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Modified NEXT-D Beam Bridge --- Experimental and Simulation Evaluation of Shear Key Performance and Development of a Design Strategy of the Superstructure

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MODIFIED NEXT-D BEAM BRIDGE — EXPERIMENTAL AND SIMULATION EVALUATION OF SHEAR KEY PERFORMANCE AND DEVELOPMENT OF A DESIGN STRATEGY OF THE SUPERSTRUCTURE

A Dissertation
Presented to
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Doctor of Philosophy
Civil Engineering

by
Huan Sheng
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Accepted by:
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Abstract

In South Carolina, the Northeast Extreme Tee (NEXT) D beam system, proposed by the Precast/Prestressed Concrete Institute Northeast (PCINE), was identified to be a promising alternative to the cast-in-place (CIP) flat bridge that has been widely used in the state. In order to accommodate a span length from 22 ft to 40 ft, as desired by South Carolina Department of Transportation (SCDOT), this original beam cross section developed for longer span length was scaled down to a 6-ft-wide cross section (NEXT-D6) and a 8-ft-wide cross section (NEXT-D8). The modified beam has U-bars extending from the edges of the flanges of the precast beam into field cast joints. Because it is a new system there is uncertainty relative to the joint durability, as seen in other precast systems, and also appropriate design strategies. As such, both experimental work and analytical studies were carried out to examine the joint performance and provide a deeper understanding of the load distribution of this system. A final design guideline was also provided to aid in the future design.

In the experiment phase, three different joint material combinations—traditional grout extended with PVA fibers and ultra-high performance concrete (UHPC) extended with either steel fibers or PVA fibers—were examined under both static and fatigue scenarios. In each test, the joint was examined either under a high-moment or high-shear loading configuration, so that the shear key performance under a wide range of moment-to-shear ratios can be known. Shear key stiffness obtained from
the static test was used to determine the fatigue demand using finite element analysis, and the subsequent fatigue load was exerted in the fatigue test. In addition to specimen tests, material tests were also conducted to study the basic material properties and bonding performances. Fatigue tests showed that all of the material combinations gave satisfactory long-term shear key performances. However, static tests and material tests showed that the UHPC combinations gave much better bond performance than the traditional grout combination in terms of strength and failure mode. Typically a substrate failure was observed for the UHPC combinations, and bond failure was observed for the standard grout combination.

In the bridge design phase, a primary analysis of demand distribution for the NEXT-D beam system was conducted using finite element analysis. In this step, the shear key stiffness matrix is the key input for demand determination, which includes many stiffness terms that were not available from the experiment. As such a shear key finite element model was built and calibrated using experimental results, from which, the remaining stiffness terms were either directly obtained by setting various boundary conditions or derived using beam analysis. This matrix was determined for each material combination and used for a demand distribution analysis. Finally a general design guideline was provided for the NEXT-D beam system with span lengths between 22 ft. and 40 ft., and beam widths between 6 ft. and 8 ft. For beam design, it is recommended to use the live load distribution factors provided by American Association of State Highway and Transportation Officials (AASHTO) for cross section I as specified in AASHTO LRFD Table 4.6.2.2.1-1. For deck design, a four-step procedure was developed to be used together with the strip method to determine the design demands on a 1 ft. strip in Strength I and Service I limit states.
Acknowledgments

Looking back to the fall of 2010, when I first came to the United States, there was a lot of knowledge I did not even know that I did not know. Studying in the US for these four years is a big gift for me. Time suddenly became so compact. There were so many things that I needed to do and needed to learn. I was like a kid beginning to learn how to talk, walk, and run. The beginning is usually the most difficult and frustrating. It is like a hone stone which improves one’s stamina, sharpens one’s mind, and firms one’s heart. With time, my knowledge grew from linear to nonlinear, from statics to dynamics, and from one-dimensional to three-dimensional. Right now when I think of those hard beginnings, I felt that they were all necessary steps for my growth, not only in knowledge, but also in personality, which are what I desired.

