently based on the following cohesion and friction factors:

\[ c = 120 \quad \mu = 1.0 \quad K_1 = 0.2 \quad K_2 = 0.8 \]

Because of the flexural failure mode, the 120 psi horizontal shear stress representing the recommended cohesion factor does not constitute the maximum horizontal shear stress that can be developed in the composite inverted T-beam system described herein. The flexural failure of the composite beam prevented it from achieving higher horizontal shear stresses at the interfaces such as those achieved by French et al.\textsuperscript{11} (135 psi) in their experiments.
• Roughening the tapered webs and the tops of the precast flanges in the longitudinal direction while providing a transverse rake finish only at the top of the precast web appears to provide adequate horizontal shear resistance in the longitudinal direction to resist at least a horizontal shear stress of 120 psi without the presence of extended stirrups. The minimum reinforcement requirements for cases in which the horizontal shear stress is smaller than 120 psi and the precast surfaces are roughened as described above can be waived for composite bridges consisting of adjacent precast inverted T-beams with tapered webs and cast-in-place topping.

References

6. ACI 318-2008, Building Code Requirements for Structural Concrete and Commentary, Farmington Hills, MI
13. Kovach J.D., Naito, C., Horizontal Shear Capacity of Composite Concrete Beams without Ties, ATLSS Report No. 08-05, PCI / PITA Research Project, Bethlehem, PA
Chapter 8: Investigation of Live Load Distribution Factors for Composite Bridges Constructed with Adjacent Precast Inverted T-Beams with Tapered Webs

(To be submitted to *ASCE Journal of Constructed Facilities*)
Investigation of Live Load Distribution Factors for Composite Bridges Constructed with Adjacent Precast Inverted T-Beams with Tapered Webs

Abstract

Short-to-medium-span composite bridges constructed with adjacent precast inverted T-beams and cast-in-place topping are intended to provide a higher degree of resistance against reflective cracking and time dependent effects compared to voided slab and adjacent box girder systems. This paper presents the results of an analytical and experimental study performed on the first inverted T-beam bridge in Virginia on US 360 over the Chickahominy River. The purpose of the investigation was to determine whether the behavior of this type of bridge is similar to that of adjacent voided slab or cast-in-place slab-span bridges. A finite element model of Phase I of the US 360 Bridge was created and the live load distribution factors were analytically determined. Live load tests using a stationary truck were performed on Phase I of the US 360 Bridge with the purpose of quantifying live load distribution factors and validating the results from the finite element analyses. The live load tests featured four transverse truck positions on the bridge. It is concluded that it is appropriate to estimate live load distribution factors using AASHTO provisions for cast-in-place slab span bridges.

8.1 Introduction

Typically, short-to-medium-span bridges are constructed using either adjacent precast voided slabs or adjacent box girder systems. The lack of site installed formwork, ease of erection, shallow superstructure depth, and aesthetic appeal make these systems attractive for the construction of short-to-medium-span highway bridges. However, compared to other types of prestressed concrete highway bridges, adjacent box structures are one the bridge systems with the highest deficiency percentages\(^1\). One common problem is the failure of the transverse connection which typically features grouted shear keys and a nominal amount of transverse post-tensioning. This leads to longitudinal reflective cracking at the joints between the adjacent members. Therefore, it is essential to address this problem by either investigating alternative connection details or by developing new bridge systems that are more resistant to reflective cracking.

Short-to-medium-span composite bridges constructed with adjacent precast inverted T-beams with a cast-in-place topping are intended to provide a higher degree of resistance against reflective cracking and time dependent effects compared to voided slab and adjacent box girder systems\(^2,3\). The system consists of adjacent precast inverted T–beams with tapered webs covered with a cast-in-place topping. Figure 8.1 and Figure 8.2 shows the plan view and elevation of the first implementation of this bridge system on US 360 near Richmond, Virginia. The US 360 Bridge is a two-span continuous bridge. Each span length, measured as the distance from the center of the intermediate support to the edge of the superstructure at the abutments is 43 ft. The west span is called span \(a\) and the east span is called span \(b\) (Figure 8.1 and Figure 8.2). Figure 8.3 shows the
construction phases of the US 360 Bridge. As can be seen from Figure 8.3 the adjacent precast inverted T-beam system is used to replace an existing bridge constructed with a combination of voided slab and T-beam structures. The width of the new completed bridge from outside edge of barrier to outside edge of barrier is 112 ft – 6 in. The width of bridge during Phase I is 36 ft. The live load test described later in this paper was conducted during the construction of Phase I of the bridge. When the live load test was conducted only the east barrier was installed on the bridge. The barrier consisted of seven precast units, each 12 ft long. The connection between the precast barrier units consisted of an un-bonded dowel type connection, which was intended to provide a loose mechanical connection between the precast units in case of a lateral impact. This connection did not provide any continuity for moment or any shear transfer between the precast units. The connection between the barrier and the cast-in-place topping featured post-installed anchors only on the traffic side.

Figure 8.4 shows the cross-sectional dimensions and reinforcing details for a typical composite transverse cross-section. The depth of the precast inverted T-beams is 18 in. and the depth of the cast-in-place concrete topping is 7.5 in. The transverse connection between the adjacent precast inverted T-beams features discrete embedded steel plates and welded bars (Figure 8.5). The embedded steel plates are located at the precast flanges and are spaced at 2 ft on center in the longitudinal direction. Each embedded steel plate is inclined to receive a field installed smooth connector rod, which is welded to each embedded steel plate with a partial penetration weld. In addition two No.4 bars are welded to the back of each embedded steel plate with a full penetration weld and are lapped with the No.4 bars coming from the other side. Detail B shows the top view of this connection and Section C-C shows a section through it. The non-shrink grout and the waterproofing membrane may be omitted provided that there is a 21.5 in. deep cast-in-place concrete topping over the longitudinal joints. This type of transverse connection is intended to provide a continuous tension tie in the transverse direction to resist the effects of transverse bending due to vehicular loads and emulate the behavior of a monolithic slab span bridge.

One of the key design parameters for this bridge type is the live load distribution factor (LLDF). For cast-in-place slab span bridges AASHTO uses the equivalent strip width method to determine how wide of a strip can be used to resist design live loads. Equations 1 and 2 can be used to determine the equivalent strip width for one design lane loaded and for two or more design lanes loaded, respectively. In equation 1 the strip width has been divided by 1.20 to account for the multiple presence effect.

\[
E = 10.0 + 5.0 \sqrt{L_1 W_1} \quad (1)
\]
\[
E = 84.0 + 1.44 \sqrt{L_1 W_1} \leq \frac{12.0 W}{N_L} \quad (2)
\]

where:
E = equivalent width (in.)
\( L_1 = \) modified span length taken equal to the lesser of the actual span or 60.0 (ft)
\[ W_1 = \text{modified edge-to-edge width of bridge taken to be equal to the lesser of the actual width or 60.0 for multilane loading, or 30.0 for single-lane loading.} \]
\[ W = \text{physical edge-to-edge width of bridge (ft)} \]
\[ N_L = \text{number of design lanes as specified in Article 3.6.1.1.1 of AASHTO}^2 \]

Alternatively, for precast solid, voided or cellular concrete boxes with shear keys and a cast-in-place concrete overlay AASHTO provides the equations shown in Table 8.1 for the calculation of live load distribution factors. The multiple presence factors have been included in the approximate equations for distribution factors provided in Table 8.1, for both single and multiple lanes loaded\(^2\). The equations are based on evaluation of several combinations of loaded lanes with their appropriate multiple presence factors and are intended to account for the worst case scenario\(^2\). The following notation applies to Table 8.1:

- \( N_b \) = number of beam, stingers or girders
- \( b \) = width of beam (in.)
- \( L \) = span of beam (ft)
- \( I \) = moment of inertia of the composite section (in\(^4\))
- \( J \) = polar moment of inertia (in\(^4\))
- \( g \) = distribution factor
- \( d_e \) = horizontal distance from the centerline of the exterior web of exterior beam at deck level to the interior edge of curb or traffic barrier (ft)
- \( e \) = correction factor

Table 8.2 provides a comparison of live load distribution factors calculated based on AASHTO’s methods for cast-in-place slab span and adjacent box structure systems. The method used for cast-in-place slab span systems does not distinguish between an interior and exterior strip. It also does not differentiate between live load distribution factors used for moment and shear. In contrast, the method used for adjacent box structure systems provides different equations for calculating live load distribution factors in interior and exterior beams, as well as for moment and shear. The live load distribution factors shown in Table 8.2 were calculated for Phase I of the US 360 Bridge as well as for the completed bridge. The differences in live load distribution factors between Phase I and the completed bridge are negligible and are primarily related to the difference in the number of beams and the width of the bridge. The live load distribution factors calculated for moment assuming an adjacent box structure system are similar to those calculated assuming a cast-in-place slab span system. However, there is a significant difference between the live load distribution factors for shear assuming an adjacent box girder system and those calculated assuming a cast-in-place slab span system. It should be noted that because the width of precast inverted T-beams is 72 in., it is outside the range of applicability of beam widths (30 ≤ \( b \) ≤ 60) for the adjacent box structure systems. However, LLDFs were calculated using for both systems
because they represent the available options to the engineer when designing a composite bridge system with adjacent precast inverted T-beams.