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Chapter 1

Introduction

1.1 Problem Description

Cast-in-place flat slab bridges have been widely used in the State of South Carolina for short span bridges. These bridges have historically performed very well including notable durability and has no restrictions on the service level of traffic they are allowed to carry. This positive performance is in large part the reason for its past prolific use in the State. However, the use of such a system comes at the cost of a lengthy construction time. The flat slab bridge requires a significant amount of on-site labor compared with other bridge systems available. Not only does this increase the cost associated with the direct construction of the bridge but it also often requires a lengthy and costly disruption of traffic flow in the area. Many state Departments of Transportation (DOTs) have been desirous to address the issue of construction time by exploring and developing Accelerated Bridge Construction (ABC) techniques. Under the umbrella of ABC, the use of precast concrete systems has proven invaluable.
The South Carolina Department of Transportation (SCDOT) uses adjacent precast hollow core slab bridges on low volume secondary roads as an alternative to the robust cast-in-place concrete flat slab bridge. These sections are used on 20 to 70 foot spans. They are low cost, easy and relatively quick to install. However, there are some drawbacks. Longitudinal reflective cracks tend to form in the asphalt overlay along the joints between adjacent sections. Transverse cracks also develop at the abutments and bents where no continuity is provided between adjacent spans. These cracks are problematic because they allow water to seep between the members and corrode the hidden reinforcement and prestressing steel. The longitudinal cracks also signify the possible break down of the load transferring capability of the shear key. Since the hollow core beams are designed to take only a fraction of the wheel line load, premature failure of the beam is possible without this load sharing action.

Since the precast hollow-core bridge system has limitations on its use, the SCDOT is seeking an alternative to the flat slab bridges which has a faster erection time but does not have the same short-comings that the hollow-core bridge system exhibits. It is desirable that the alternative system will:

- eliminate or minimize longitudinal reflective cracking;
- lower, if possible, the initial price and maintenance costs;
- have a shorter erection time than cast-in-place (CIP) slabs;
- provide a longer service life than the current precast hollow-core slabs;
- be available for use on all routes (no Annual Daily Traffic (ADT) or National Highway System (NHS) restriction);
- be designed so it does not need an asphalt or concrete overlay.

Through in-depth literature searches, surveys, interviews, and a mini-workshop, the NEXT-D beam system, originally proposed by PCI Northeast (PCINE), was iden-
tified (Deery, 2010), and it was deemed to be the most viable candidate by a SCDOT steering committee.

However, before the modified NEXT beam bridge is implemented in the state, the performance of the joint used in this bridge system needs to be well understood; The fatigue behavior of the joint, especially needs to be explored before this bridge can be confidently put under high traffic volume. Moreover, the live load distribution factors and transverse demand distribution of bridges constructed with NEXT-D beams need to be investigated to facilitate design.

1.2 Objective and Scope of Research

The overall objective of this research is to: 1) validate the selected shear key configuration with the chosen shear key material in terms of both short-term and long-term performance; and 2) provide a guideline to the design of a NEXT-D beam system with span length from 22 ft to 40 ft, including appropriate load modeling and analysis. Accordingly, this study is mainly composed of an experimental component, an analytical component and a design component.

1.3 Outline of Dissertation

This dissertation is organized into seven chapters with the following contents:

Chapter 2 provides a background literature review concerning an overview of ABC, the development of joints connecting deck panels, and the development of double-tee and NEXT-D beam sections.
Chapter 3 documents the whole performance evaluation procedure of the longitudinal joint from joint material selection to an experimental evaluation of the joint in terms of both short-term and long-term performances.

Chapter 4 presents and analyzes the experimental results from short-term and long-term tests.

Chapter 5 presents a set of analytical studies, used in conjunction with the experimental program.

Chapter 6 deals with the design of the NEXT-D beam bridge. In addition to providing some typical design details for the bridge, guidelines are provided pertaining to design approaches for the system.

Chapter 7 provides a brief description of overall findings, recommendations and future work.
Chapter 2

Background

2.1 Accelerated Bridge Construction (ABC)

According to the Federal Highway Administration (FHWA), ‘ABC is a paradigm shift in the project planning and procurement approach where the need to minimize mobility impacts which occur due to onsite construction activities are elevated to a higher priority’ (Administration, 2013). Therefore the concept of ABC can be applied to each and every aspect from initial bridge design to final realization of the new or rehabilitated bridge. Various concrete techniques are involved in ABC including cast-in-place (CIP) and reinforced/post-tensioned concrete, and precast, reinforced/pre-tensioned/post-tensioned concrete (Ralls, 2007). Structurally speaking, this concept is most commonly reflected in the assembling of parts of or the whole structure with precast components with the aid of prestressing technique and/or high performance materials. For the substructure, a typical example is the use of precast concrete abutment segments post-tensioned together. Likewise for the superstructure, it is commonly seen that concrete deck panels are connected by transverse and/or longitudinal joints.
The advantages of ABC are detailed by Fowler and Eng (2006). As indicated by its name, the most characteristic property of ABC is fast construction speed due to reduced on-site formwork and cure time as typically required by CIP construction. This accelerated speed leads to increased safety for both construction crews and motorists and less negative impact on the surrounding environment and business. In addition, ABC also means improved element quality and durability due to a controlled curing environment, and the resulting reduced long-term maintenance cost. Moreover, ABC provides a very convenient construction strategy in regions where CIP is not readily available or the weather is unfavorable for concrete curing. In such a situation, ABC is useful due to the fast installation and pre-shrinkage of precast concrete (Nottingham, 1996). Finally, fabrication standardization and familiarity of the contractors with this technique can bring long-lasting economic benefits.

ABC is a necessary trend. According to FHWA, almost a quarter of ‘the Nation’s 600,000 bridges require rehabilitation, repair, or total replacement’ (Administration, 2013). Safety issues and economic impacts are the driving factors for using ABC to minimize traffic disruption.

The future bridge, as pointed out by Chase (2005), is expected to have a service life of up to 100 years and reduce maintenance requirements. A big barrier that limits the long-term durability of ABC is the joint between prefabricated components, such as the connection between precast decks as pointed out by Tang (2006) and Slavis (1982). Once this problem is solved, the above goals should be readily achieved. However, the potential benefits that can be brought by this technique are not guaranteed. It is important to make a rational decision as far as whether ABC is suitable for a particular case. A general guideline for sound decision making can be found in Tang (2006).
2.2 Joint Durability

Here a joint refers to either the transverse or longitudinal connection between full-depth deck panels or beams. The function of a joint is not only to transfer force (tension, shear, or/and moment) to the adjacent member, but also to protect the reinforcement and/or strands within the joint from corrosion.

Durability of a joint is the main issue that has been frequently encountered in ABC. It significantly reduces the long-term serviceability of a bridge and increases maintenance cost. This issue is initiated mainly from cracking at the joint interface or within the joint itself, which exposes the hidden rebar and/or prestressing strands to salt-laden water and/or other harmful elements. These rebar and/or strands, if not well protected, get corroded, resulting in the loss of load transfer capacity of the joint. For a multibeam superstructure, this phenomenon can disturb the load distribution originally assumed for design. As such, certain members, under much higher loads than designed, can experience damage, increasing maintenance cost and imposing a potential safety threat to the motorists. For a superstructure with full-depth panels resting on steel stringer beams, the longitudinal joint is usually located above the beam. Leakage from the joint can cause corrosion and damage of the supporting beam.

This joint durability issue is so common that researchers have been trying to identify the reasons and search for effective solutions to it. Depending on the structural type of the superstructure, approaches have been made mainly focusing on joint shape and size, joint materials, joint reinforcement, and bonding interface. It is very important for the superstructure to achieve a near monolithic behavior in the proximity of the joints. Studies show that the monolithic behavior like that of CIP slabs, if achieved, can significantly improve the traffic volume capacity of a
bridge (Roberts, 2010). In the following sections, the efforts made by researchers are reviewed based on the superstructure type including full-depth precast concrete deck panels and prestressed concrete multibeam superstructures, which is further subdivided into stemmed beams and box beams.

2.2.1 Joints connecting full-depth panels

According to a survey by Issa et al. (1995), full-depth precast concrete panels became popularly used in North America since 1970. It was first used in the early 1970s in Indiana, New York, and Alabama for new bridge construction as well as rehabilitation. According to Martin and Osborn (1983), those early Indiana bridges used tongue-and-groove joints to connect the precast panels (see Figure 2.1). Water sealant was placed in the joint to prevent water leakage as well as reduce stress concentrations. Cracking, spalling, and leakage were observed at the joints. These joints did not perform satisfactorily due to their high sensitivity to forming tolerances.

![Figure 2.1: Male-to-female connection](image)

Significant advances have been made in full-depth precast concrete panels since the mid-1970s to the beginning of 1980s (Issa et al., 1995). Various design and
construction details of joints were used in different state DOTs, including female-to-
female (see Figure 2.2-2.4), male-to-female (see Figure 2.1), and flat connections (see
Figure 2.5). There are mainly three types of female-to-female connections: connection
with snugged bottom ends (see Figure 2.2), connection with a backer rod at the
bottom throat (see Figure 2.3), and connection with full-depth grouted shear key
(see Figure 2.4). The male-to-female connection refers to the tongue-and-groove joint
as seen in Figure 2.1, which can be dry or epoxied with or without post-tensioning.
As mentioned above, this type of joint has strict requirement for form tolerance and
therefore is usually match-cast. The flat connection is usually used together with
post-tensioning.