The purpose of the research presented in this paper is to calculate LLDFs for composite bridges constructed with adjacent precast inverted T-beams with tapered webs. LLDFs for moment are calculated by performing a live load test on Phase I of the US 360 bridge and measuring longitudinal strains at mid-span of the east span. The longitudinal strains at mid-width of each beam are divided by the sum of longitudinal strains to compute LLDF for moment in each girder. In addition, LLDFs for moment are computed using a finite element model of Phase I of the US 360 bridge using the same approach. After the results from the finite element model are validated based on field test results, the model is used to calculate LLDFs for shear. The measured and computed LLDFs are compared with each other and those calculated using the AASHTO’s methods to determine which method is best suited to be used in the design of this new bridge system.

Figure 8.1 Plan view of US 360 inverted T-beam Bridge
Figure 8.2 Elevation of US 360 inverted T-beam Bridge

Figure 8.3 Construction Phasing of US 360 Bridge
Figure 8.4 (a) Typical composite cross-section, (b) Typical reinforcing details

Figure 8.5 Transverse connection between adjacent precast inverted T-beams
Table 8.1 Live Load Distribution Factors for precast solid, voided or cellular concrete boxes with shear keys and a cast-in-place concrete overlay

<table>
<thead>
<tr>
<th>Distribution Factors</th>
<th>Range of Applicability</th>
</tr>
</thead>
<tbody>
<tr>
<td>Moment in interior beams</td>
<td></td>
</tr>
<tr>
<td>One Design Lane Loaded:</td>
<td></td>
</tr>
<tr>
<td>( g_{\text{interior}} = k \left( \frac{b}{33.3L} \right)^{0.5} \left( \frac{l}{J} \right)^{0.25} )</td>
<td></td>
</tr>
<tr>
<td>where: ( k = 2.5 \left( N_b \right)^{-0.2} \geq 1.5 )</td>
<td></td>
</tr>
<tr>
<td>Two or More Design Lanes Loaded:</td>
<td></td>
</tr>
<tr>
<td>( g_{\text{interior}} = k \left( \frac{b}{305} \right)^{0.6} \left( \frac{b}{12.0L} \right)^{0.2} \left( \frac{l}{J} \right)^{0.06} )</td>
<td></td>
</tr>
<tr>
<td>Moment in exterior beams</td>
<td></td>
</tr>
<tr>
<td>One Design Lane Loaded:</td>
<td></td>
</tr>
<tr>
<td>( g = e g_{\text{interior}} )</td>
<td>( d_e \leq 2.0 )</td>
</tr>
<tr>
<td>( e = 1.125 + \frac{d_e}{30} \geq 1.0 )</td>
<td></td>
</tr>
<tr>
<td>Two or More Design Lanes Loaded:</td>
<td></td>
</tr>
<tr>
<td>( g = e g_{\text{interior}} )</td>
<td></td>
</tr>
<tr>
<td>( e = 1.04 + \frac{d_e}{25} \geq 1.0 )</td>
<td></td>
</tr>
<tr>
<td>Shear in interior beams</td>
<td></td>
</tr>
<tr>
<td>One Design Lane Loaded:</td>
<td></td>
</tr>
<tr>
<td>( g_{\text{interior}} = \left( \frac{b}{130L} \right)^{0.15} \left( \frac{l}{J} \right)^{0.05} )</td>
<td></td>
</tr>
<tr>
<td>Two or More Design Lanes Loaded:</td>
<td></td>
</tr>
<tr>
<td>( g_{\text{interior}} = \left( \frac{b}{156} \right)^{0.4} \left( \frac{b}{12.0L} \right)^{0.1} \left( \frac{l}{J} \right)^{0.05} \left( \frac{b}{48} \right) )</td>
<td></td>
</tr>
<tr>
<td>( b \geq 48 \geq 1.0 )</td>
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<tr>
<td>Shear in exterior beams</td>
<td></td>
</tr>
<tr>
<td>One Design Lane Loaded:</td>
<td></td>
</tr>
<tr>
<td>( g = e g_{\text{interior}} )</td>
<td>( d_e \leq 2.0 )</td>
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<td>( e = 1.25 + \frac{d_e}{20} \geq 1.0 )</td>
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<tr>
<td>Two or More Design Lanes Loaded:</td>
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<tr>
<td>( g = e g_{\text{interior}} \left( \frac{48}{b} \right) )</td>
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<tr>
<td>( \frac{48}{b} \geq 1.0 )</td>
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</tr>
<tr>
<td>( e = 1 + \left( \frac{d_e + \frac{b}{12} - 12}{40} \right)^{0.5} \geq 1.0 )</td>
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Table 8.2 Comparison of Live Load Distribution Factors

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<thead>
<tr>
<th></th>
<th>Cast-in-place slab span system</th>
<th>Adjacent voided/box structures</th>
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<tr>
<td></td>
<td>Phase I</td>
<td></td>
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<tr>
<td>LLDF</td>
<td>0.52</td>
<td>Interior</td>
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<td>Moment</td>
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<td>Completed Bridge</td>
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<td></td>
<td>Moment</td>
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</table>

8.2 Previous Studies

French et al.\(^3\) combined numerical modeling with observations from a live load truck test on a bridge in Center City, MN along with load distribution tests of laboratory bridge specimens to determine the applicability of current live load distribution factors in AASHTO\(^4\) for cast-in-place slab-span bridges. The numerical modeling, field tests and laboratory tests were conducted for composite bridges constructed with adjacent precast inverted T-beams with straight webs and covered with a cast-in-place topping. In addition, the transverse connection between adjacent members featured transverse bars with 90° hooks that protruded from the sides of the precast webs and lapped with those extending from the adjacent member. A total of nine finite element models were created which featured bridges built with monolithic slab-spans and those built with precast inverted T-beams with longitudinal joints. Most of the bridge models featuring precast inverted T-beams incorporated the presence of a 3 in. longitudinal joint with the exception of one model which featured a 15 in. longitudinal crack, which extended to within 3 in. of the extreme compression fiber. The purpose of the model with an induced crack was to simulate the presence of a reflective crack.

The numerical models illustrated that the longitudinal curvatures measured in the inverted T-beam system with a reflective crack extending to within 3 in. of the extreme compression fiber and a tandem load greater than that which could be physically applied in the field, were only 84 percent of the longitudinal curvatures predicted using the AASHTO’s distribution factors for cast-in-place slab span bridges\(^3\). This observation suggested that Minnesota’s precast inverted T-beam system could reasonably and conservatively be designed using the current live load distribution factors for monolithic slab type bridges\(^3\).

In addition, the results from the live load truck test on the Center City Bridge suggested that the measured longitudinal curvatures were approximately three times smaller than those calculated using monolithic slab span equations\(^3\).

Laboratory tests were conducted on specimens with and without induced reflective cracking to investigate the capability of the system to transfer load from one beam to the adjacent beam. Little variation in the measured longitudinal curvatures was observed in the unloaded panels.
compared to loaded panels, which suggested that the load was effectively transferred across the longitudinal joint despite the presence and increase in the size of reflective cracking induced in/near the joint\(^3\).

In summary, the numerical and experimental studies in regards to live load distribution factors indicated that the precast inverted T-beam system with straight webs was well represented by monolithic finite element models, suggesting that the discontinuity at the precast joint did not significantly affect the load distribution characteristics of the system\(^3\). Also, the performance of large scale laboratory bridge specimens reinforced the notion that the system provided sufficient transverse load distribution, with and without the presence of reflective cracking near the joint region\(^3\).

As stated earlier, the studies performed by French et al.\(^3\) were conducted on the precast inverted T-beam system with straight precast webs. In addition, the connection between adjacent precast members featured extended bars that protruded from the precast webs and lapped with the extended bars from the adjacent members. The precast inverted T-beam system used in Virginia features tapered precast webs and the connections between adjacent precast members consist of discrete embedded steel plates and welded bars (Figure 8.5). Therefore, the purpose of the research presented in this paper is to determine the applicability of the available methods in AASHTO for calculating live load distribution factors for Virginia’s inverted T-beam system.

### 8.3 Preliminary Analytical Investigation

Before conducting the live load test on Phase I of the US 360 Bridge, a beam line model representing the two-span continuous beam was created in RISA\(^5\) with the purpose of determining the position of the controlled vehicle (VDOT truck) on the East span (span \(b\), see Figure 8.1 and Figure 8.2) that created the worst case positive bending moment at mid-span. The beam line model consisted of a prismatic beam section supported on rollers supports at the abutments and on two pin supports at the intermediate pier. The distance between the pin supports at the intermediate pier represented the distance between the centerlines of bearing of the precast inverted T-beams. The distance from the roller supports to the end of the beam line represents the distance from the centerline of bearing of the precast inverted T-beams to the edge of the superstructure. The moment envelope for the moving controlled vehicle is shown in Figure 8.6. The maximum positive moment was 237 ft-kips. The maximum positive moment at mid-span was approximately 210 ft-kips. To create the worst case positive moment at mid-span, the inner rear axle was positioned at mid-span (Figure 8.7). This truck position was used during field testing to create maximum longitudinal strains.