As far as female-to-female connections are concerned, the Virginia DOT used
transverse bulb joints with snugged bottom ends filled with non-shrink mortar for
Route 229 Bridge and Route 235 Bridge over Dougue Creek, Fairfax. Alaska DOT
used transverse bulb joint with backer bar at the bottom throat between adjacent
precast panels for Dalton Highway Bridges and Chulitna River Bridge. Similar config-
uration was also adopted for Bridge 03200 by Connecticut DOT, where the transverse
joint was filled with high-strength, non-shrink grout, and tightened by longitudinal post-tensioning. New York State DOT used longitudinal bulb joints with full-depth shear keys filled with non-shrink cement grout for Route 155 Bridge over Norman-skill State Highway 1928. This shear key configuration was also adopted by several other DOTs using different closure materials. Maine DOT used epoxy mortar as transverse joint material for Deer Isle-Sedgwick Bridge without prestressing to the slabs; and the Texas Department of Highways and Public Transportation used epoxy
mortar between precast panels for A.T. and S.F. Railway Overpass. In addition to female-to-female connections, the Illinois DOT used male-to-female transverse joints filled with epoxy adhesives in between the precast panels for Seneca Bridge; Maryland DOT used 1 $\frac{1}{4}$ in.-wide transverse flat joints filled with polymer concrete with longitudinal post-tensioning for Woodrow Wilson Memorial Bridge.

According to a follow-up investigation by Yousif (1995) of the joint performance between adjacent precast full-depth concrete panels, cracking and rusting were observed at the transverse bulb joint with snugged bottom ends for Route 235 Bridge rehabilitated in 1982 by the Virginia DOT as mentioned above. Leakage was also found in similar transverse joints for the William Preston Jr. Memorial Bridge in Maryland due to the closed ends of the joint. This was even in the presence of longitudinal post-tensioning. This is because the closed ends crack easily under tension. As such it is recommended to use at least a $\frac{1}{4}$ in. opening at the bottom of the joint to account for dimension irregularities or misalignment. Transverse bulb joints with a backer rod at the bottom throat, as adopted by the Alaska DOT for the Chulitna River Bridge, was found to have debonded at the bottom of the shear key because the backer rod allows rotation and, as believed by Yousif (1995), there was no longitudi-
nal post-tensioning. The similar joint adopted by the Connecticut DOT for Bridge 03200 reconstructed in 1989, however, performed satisfactorily, which attributed to post-tensioning with a minimum stress of 150 psi. Nevertheless, this backer rod was later strongly suggested to never be used for structural joints due to the tolerance issue that can lead to much weaker joints than are designed (Nottingham 1996). Leakage was also observed at the transverse joint for the Amsterdam Interchange Bridge set up from 1973 to 1974 by the New York State Thruway Authority, which utilized bulb joints with full-depth shear keys filled with a low modulus epoxy mortar. This leakage was believed to be attributed to the lack of presence of any post-tensioning in the longitudinal direction. The match-cast transverse male-to-female joint glued with epoxy as adopted by the Illinois DOT for the Seneca Bridge reconstructed in 1986 also showed signs of leakage at the approach spans. It was concluded that joint shape, joint material, post-tensioning, and construction procedures all play important roles in better joint performance. A recommended shear key shape is a female-to-female type with at least a \( \frac{1}{4} \) in. opening at the bottom to allow for any panel size irregularities. High strength polymer grout and post-tensioning were also suggested by Yousif (1995). Also when evaluating joint performance, the traffic volume should be considered. A joint that performs well under low traffic volume does not guarantee the feasibility of the joint under high traffic volumes.