Table 8.3 provides a summary of service level design moments, the moments created by the controlled vehicle and the positive and negative cracking moments for a typical composite section. The design live load moments were based on a LLDF calculated assuming a cast-in-place slab span. The goal of the live load test was to create measurable longitudinal tensile strains at the bottom of each precast inverted T-beam but not exceed the service level design moments nor cause...
any cracking on the bridge. The maximum longitudinal mid-span tensile strain created due to the controlled vehicle on the east span calculated using the positive moment from the beam line model and the LLDF assuming a cast-in-place slab span is 31 microstrain. A preliminary test on the strain gages showed that they could report reasonably stable strain measurements within the range of ±1 microstrain. The maximum positive design moment due to superimposed loads is 396 ft-kips and the maximum positive moment due to the controlled vehicle is 123 ft-kips. In addition, the maximum negative design moment due to superimposed loads is 355 ft-kips and the maximum negative moment due to the controlled vehicle is 162 kips. The positive and negative cracking moments due to superimposed loads are 558 ft-kips and 423 ft-kips, respectively. As can be seen the moments created due to the controlled vehicle were smaller than the service level design moments and cracking moments.

**Table 8.3.** Service level moments for each composite beam

<table>
<thead>
<tr>
<th>LLDF = 0.52</th>
<th>Cracking Moment for each composite beam due to superimposed loads(ft-kips)</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Service Level Design Moments</strong> (ft-kips) (Service I)</td>
<td><strong>Max. Controlled Vehicle Moment</strong> (based on beam-line model) (ft-kips)</td>
</tr>
<tr>
<td>+M_{invT}</td>
<td>173</td>
</tr>
<tr>
<td>+M_{deck}</td>
<td>231</td>
</tr>
<tr>
<td>+M_{live}</td>
<td>336</td>
</tr>
<tr>
<td>-M_{live}</td>
<td>248</td>
</tr>
<tr>
<td>+M_{superD}</td>
<td>60</td>
</tr>
<tr>
<td>-M_{superD}</td>
<td>107</td>
</tr>
</tbody>
</table>

8.4 Field Testing

The live load test was conducted on Phase I of the US 360 Bridge (Figure 8.8) and was executed the day before this phase was opened to traffic. This approach was selected to avoid traffic control issues that would have had to be addressed in case the live load test was conducted.
after Phase I was opened to traffic. Each precast inverted T-beam was instrumented with surface mounted gages oriented in the longitudinal direction. The gages were manufactured by the Bridge Diagnostic Incorporated and will be referred to as BDI gages. These gages were placed at mid-span of the east span. Figure 8.9 illustrates the location of gages at the bottom of each precast inverted T-beam. With the exception of the exterior precast inverted T-beams, BDI gages were installed at mid-width of each beam and at 6 in. from the edge of the precast flange. For the two exterior beams only two gages were installed, one at mid-width and another one at 6 in. away from the edge of the precast flange.

The VDOT truck used for load testing was a three-axle dump truck that was loaded with gravel from a nearby rock quarry and weighed before it arrived at the bridge site. The truck weight was 56.2 kips when filled with gravel. The weight of the front axle was 19.7 kips and the combined weight of the rear axles was 36.5 kips. The distance between the front axle and the near most rear axle was 19 ft - 1 in. and the distance between the rear axles was 4ft - 4 in (Figure 8.11).

A total of four static truck tests were conducted. As stated earlier, the truck was positioned such that the inner rear axle aligned with the mid-span of the east span to create the maximum positive moment at mid-span, which is where the longitudinal strain gages were installed. Four transverse truck positions were investigated with the purpose of measuring longitudinal strains under each precast beam and investigating the distributions of these strains across the width of the bridge (Figure 8.10). The truck position for Test 1 was selected such that the tires were predominately over the precast webs and the center of the truck aligned with the center of the
bridge. The truck position for Test 2 was selected such that the wheel loads were predominately over the longitudinal joints. Figure 8.10 shows the location of the front wheels in the transverse cross-section of Phase I for each truck configuration. Because the width of the rear wheels was greater than the front wheels they covered a larger area of the precast webs in Test 1 and a larger area of the longitudinal joints in Test 2. The truck configuration for Test 3 was selected such that the south most precast beams would carry the majority of the load. During this test the truck was positioned such that the side of the wheels was aligned with inner edge of the barrier. Finally, the truck configuration for Test 4 was selected such that the north most precast beams carried the majority of the load. Tests 1 and 2 were intended to induce maximum longitudinal strains in the interior beams, whereas Tests 3 and 4 were intended to induce maximum longitudinal strains in the exterior beams. LLDFs were calculated by dividing the longitudinal strains at mid-width of each beam by the sum of the longitudinal strains at mid-width of all beams.

Figure 8.10 Location of controlled vehicle during Live Load Tests (Abutment is beyond)
8.5 Finite Element Analyses

After the live load test was completed two finite element models of Phase I of the US 360 Bridge were created with the purpose of determining whether the behavior of the bridge could be accurately simulated using finite element analyses and to investigate additional cases which were outside the scope of the live load test. The commercially available finite element software Abaqus\textsuperscript{6} was used to conduct the analyses. One of the finite element models had no barriers whereas the other had one continuous barrier. The actual behavior of the bridge was expected to be bracketed by the results from these two models because the actual barrier provided some additional stiffness along the east edge of the bridge but was not continuous. 3D solid elements were used to model the individual precast inverted T-beams, the cast-in-place topping and the continuity diaphragm. The bond between the precast surfaces and the cast-in-place topping was modeled as a perfect bond. In addition, the bond between the top of the cast-in-place topping and the bottom of the barrier was also modeled as a perfect bond in the model that featured the barrier. The boundary conditions were modeled as roller supports at the abutments and pin supports at the intermediate pier. Truck wheel loads matched those of the controlled vehicle used during the live load test. The wheel loads were applied as a uniformly distributed pressure over the tire prints illustrated in Figure 8.11. No dynamic load allowance was used to be consistent with the four static truck tests conducted in the field. Because no cracking was expected during the live load test a linear elastic analysis was conducted. The moduli of elasticity were based on the design compressive strengths and were calculated using the equations in AASHTO\textsuperscript{2}. Poisson’s ratio was taken as 0.2.

Both models were loaded with all four truck positions investigated during the live load test so that longitudinal strains recorded during the live load test could be compared with those obtained analytically. Longitudinal strains were recorded at the same locations where the BDI gages were installed on the bridge. Figure 8.12 shows longitudinal strain contours for Truck Position 4.

Because AASHTO’s method for calculating LLDFs in adjacent box structures yields different values for moment and shear, four additional cases were investigated in both models with the purpose of calculating live load distribution factors for shear. In these four additional cases the position of the controlled vehicle in the model in the longitudinal direction was chosen such that it caused the maximum shear at the critical section near the intermediate support (Figure 8.13). The locations of the truck in the transverse direction matched those used at mid-span. LLDFs for shear were calculated by dividing each beam reaction at the interior support by the sum of reactions. This additional investigation was performed to determine whether there is a significant difference between LLDFs calculated based on longitudinal strains at mid-span and those calculated based on the beam reaction at the intermediate support.
Figure 8.11 Truck axle weight and distances

Figure 8.12. Longitudinal strain contours due to Truck Position 4, (a) no barrier, (b) with one continuous barrier

Figure 8.13 Controlled vehicle position to create the maximum shear at the critical section

8.6 Results

The longitudinal strains recorded during the live load test and those computed from the two finite element models are shown in Figure 8.14 to Figure 8.17. During the live load test for truck position 1, BDI gage 14, 15 and 16 did not provide stable recordings. Accordingly, the data from
these three gages were considered unreliable. At the end of the first test the connection between these gages and the nodes was checked and it was ensured that they were properly working prior to conducting the second test. Figure 8.14 shows that the response of the bridge measured in terms of longitudinal strains during the first test is bracketed reasonably well by the response obtained from the finite element model without barriers (parapets) and the model with one continuous barrier. The maximum longitudinal strain recorded during the first test was about 16 microstrain, which is almost half of that calculated using the beam line model and the LLDF assuming a cast-in-place slab span system (31 microstrain), which suggests that it would be conservative to design the inverted T-beam system assuming cast-in-place slab span behavior. The position of truck in Test 1 was such that the center of the truck aligned with the mid-width of the bridge. As a result, the computed response of the bridge based on the model without the barrier was symmetric about the mid-width of the bridge. As expected, the presence of the barrier in the model, resulted in a computed response that was softer near the edge of the bridge without the barrier and stiffer near the edge with the barrier. Because of a lack of data from BDI gage 14, 15 and 16 such an observation could not be confirmed by in the measured response.

The results from Test 2 and Test 3 corroborate some of the observations made in Test 1 (Figure 8.15 and Figure 8.16). The measured response of the bridge in terms of longitudinal strains is bracketed reasonably well by the computed response obtained from the two finite element models. The maximum longitudinal strain measured in Test 2 and Test 3 was also 16 to 17 microstrain. In both Test 2 and 3 the influence of the barrier in the computed response was more noticeable towards the end of the bridge where the barrier was installed and less noticeable towards the opposite end of the bridge. The measured response of the bridge was closer to the computed response obtained from the model with the continuous barrier.

The measured response of the bridge during Test 4 was slightly stiffer than the computed response from the finite element models but still reasonably close considering the number of uncertainties that affect the actual behavior of the bridge. The highest measured longitudinal strain was about 17 microstrain, which was similar to those measured during the first three tests. The computed maximum longitudinal strains were slightly higher than the first three tests. As expected, these higher longitudinal strains were exhibited in the two exterior beams where the majority of the truck load was applied. In addition, there was not a significant difference between the computed responses with and without the barrier, especially towards the end of the bridge without a barrier. This observation was also expected because the precast beams in the vicinity of the barrier were the farthest from the controlled vehicle and they were not as influential in their load sharing capabilities as were the rest of the precast beams. Therefore, the presence or lack of a barrier did not make a marked difference. In addition, the computed higher longitudinal strains in the two exterior beams resulted because the two exterior beams had less opportunity to share the truck load compared to interior beams that have a larger number of neighboring beams.

These results suggest that the behavior of the bridge can be simulated reasonably well using a finite element model that is created based on the assumptions stipulated earlier. The measured and computed longitudinal strains at mid-width of the precast beams were used to calculate LLDFs
for moment using Equation 3. The LLDFs calculated using this approach were compared with those calculated based on AASHTO’s methods. LLDFs based on the measured response could not be calculated for Test 1 because, as stated earlier, the strain data from BDI 14, 15 and 16 were discarded as unreliable. Figure 8.18 through Figure 8.21 shows that the difference between the measured and computed LLDFs for moment is negligible. In Tests 1 and 2 the highest LLDF was 0.22, and the LLDFs calculated assuming cast-in-place slab span and adjacent box structure behavior were 0.39 and 0.30, respectively, for one design lane loaded. In Tests 3 and 4 the highest LLDFs were 0.24 and 0.28, respectively, which are still lower than the ones calculated based on ASSHTO’s methods but closer to the one calculated assuming adjacent box structure behavior. In both Tests 3 and 4 the truck was positioned such that the exterior beams were loaded the most. Therefore, it is expected that the LLDFs calculated based on these two tests would be higher because of the smaller load sharing opportunity present in the exterior beams.

However, in all four tests LLDFs were lower than those calculated assuming either a cast-in-place slab span or an adjacent box system for one design lane loaded.

Table 8.4 provides a summary of the LLDFs calculated based on the measured and computed behavior. As can be seen the design of the inverted T-beam system for moment can be conservatively based assuming a cast-in-place slab span or adjacent box structure behavior.

\[
LLDF_i = \frac{\varepsilon_i}{\sum_{i=1}^{6} \varepsilon_i}
\]

where:
\(\varepsilon_i\) = longitudinal strain at mid-width of each inverted T-beam
\(LLDF_i\) = Live Load Distribution Factor for each inverted T-beam

Because the behavior of the bridge superstructure system when subject to the controlled vehicle was linear elastic, the measured and computed longitudinal strains for truck positions 1+4, 2+4 and 3+4 were combined to account for the multiple presence effect. LLDFs for moment were computed using Equation 3 and are provided in

Table 8.5. In addition, the maximum measured and computed LLDFs for one design lane loaded were multiplied by the multiple presence factor of 1.2 given in AASHTO as an alternative way to account for the multiple presence effect. The highest LLDF based on the measured response was 0.30 and was calculated by multiplying the LLDF factor obtained from the measured response during Test 4, by the multiple presence factor of 1.2. LLDFs obtained by superimposing the results from individual truck tests were lower than those obtained based on single truck loading. Similarly, LLDFs calculated by superimposing the results from the computed responses of individual truck tests were lower than those computed based on single truck loading. The highest LLDF including the multiple presence effect for the measured and computed responses was 0.34, and was calculated by multiplying the LLDF calculated based on the computed response during Test 4, by the multiple presence factor 1.2. LLDFs calculated based on AASHTO’s methods assuming cast-in-place slab span and adjacent box structure behavior were 0.52 and 0.47, respectively.
Figure 8.14 Longitudinal strains in each girder – Truck Position 1

Figure 8.15 Longitudinal strains in each girder – Truck Position 2
Figure 8.16 Longitudinal strains in each girder – Truck Position 3

Figure 8.17 Longitudinal strains in each girder – Truck Position 4
Figure 8.18 LLDFs in each girder – Truck Position 1

Figure 8.19 LLDFs in each girder – Truck Position 2
Because LLDFs for moment including the multiple presence effect were lower than those calculated based on AASHTO’s methods, the design of composite bridges constructed with adjacent precast inverted T-beams with tapered webs can be conservatively based on LLDFs calculated assuming cast-in-place slab span or adjacent box structure behavior. Although LLDFs for moment were higher in cases where the exterior beams were loaded the most they were still lower than those calculated assuming cast-in-place slab span behavior. As stated earlier, the cast-
in-place slab span method does not distinguish between an interior strip and an exterior strip and is therefore attractive because of its simplicity. In addition, in a completed bridge constructed with adjacent precast inverted T-beams and cast-in-place topping, the exterior beams are rectangular precast beams and the cast-in-place topping is extended to match the exterior face of the rectangular beam. Accordingly, the presence of these rectangular beams and the extension of the cast-in-place topping will provide additional load sharing capabilities for the precast inverted T-beams closest to the edge.

Table 8.4. Live Load Distribution Factors (Moment) – One Design Lane Loaded

<table>
<thead>
<tr>
<th>One Design Lane Loaded</th>
<th>Adjacent Inverted T-beams with tapered webs</th>
<th>Cast-in-place slab spans</th>
<th>Adjacent voided/box structures</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Measured</td>
<td>Computed with Continuous parapet</td>
<td>Computed with no parapet</td>
</tr>
<tr>
<td>Truck Position 1</td>
<td>NC*</td>
<td>0.22</td>
<td>0.21</td>
</tr>
<tr>
<td>Truck Position 2</td>
<td>0.20</td>
<td>0.22</td>
<td>0.21</td>
</tr>
<tr>
<td>Truck Position 3</td>
<td>0.24</td>
<td>0.24</td>
<td>0.24</td>
</tr>
<tr>
<td>Truck Position 4</td>
<td>0.25</td>
<td>0.28</td>
<td>0.26</td>
</tr>
<tr>
<td><strong>One Lane Max</strong></td>
<td><strong>0.25</strong></td>
<td><strong>0.28</strong></td>
<td><strong>0.26</strong></td>
</tr>
</tbody>
</table>

*NC = not calculated because the last three gages recorded unreliable data

Table 8.5 Live Load Distribution Factors (Moment) – Including Multiple Presence Effect

<table>
<thead>
<tr>
<th>One Design Lane Loaded</th>
<th>Adjacent Inverted T-beams with tapered webs</th>
<th>Cast-in-place slab spans</th>
<th>Adjacent voided/box structures</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Measured</td>
<td>Computed with Continuous parapet</td>
<td>Computed with no parapet</td>
</tr>
<tr>
<td>Truck Position 1+4</td>
<td>NC*</td>
<td>0.22</td>
<td>0.20</td>
</tr>
<tr>
<td>Truck Position 2+4</td>
<td>0.19</td>
<td>0.21</td>
<td>0.19</td>
</tr>
<tr>
<td>Truck Position 3+4</td>
<td>0.18</td>
<td>0.19</td>
<td>0.17</td>
</tr>
<tr>
<td>*<em>One Lane Max.<em>Multiple Presence Factor</em></em></td>
<td><strong>0.30</strong></td>
<td><strong>0.34</strong></td>
<td><strong>0.31</strong></td>
</tr>
<tr>
<td><strong>Max.</strong></td>
<td><strong>0.30</strong></td>
<td><strong>0.34</strong></td>
<td><strong>0.31</strong></td>
</tr>
</tbody>
</table>
Because the simulated response obtained from the finite element models matched reasonably well with the measured response during the live load test, the finite element models were used to investigate additional cases in which the bridge was loaded with the controlled vehicle to create the maximum shear at the interior support. The investigation of these additional cases was required to determine whether different LLDFs for shear and moment are warranted as suggested by AASHTO’s method for adjacent box structure system. The beam line model in RISA\textsuperscript{5} was used to determine the location of the controlled vehicle in the longitudinal direction that generates the highest vertical shear at the interior support. Once the position of the truck in the longitudinal direction was established, the 3D finite elements models with and without the barrier were analyzed for the four truck positions used during the live load test. LLDFs for shear were calculated by dividing the interior reaction at each inverted T-beam by the sum of all interior reactions. Equation 4 presents the equation used for calculating LLDFs for shear. A summary of these LLDFs is provided in Table 8.6.

\[
LLDF_i = \frac{R_i}{\sum_{i=1}^{6} R_i} \tag{4}
\]

where:

- \( R_i \) = vertical reaction at each inverted T-beam
- \( LLDF_i \) = Live Load Distribution Factor for each inverted T-beam

The maximum computed LLDFs for shear for one design lane loaded were 0.35 and 0.34 based on the model without and with the continuous barrier, respectively. The controlling truck position was again truck position 4 in which the exterior beams are loaded the most. LLDFs for shear are higher than those computed for moment (0.28 and 0.26) but still lower than the LLDFs calculated based on AASHTO’s methods based on one design lane loaded.

To account for the multiple present effect the reactions obtained from individual truck loading were superimposed for truck positions 1+4, 2+4 and 3+4. In addition, the maximum LLDFs for shear computed for one design lane loaded were multiplied by the multiple presence factor of 1.2 given in AASHTO as an alternative way to account for the multiple presence effect. The maximum computed LLDFs for shear including the multiple presence effect were 0.42 and 0.41 based on the model without and with the continuous barrier, respectively. These LLDFs are also higher than the ones computed for moment (0.34 and 0.31), but still lower than the ones calculated using AASHTO equations. There is a noticeable difference between the two AASHTO methods for calculating LLDFs for shear. Because the method used for cast-in-place slab spans does not distinguish between LLDFs for shear and moment, the calculated LLDF for shear is the same as the one calculated for moment (0.52). If adjacent box structure behavior is assumed then the LLDF for shear is 0.86. These results suggest that the design of composite bridges constructed with adjacent precast inverted T-beams with tapered webs can be conservatively based on LLDFs calculated assuming cast-in-place slab span behavior. Even though the computed LLDFs for shear were slightly higher than those measured and computed for moment, this difference was not large.
enough to support AASHTO’s method for adjacent box structure systems. A shear design using LLDFs for shear assuming adjacent box structure behavior would lead to overly conservative results.

| Table 8.6 Live Load Distribution Factors (Shear) – One Design Lane Loaded |
|-----------------------------|-----------------------------|-----------------------------|
| **One Design Lane Loaded** | **LLDF**                    | **Adjacent voided/box**     |
|                            | **Adjacent Inverted T-beams with tapered webs** | **Cast-in-place slab spans** | **structures** |
|                            | **Computed with Continuous parapet** | **Computed with no parapet** |
| Truck Position 1           | 0.26                         | 0.26                        |                |
| Truck Position 2           | 0.27                         | 0.28                        |                |
| Truck Position 3           | 0.33                         | 0.33                        | 0.39           | 0.5             |
| Truck Position 4           | 0.35                         | 0.34                        |                |
| **One Lane Max**           | 0.35                         | 0.34                        |                |

| Table 8.7 Live Load Distribution Factors (Shear) – Including Multiple Presence Effect |
|-----------------------------------------------|-----------------------------|-----------------------------|
| **One Design Lane Loaded**                    | **LLDF**                    | **Adjacent voided/box**     |
| **Adjacent Inverted T-beams with tapered webs** | **Cast-in-place slab spans** | **structures** |
| **Computed with Continuous parapet** | **Computed with no parapet** | |
| Truck Position 1+4                           | 0.27                         | 0.26                        |                |
| Truck Position 2+4                           | 0.24                         | 0.24                        |                |
| Truck Position 3+4                           | 0.22                         | 0.21                        | 0.52           | 0.86            |
| **One Lane Max.*Multiple Presence Factor**   | 0.42                         | 0.41                        |                |
| **Max.**                                      | 0.42                         | 0.41                        |                |

8.7 Comparison of Phase I with the Completed Bridge and Other Bridges

The results presented in this paper were based on field tests and analytical work conducted on Phase I of the US 360 Bridge. It is reasonable to ask how these results can be related to the completed US 360 bridge and other bridges. Figure 8.22 shows transverse cross-sections of Phase I of the US 360 Bridge, the completed US 360 Bridge and Towlston Road Bridge constructed in northern Virginia. Towlston Road Bridge is the second application of the inverted T-beam system with tapered webs in the state of Virginia and features a simple span two lane bridge.

Phase I of the US 360 Bridge represents the least redundant superstructure from the standpoint of being able to distribute live loads with the adjacent members. The difference between Phase I and the completed US 360 Bridge is clear because the completed US 360 bridge features
a greater number of beams, which can help share some of the applied truck loads. The Towlston Road Bridge is still more redundant than Phase I of the US 360 Bridge, even though the number of precast inverted T-beams is the same, because it features two exterior rectangular precast beams, the cast-in-place topping is extended to match the exterior face of the rectangular beams and it includes two parapets. These additional features make the Towlston Road Bridge more redundant than Phase I of the US 360 Bridge. Because the Towlston Road Bridge features a two lane bridge it represents one of the narrowest applications in terms of bridge width for the inverted T-beam system.

Because other applications of the inverted T-beam system will feature bridges with at least two lanes, the conclusions drawn from the live load test conducted on Phase I of the US 360 Bridge and simulations using finite element models can be conservatively applied to those cases.

![Figures](a) Transverse cross-section of Phase I of the US 360 Bridge, (b) transverse cross-section of the completed US 360 Bridge, (c) transverse cross-section of Towlston road bridge.

**Figure 8.22** (a) Transverse cross-section of Phase I of the US 360 Bridge, (b) transverse cross-section of the completed US 360 Bridge, (c) transverse cross-section of Towlston road bridge.

### 8.8 Conclusions and Recommendations

The measured and computed responses of Phase I of the US 360 Bridge in terms of longitudinal strains and interior reactions resulted in LLDFs for shear and moment that were lower than those calculated based on AASHTO’s methods for cast-in-place slab spans and adjacent box structure systems.

While LLDFs for moment and shear calculated when the exterior beams were loaded the most were higher than when the interior beams were loaded the most, the calculated LLDFs were still lower than those calculated based on AASHTO’s methods. In addition, even though LLDFs for shear were higher than those calculated for moment they were still lower than those calculated assuming the cast-in-place slab span and adjacent box structure behavior. AASHTO’s method for cast-in-place slab spans is attractive because of its simplicity and provided an upper bound to the LLDFs calculated in this study. A shear design using LLDFs for shear assuming adjacent box structure behavior would lead to unnecessarily conservative results. As a result, it is recommended
that LLDFs for moment and shear should be based assuming cast-in-place slab span system behavior.

Because other applications of the inverted T-beam system will feature bridges with at least two lanes, the conclusions drawn from the live load test conducted on Phase I of the US 360 Bridge and simulations using finite element models can be conservatively applied to other applications.

References
6. RISA Technologies 26632 Towne Centre Drive, Suite 210, Foothill Ranch, California
7. ABAQUS (2011) `ABAQUS Documentation', Dassault Systèmes, Providence, RI, USA.
Chapter 9: Conclusions and Recommendations
Conclusions and Recommendations

9.1 Introduction

The primary goal of the work presented in this study was to come up with a bridge system that is relatively more resistant to traffic and environmental effects compared to the traditional systems used for short-to-medium-span bridges. A new bridge system, intended to reduce the extent and likelihood of reflective cracking specifically and deck cracking in general, was developed and consists of adjacent precast inverted T-beams with tapered webs covered with a cast-in-place concrete topping. Most of the issues that were investigated are related to the serviceability limit state and included studies on reflective cracking, stresses created due to time dependent and temperature effects and the potential for cracking at the end zones because of the transfer of the prestressing force. In addition, the investigations on composite action and live load distribution factors are related not only to the service limit state but also to the strength limit state and concluded in recommendations that could be used in the design of the proposed bridge system.

The load combinations provided in AASHTO1 require the synthesis of the effects of traffic and the environment for the design of bridges. These load combinations and load factors are provided in Table 9.1. Transverse bending effects due to live loads are more likely to cause longitudinal cracking in adjacent box structure systems, because the depth of the grouted shear keys is not as deep as the one provided by the cast-in-place topping in the precast inverted T-beam system. In addition, even if a crack were to develop above the longitudinal joint, the bent bars provided in the cast-in-place topping, will arrest the crack and control its width. Also, the presence of a horizontal interface between the precast flange and the cast-in-place topping helps better emulate monolithic action compared to adjacent box structures. The horizontal shear strength of the interface above the precast flange, and the combination of the tensile and shear strength of the interface at tapered precast web, along with mechanical interlock offered by the roughened surface, provided sufficient resistance against cracking at service level loads.

While the magnitude of the transverse bending moment due to live loads was not high enough to cause cracking, differential shrinkage caused tensile stresses in longitudinal sections 2 and 3 that were higher than the modulus of rupture of the cast-in-place concrete topping. When the tensile stresses due to differential shrinkage were combined with those created due to positive and negative temperature gradients the magnitude of tensile stresses in the cast-in-place topping further exceeded the modulus of rupture of concrete. It was demonstrated that the most critical section was longitudinal section 2. In the inverted T-beam system the extent of this section is much smaller compared to adjacent box structure systems. Accordingly, the extent of longitudinal cracking should be much smaller. Tapering the precast webs further reduces the region where section 2 applies, which is where the majority of the longitudinal cracks in the bridges in Minnesota were observed. In addition, even if cracks were to develop above the precast web, the combination of the bent and straight bars in the cast-in-place topping should be able to control the width of these cracks.
At the structure level it was demonstrated that any potential axial restraint at the abutments and restraint moments at the intermediate pier created tensile stresses in the longitudinal direction because of time dependent and temperature effects that far exceeded the moduli of rupture in the cast-in-place and precast components. However, as suggested in Chapter 4 these effects can be either eliminated or reduced by accommodating axial movement in the longitudinal direction and by choosing the age of continuity such that the competing effects of negative and positive restraint moments cancel each other out as much as possible.

When flexural stresses are aggregated to the ones created by time dependent and temperature gradients the magnitude of tensile stresses in the cast-in-place topping increases even further. However, the analyses presented in this study were based on the assumption that the investigated transverse and longitudinal sections will remain un-cracked. Once cracking initiates, the results obtained from analyses using un-cracked section properties cannot be superimposed. In addition, cracking and the subsequent straining of the reinforcing steel will accommodate to some extent the deformations imposed by time dependent and temperature effects and the stresses created because of them will diminish to some degree. To capture this behavior an analysis that accounts for cracking and includes material non-linearities in concrete and reinforcing steel should be performed. In lieu of performing such analysis, the individual effects of transverse bending, time dependent and temperature effects may be conservatively superimposed and enough reinforcing steel may be selected to resist the tensile stresses created in concrete without allowing the yielding of the reinforcing steel.

| Load Combination Limit State | DC | DD | DW | EH | EV | ES | EL | PS | CR | SH | LL | IM | CE | BR | PL | LS | WA | WS | WL | FR | TU | TG | SE | EQ | IC | CT | CV | Use One of These at a Time |
|-----------------------------|----|----|----|----|----|----|----|----|----|----|----|----|----|----|----|----|----|----|----|----|----|----|----|----|--------------------------|
| STRENGTH I (unless noted)   | γp | 1.75 | 1.00 | — | — | 1.00 | 0.50/1.20 | γTQ | γTQ | — | — | — | — | — | — | — | — | — | — | — | — | — | — | — | — | — | — | — |
| STRENGTH II                 | γp | 1.35 | 1.00 | — | — | 1.00 | 0.50/1.20 | γTQ | γTQ | — | — | — | — | — | — | — | — | — | — | — | — | — | — | — | — | — | — |
| STRENGTH III                | γp | — | 1.00 | 1.40 | — | 1.00 | 0.50/1.20 | γTQ | γTQ | — | — | — | — | — | — | — | — | — | — | — | — | — | — | — | — |
| STRENGTH IV                 | γp | 1.00 | — | — | — | 1.00 | 0.50/1.20 | — | — | — | — | — | — | — | — | — | — | — | — | — | — | — | — | — | — | — |
| STRENGTH V                  | γp | 1.35 | 1.00 | 0.40 | 1.0 | 1.00 | 0.50/1.20 | γTQ | γTQ | — | — | — | — | — | — | — | — | — | — | — | — | — | — | — | — |
| EXTREME EVENT I             | γp | γEQ | 1.00 | — | — | 1.00 | — | — | — | 1.00 | — | — | — | — | — | — | — | — | — | — | — | — | — | — | — | — | — | — |
| EXTREME EVENT II            | γp | 0.50 | 1.00 | — | — | 1.00 | — | — | — | 1.00 | 1.00 | 1.00 | 1.00 | 1.00 | 1.00 | 1.00 | 1.00 | 1.00 | 1.00 | 1.00 | 1.00 | 1.00 | 1.00 | 1.00 | 1.00 | 1.00 | 1.00 | 1.00 | 1.00 | 1.00 | 1.00 | 1.00 |
| SERVICE I                   | 1.00 | 1.00 | 1.00 | 0.30 | 1.0 | 1.00 | 1.00/1.20 | γTQ | γTQ | — | — | — | — | — | — | — | — | — | — | — | — | — | — | — | — | — | — | — | — | — | — | — | — | — |
| SERVICE II                  | 1.00 | 1.30 | 1.00 | — | — | 1.00 | 1.00/1.20 | — | — | — | — | — | — | — | — | — | — | — | — | — | — | — | — | — | — | — | — | — | — | — | — | — |
| SERVICE III                 | 1.00 | 0.80 | 1.00 | — | — | 1.00 | 1.00/1.20 | γTQ | γTQ | — | — | — | — | — | — | — | — | — | — | — | — | — | — | — | — | — | — | — | — | — | — | — | — | — |
| SERVICE IV                  | 1.00 | — | 1.00 | 0.70 | — | 1.00 | 1.00/1.20 | — | 1.0 | — | — | — | — | — | — | — | — | — | — | — | — | — | — | — | — | — | — | — | — | — | — | — | — |
| FATIGUE—LL, IM & CE ONLY    | — | 0.75 | — | — | — | — | — | — | — | — | — | — | — | — | — | — | — | — | — | — | — | — | — | — | — | — | — | — | — | — | — | — | — | — | — |

Table 9.1 Load combinations and load factors

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It was demonstrated that utilization of the proposed bridge system combined with a set of specifications that stress the importance of a deck mix with low shrinkage and high creep and a superstructure design that accommodates deformations due to time dependent and temperature effects should reduce the extent of longitudinal and transverse cracking manifested in the traditional bridge systems. Various methods for sizing reinforcing in the end zones were proposed and necessity of extended stirrups to achieve full composite action at all load level was investigated. Live load distribution factors were quantified analytically and via live load tests and recommendation for design were presented. A summary of all the conclusions and recommendations presented in each chapter is provided below for convenience.

9.2 Transverse Bending

While the traditional adjacent box girder and voided slab systems promote rapid construction they suffer from reflective cracking problems over the joints between the precast members. Reflective cracking is primarily a serviceability issue because it provides an avenue for water and deicing salts to get to the reinforcing steel more quickly and corrode it. Corrosion leads to expensive repairs and rehabilitations. However, when reflective cracking becomes excessive, then the assumptions about live load distribution factors used in design are called into question. The inverted T-beam concept is a useful and promising concept that delivers the advantages of jointless, cast-in-place, concrete construction while eliminating the need for installing formwork. It addresses the reflective cracking problem by providing a thicker cast-in-place topping over the joint between the precast members and by offering a horizontal interface in addition to the vertical/tapered interface.

9.2.1 Designing for Transverse Bending

- It is difficult to propose a simple design methodology for transverse bending that would be suitable for all applications involving precast inverted T-beams. For example the magnitude of the transverse tensile stress over the longitudinal joint depends on the thickness of the cast-in-place topping over the joint. The greater the thickness the lower the stress and vice versa. Accordingly, the most rational approach is to perform an analysis that can capture transverse plate bending behavior and quantify transverse bending moments (or tensile stresses) at service. Transverse bottom reinforcing can then be sized using allowable stress design principles by ignoring any contribution from concrete in tension. This was the approach adopted in sizing transverse reinforcing in Specimens 2-7. The transverse reinforcing sized in this manner will act as transverse load distribution reinforcing and reflective crack control reinforcing for details that are similar to the ones tested in Specimen No.2-7.
In lieu of performing such an analysis, if the ratio of the thickness of the cast-in-place topping over the joint to the thickness of the precast flange is similar to the one used in the US 360 Bridge described in this study then the transverse bottom reinforcing (in the cast-in-place topping and precast) can be sized based on Equation 1-Chapter 3. The areas of transverse steel calculated based on the transverse live load moment obtained from Finite Element Analysis and Equation 1-Chapter 3 were similar for the US 360 Bridge.

For transverse connection details that feature transverse bars with 90° hooks through the web of the precast inverted T-beam (Minnesota’s detail) reinforcing in the trough region may be based on recommendations by French et al.².

9.2.2 Cross-sectional Shape and Transverse Connection

Tapering the webs of the precast inverted T-beams helps reduce stress concentrations at the re-entrant corners in addition to providing a better resistance to normal tensile stresses in the transverse direction. Roughening the surfaces of the precast inverted T-beams that will be in contact with the cast-in-place topping appears to emulate monolithic action and provide the necessary integrity to prevent cracking due to service level loads as a result of plate bending in the transverse direction. All eight specimens, which featured combinations of two cross-sectional shapes and three connections, performed well and similarly at service load levels. The detail with the tapered webs and no mechanical connection is the least expensive one and the easiest to fabricate. This is due to the fact that it does not have any reinforcing steel protruding from the sides of the webs, which presents a forming challenge to the precaster. It also does not have any mechanical connection between the precast members, which might take additional time in the field and work against the concept of accelerating bridge construction.

9.3 Time Dependent and Temperature Effects

9.3.1 General

This study has demonstrated that time dependent effects can cause significant stresses in composite concrete bridge superstructures with precast inverted T-beams. To reduce the likelihood of excessive cracking recommendations are made in the following five frameworks. While these recommendations were based on the analysis of a two-span continuous bridge with precast inverted T-beams, most of them apply to most types of composite bridges.

- **Mix Design** – It is recommended that the concrete mix for the CIP topping possesses low shrinkage and high creep, as these properties help relax tensile stresses built-up due to time dependent effects. While it may be onerous to the supplier to conduct creep tests for various mix designs, it is relatively simple to collect short term shrinkage data (up to 28 days) on a variety of mixes. This will help create a database of shrinkage values for
various mixes that could be used in future projects if the specifications require a mix with certain shrinkage parameters. The engineer of record can use one of the shrinkage models available in AASHTO or ACI 209 “Guide for Modeling and Calculating Shrinkage and Creep in Hardened Concrete” to extrapolate the ultimate shrinkage strain based on strain data collected up to 28 days. A mix design for the cast-in-place concrete topping with low shrinkage and high creep is also recommended for composite bridges with steel girders. Because structural steel does not shrink or creep the effects of differential shrinkage and creep would be as pronounced as in the case considered in this study when the age of continuity for the precast beams is assumed to be 90 days.

- **Cross-Sectional Shape** - Inverted T-beam system reduces the tensile stresses in the CIP topping as a result of differential shrinkage compared to voided slabs and adjacent box girders by providing a smaller moment arm between the centroid of cast-in-place topping and that of the precast beam. In addition, the time dependent analysis performed at the cross-sectional level demonstrated that tensile stresses in the CIP topping were highest at section 2. Tapering the webs of the precast inverted T-beams helps reduce the area over which Section 2 applies and consequently reduces the likelihood of deck cracking. As a result it is recommended that precast inverted T-beams be used in the construction of short-to-medium-span bridges.

- **Mild steel** – While mild steel in the cast-in-place topping restrains its free shrinkage, tensile stresses in the CIP topping are not greatly influenced by the amount of mild steel in the topping. Mild steel needs to be provided to control the cracks and help distribute live loads in the transverse direction and its presence does not significantly contribute towards increasing the likelihood of deck cracking

- **Boundary Conditions** – In two-span continuous bridges it is essential accommodate axial movement in the longitudinal direction at the abutments to avoid tensile stresses in the deck due to differential shrinkage, negative temperature gradients and uniform decreases in temperature. As a result, it is recommended that bearings at the abutments are detailed such that they emulate a roller support.

- **Age of Continuity** – It is recommended that the age of continuity is selected such that the competing effects of positive and negative restraint moment cancel each other as much as possible. High positive restraint moments negate the effects of negative live load moments and may render a continuous design even more expensive than a design based on simply supported beams. High negative moments may create excessive cracking on the bridge decks and reduce the service life of bridges.

9.3.2 Comments

- It is important to note that the analysis performed at the cross-sectional level shows stress distributions that apply along the entire bridge superstructure. Because the precast inverted T-beams serve as stay-in-place forms for the cast-in-place topping, there are no
stresses in the cast-in-place topping due to its self-weight. Accordingly, any tensile stresses created in the topping due to time dependent effects could not be counter balanced by any compressive stresses due to the self-weight of the topping as those stresses apply only to the precast beams. Therefore, if the tensile stresses in the deck are higher than its rupture stress, cracks could potentially develop in the entire top surface of the bridge in the longitudinal and transverse directions.

- Similarly, tensile stresses developed as a result of axial restraints at the abutments in the longitudinal direction due to differential shrinkage, negative temperature gradient and a uniform decrease in temperature apply not only to the entire bridge superstructure but are also constant throughout the depth of the composite cross-section of the bridge. These high tensile stresses have the potential to develop full depth transverse cracks. In addition to the obvious serviceability and durability problems that these high tensile stresses can create, full depth cracks in regions of small moment can cause reductions in shear strength.

- Tensile stresses developed as a result of moment restraint due to differential shrinkage and negative/positive temperature gradients are maximum at the intermediate support and reduce linearly towards the abutments (for a two-span continuous bridge).

In summary, time dependent effects can cause stresses that are higher than the ones created due to mechanical loads and need to be considered in the analysis and design of bridges to prolong their longevity.

**9.4 Study on Seven Deck Mixes**

Reducing the likelihood of bridge deck cracking is essential for increasing the longevity of bridges. This study has demonstrated that the effects of differential shrinkage and creep in composite bridges can cause tensile stresses in the deck that exceed its tensile strength. To reduce the likelihood of excessive cracking it is recommended that the concrete mix for the cast-in-place deck possesses low shrinkage and high creep properties. Low free shrinkage reduces the tensile stresses developed due to restrained differential shrinkage and high creep helps relax any tensile stresses that may develop. In addition, the short term properties that will help reduce the extent of cracking due to differential shrinkage include a mix with high tensile strength and low modulus of elasticity. Because it is difficult to find a concrete mix that embodies all the aforementioned long term and short term properties, priority should be given the mix with the lowest shrinkage because it is the free shrinkage of the deck that serves as a catalyst for the creation of tensile stresses in the cast-in-place topping. In addition, sensitivity studies can be performed for a given structure to determine the influence of the short and long term properties of the concrete materials on the structural effects of shrinkage and creep. While a concrete mix with high creep is advocated for the cast-in-place deck to help alleviate tensile stresses due to differential shrinkage, long term deflections should be checked to ensure that they are within the
allowable limits. Although the investigation described in this study was focused on short-to-medium-span composite concrete bridges, the benefits of a cast-in-place deck with low shrinkage and high creep apply also to composite steel bridges in which the free shrinkage of the deck is restrained by the steel girder.

Seven deck mixes developed by the researchers at Virginia Center for Transportation Innovation and Research were investigated with the goal of identifying a mix with low shrinkage and high creep. Drying shrinkage strains recorded in the shrinkage prisms showed that the mix with normal weight coarse aggregates and saturated light weight fine aggregates exhibited the lowest shrinkage strain. In addition, data obtained from the creep test showed that the mix with normal weight coarse aggregates and saturated light weight fine aggregates also exhibited the highest creep coefficient. It should be noted that the tests performed on the deck mixes described in this investigation were performed only on one batch for each mix. It was outside the scope of study to investigate the repeatability and consistency of the results obtained from each mix.

Three composite bridge systems intended for short-to-medium-span bridges were investigated with the purpose of identifying the system that is most resistant against time dependent effects. The voided slab system is the traditional system and the inverted T-beam system with straight and tapered precast webs are relatively new bridge systems used in Minnesota and Virginia, respectively. The precast inverted T-beam system with tapered webs minimizes the width of the interface between the precast web and the cast-in-place topping, which is the most vulnerable interface to longitudinal and transverse cracking as a result of differential shrinkage. While the magnitude of transverse normal stresses was similar in all three bridge systems, the inverted T-beam with the tapered webs exhibited lower longitudinal tensile stresses at the interface between the precast web and cast-in-place topping, which is desirable for reducing the likelihood of transverse cracking.

The results presented in this investigation suggest that the combination of the precast inverted T-beam system with tapered webs and the implementation of a cast-in-place topping mix with normal weight coarse aggregates and saturated light weight fine aggregates should help increase the resistance of short-to-medium-span bridges against time depend effects.

9.5 End Stresses

Precast inverted T-beams with tapered webs present a unique shape that is being implemented for the first time in Virginia in the construction of the US 360 Bridge near Richmond. Properly accounting for stresses created in the end zones as a result of the diffusion on the prestressing force from the strands into the surrounding concrete is essential to preclude excessive cracking that may lead to strength and serviceability concerns. While 3D linear elastic finite element analyses were employed in this study to gain an understanding of the stresses that develop at the end zones of precast inverted T-beam in the vertical and horizontal planes, such analysis may not always be a viable option in a design office. Accordingly, the following
conclusions and recommendations are intended to aid engineers when sizing reinforcing in the pre-tensioned anchorage zones of precast inverted T-beams with tapered webs.

Vertical Plane:

- Although this study did not include an exhaustive array of various precast beam depths, it can be concluded that precast inverted T-beams 18 in. deep or less experience spalling and bursting stresses that are lower than the modulus of rupture of concrete at transfer. As a result, theoretically no vertical reinforcing is required to resist these stresses. The recommendations provided in NCHRP Report\(^2\) corroborate this conclusion and may be used to evaluate the need for such reinforcing. The application of AASHTO Provisions\(^1\) for pre-tensioned anchorage zones in the vertical plane of precast inverted T-beams with tapered webs that are 18 in. deep or less, provides a conservative alternative. If such vertical reinforcing is provided, it should be placed with a distance equal to \(h/4\) from the end of the beam, where \(h\) is the depth of the precast member, or as close to end face as practically possible, because spalling stresses at the end face were the dominating type of tensile stresses in terms of magnitude.

- While the 18 in. depth for precast inverted T-beams with tapered webs does not represent the dividing line at which spalling stresses at the end faces exceed the modulus of rupture of concrete, it can be conservatively stated that the application of AASHTO provisions for beams that are 18 in. deep or greater is also conservative. Similarly, for the beams in this bracket, the vertical reinforcing at the end zones should be placed with a distance equal to \(h/4\) from the end of the beam, where \(h\) is the depth of the precast member, or as close to end face as practically possible, because the magnitude of vertical tensile stresses at the end zones diminishes quickly past the first few inches from the end face. Vertical steel at the end zones can consist of stirrups as well as the vertical component of the AASHTO required confinement steel.

- As an alternative to AASHTO provisions and NCHRP recommendations, vertical reinforcing in the end zones can be calculated based on strut and tie model V1. Compared to the 4% AASHTO\(^1\) rule, strut and tie model V1 leads to designs that are more economical and creates less congestion in the end zones. However, experimental testing is required to validate the suitability of this model for sizing vertical reinforcing in the end zones, especially for cases when spalling stresses exceed the modulus of rupture of concrete at transfer.

Horizontal Plane:

- In none of the cases considered in this study did the bursting stresses exceed the modulus of rupture of concrete at transfer. Accordingly, no reinforcing is required in the horizontal plane to resist these stresses. However, the application of the 4% rule presented in AASHTO for sizing reinforcing in the horizontal plane is a conservative alternative. If such reinforcing is provided, it should be placed within a distance \(h\) from the end of the
precast flange. The AASHTO required confinement steel can be used for this purpose given that it needs to be provided for a distance up to $1.5d$ from the end of the member. In addition, the straight transverse bars in the precast flanges provided to resist the weight of wet concrete and transverse bending moments due to live loads can be used to resist the bursting force based on the 4% rule.

- Alternatively, horizontal reinforcing at the end zones can be sized based on strut and tie model H3. The utilization of this model in design presents an even more conservative approach compared to the 4% AASHTO\textsuperscript{1} rule. If this model is selected, then the horizontal reinforcing can be distributed uniformly throughout the disturbed region.

### 9.6 Composite Action

The analytical and experimental results presented in this study lead to the following conclusions and recommendations:

- The full scale test described in this investigation has demonstrated that full composite behavior is assured not only at service and strength level design loads, but also up to flexural failure in such a uniquely shaped composite system.

- The presence of extended stirrups in one half of the span did not result in any differences in behavior between the two halves of the span. Because the composite inverted T-beam bridge system with tapered webs and cast-in-place topping features a broad contact surface between the precast and cast-in-place components, adequate horizontal shear resistance along the interface to develop the nominal moment capacity of the composite section can be provided solely by the natural adhesion and friction between the two components.

- The composite bridge system described in this investigation and used in the construction of the US 360 Bridge was able to develop at least a horizontal shear stress equal to 120 psi without the presence of extended stirrups. The failure mode of the composite section was a flexural failure. Because Equation 3 is provided in AASHTO as a reasonable approximation of the horizontal shear stress, these results suggest that the design for horizontal shear of adjacent precast inverted T-beams with tapered webs and cast-in-place topping can be confidently based on the following cohesion and friction factors:

$$c = 120 \quad \mu = 1.0 \quad K1 = 0.2 \quad K2 = 0.8$$

Because of the flexural failure mode, the 120 psi horizontal shear stress representing the recommended cohesion factor does not constitute the maximum horizontal shear stress that can be developed in the composite inverted T-beam system described herein. The flexural failure of the composite beam prevented it from achieving higher horizontal shear stresses at the interfaces such as those achieved by French et al.\textsuperscript{11} (135 psi) in their experiments.
Roughening the tapered webs and the tops of the precast flanges in the longitudinal direction while providing a transverse rake finish only at the top of the precast web appears to provide adequate horizontal shear resistance in the longitudinal direction to resist at least a horizontal shear stress of 120 psi without the presence of extended stirrups. The minimum reinforcement requirements for cases in which the horizontal shear stress is smaller than 120 psi and the precast surfaces are roughened as described above can be waived for composite bridges consisting of adjacent precast inverted T-beams with tapered webs and cast-in-place topping.

9.7 Live Load Distribution Factors

The measured and computed responses of Phase I of the US 360 Bridge in terms of longitudinal strains and interior reactions resulted in LLDFs for shear and moment that were lower than those calculated based on AASHTO’s methods for cast-in-place slab spans and adjacent box structure systems.

While LLDFs for moment and shear calculated when the exterior beams were loaded the most were higher than when the interior beams were loaded the most, the calculated LLDFs were still lower than those calculated based on AASHTO’s methods. In addition, even though LLDFs for shear were higher than those calculated for moment they were still lower than those calculated assuming the cast-in-place slab span behavior. AASHTO’s method for cast-in-place slab spans is attractive because of its simplicity and provided an upper bound to the LLDFs calculated in this study. A shear design using LLDFs for shear assuming adjacent box structure behavior would lead to unnecessarily conservative results. As a result, it is recommended that LLDFs for moment and shear should be based assuming cast-in-place slab span system behavior.

Because other applications of the inverted T-beam system will feature bridges with at least two lanes, the conclusions drawn from the live load test conducted on Phase I of the US 360 Bridge and simulations using finite element models can be conservatively applied to other applications.

9.8 Recommendations for Future Work

The work presented in this study was primarily centered on the US 360 Bridge provided that it was the first implementation of the inverted T-beam system with tapered precast webs. Accordingly, to validate the generality of some of the recommendations presented herein additional investigations are required. For example, while Equation 1 in Chapter 3 estimated fairly well the demand for transverse steel because of transverse bending, the applicability of this equation for different bridges featuring various span lengths, bridge widths, number of spans and boundary conditions should be investigated to validate its generality. In addition, the sub-assemblage specimens described in Chapter 3 were primarily tested under monotonic loading.
They were not subject to fatigue loading because the effects of transverse bending alone did not cause cracking. However, if a crack were to initiate above the longitudinal joints as a result of time dependent effects, then the behavior of the cracked sub-assemblage specimens under fatigue loading would be of interest.

In addition, while it was essential to raise awareness that time dependent effects have to potential to cause tensile stresses in the cast-in-place and precast components that far exceed those created by superimposed dead and live loads, some of the assumptions made during the time dependent analyses need to be investigated. For example, one of the assumptions was that tensile creep is the same as compressive creep. In a study performed by Nelson\cite{3}, it was demonstrated that tensile creep is higher than compressive creep. A higher tensile creep should help alleviate further the tensile stresses created as a result of differential shrinkage. In addition, it was further assumed that flexural creep was the same as compressive creep. If tensile creep is higher than compressive creep than it can be deducted that flexural creep should higher than compressive creep but lower than tensile creep. Experimental investigations are needed to quantify creep coefficients for compression, tension and flexure so that than can be incorporated in the age adjusted effective modulus method employed in this study. Also, viscoelastic models should be investigated using finite element analysis to capture the effects of creep so that the validity of the age adjusted effective modulus method can be examined. In Chapter 4 time dependent and temperature effects were examined at the cross-sectional level for one transverse and two longitudinal cross-sections and at a structure level in the longitudinal direction. As a result, their effects at the structure level in the transverse direction were not examined. This can be done by using 3D finite elements models such as the one used to investigate the effects of transverse bending and by simulating the time dependent and temperature effects. For example, differential shrinkage can be simulated by subjecting the cast-in-place topping to a uniform change in temperature. Temperature gradients can be simulated by varying the temperature field along the depth of the composite cross-section. Uniform changes in temperature can be simulated by subjecting the entire composite cross-section to a uniform change in temperature. Such an investigation will also reveal how the various longitudinal cross-sections will interact with each other. In addition, the influence of the boundary conditions in the longitudinal and transverse direction can be investigated. For example French et al.\cite{2} suggested that dowels in block-outs away from the longitudinal bridge center should be minimized as these can add restraint to shrinkage which may induce cracking in the deck, and that dowels at piers should be wrapped with polystyrene sleeves to reduce restraint-induced cracking.

The study in Chapter 6 examined various ways of modeling the prestressing force. While the presence of the transfer length was accounted for, the Hoyer effect (the tendency of the prestressing strands to assume their original diameter) was not captured because the prestressing strands were modeled as truss elements. In addition the effect of strand slip on the magnitude of stresses in the end zones was not explored. Burgueno and Sun\cite{4} suggest that the Hoyer effect can be captured by modeling the prestressing strands as 3D deformable solid elements and provide details for how to account for bond slip.
Finally, the proposed bridge system is intended for short to medium span bridges with spans ranging from 20-65 feet. To make the system viable for longer spans it should be made lighter. This can be accomplished by utilizing lightweight concrete for both the precast beams and the cast-in-place topping. In addition, voids could be introduced in the precast beams to make them lighter. To increase the confidence level in implementing this change some of the issues investigated in this study will have to be re-examined. For example the presence of voids should affect the magnitude and distribution of stresses in the end zone because of the transfer of the prestressing force. Although the concrete mixes with light weight coarse aggregates investigated in this study exhibited the highest drying shrinkage strains, other lightweight mixes can be developed that possess more favorable long term properties.

References