The importance of joint configuration and joint material was also stressed by Nottingham (1996) when talking about the application of precast concrete deck panels in Alaskan bridges. As pointed out by the author, the typical joint used in Alaska was not a good choice because the bottom throat can be either tight or loose, which can lead to improper joint fit and incomplete grouting, ending up with a much weaker joint than designed (see Figure 2.6). Form packing rods should never be used in structural grouted joints. A bulb joint with a full-depth grouted shear key was recommended
because it has sufficient size to account for panel dimension irregularities and accelerates construction speed (see Figure 2.7). As far as jointing material is concerned, the following properties were desired: low shrinkage, impermeable, high bond, high early strength with user friendly characteristics and low temperature curing ability. Based on field observance, the magnesium ammonium phosphate ($MgNH_4PO_4$) grout often extended with pea gravel was found to give satisfactory joint performance if properly used (Nottingham, 1996). When using this type of material, joint interfaces must be sandblasted and pressure washed prior to grouting to remove laitance, and most importantly, carbonated concrete substrate. It is recommended to use phenolphthalein to verify the absence of carbonation. The influence of substrate carbonation on the bond was explained by Gulyas et al. (1995): ‘The chemical component of set45 is a magnesium ammonium phosphate, which at the early phase of hydration, has a low PH. When meeting with the carbonated substrate ($CaCO_3$), this acid phase reacts with $CaCO_3$ and produces a noticeable fizzing together with a lot of bubbles, therefore decreasing bond strength greatly.’

Nottingham’s perspectives concerning what makes a good joint between precast concrete panels were confirmed by Gulyas (1996), who studied and compared the bond performances of a bulb joint with full-depth shear key filled with non-shrink grout/set45/set45 (hot weather) (Gulyas et al., 1995) through both component and composite tests. The composite tests were designed to test the bonding properties of the joint material, including composite direct tensile and shear bond testing. In the direct tensile test and vertical shear test, non-shrink grout gave bond failure, while set45 gave either substrate failure or a mixture of substrate and grout failure, indicating a much stronger bond. The better bond performance of set45 was considered to be related to its low shrinkage and the proper preparation of the joint interfaces. However, the bonding property of set45 is very sensitive to carbonation.
and moisture. It was concluded that when evaluating joint materials, a composite test is more practical than a component test as expressed by Gulyas (1996): ‘The composite test evaluates the interdependence of the grout to the precast unit and the preparation technique as well as the effects of drying shrinkage.’ These two test methods—composite and component—combined with proper interface preparation, can provide sufficient proof of whether a material is adequate or not as joint material. For material component test with respect to durability, Gulyas (1996) suggests the
use of sulfate durability, freeze-thaw durability, and chloride ingress tests considering the exposure environment of concrete.

Joint materials for the same joint shape were further explored by Issa et al. (2003) using both component and composite tests. Four materials were studied including set45 for normal temperatures, set45 for hot weather, set grout, and polymer concrete. The component test included chloride penetration and shrinkage tests, and the composite test included a direct tensile test, a vertical shear test, and a 4-point flexural test. Before grouting, joint interfaces were sandblasted until coarse aggregates were slightly exposed, followed by air-pressure cleaning, and then high-pressure washing. The polymer concrete had the best performance in the component test and composite test relative to capacity and failure mode. In all the cases, failure happened in the substrate. In addition, the polymer concrete needs no curing and has high early strength. The disadvantage is that polymer concrete is very expensive and the mixing procedure is very complicated. In comparison, the bond performance and strength of set45 were negatively affected by either moisture or carbonation in the
substrate. This again confirms the importance of the interface preparation were set to be used.

Zhu and Ma (2010) developed a set of selection criteria for closure pour materials for ABC, including the overnight-cure materials and 7-day-cure materials. Examined materials included a cement-based material, magnesium-phosphate based materials, high-performance concrete (HPC) mixtures, Emaco T430 mix with latex, etc. Compared with the previous criteria concerning bonding properties proposed by Russell and Ozyildirim (2006) (see Table 2.1) and Tepke and Tikalsky (2007) (see Table 2.2), the criteria proposed by Zhu and Ma (2010) (see Table 2.3) was developed considering the different curing schemes required for ABC. Moreover, some of the test methods used in the previous criteria were also changed relative to shrinkage and chloride penetration. In addition to compressive strength (ASTM C39 modified), shrinkage (AASHTO PP 34-99 modified), chloride penetration (ASTM C1543 modified), and freeze-and-thaw durability (ASTM C666 Procedure A modified), a new criterion is bond strength (ASTM C882 modified).
### Table 2.1: Selective performance characteristic grades by Russell and Ozyildirim (2006)

<table>
<thead>
<tr>
<th>Performance characteristic</th>
<th>Test method</th>
<th>Grade 1</th>
<th>Grade 2</th>
<th>Grade 3</th>
</tr>
</thead>
<tbody>
<tr>
<td>Compressive strength (CS) (MPa)</td>
<td>AASHTO T22 ASTM C39</td>
<td>55 ≤ CS &lt; 69</td>
<td>69 ≤ CS &lt; 97</td>
<td>97 ≤ CS</td>
</tr>
<tr>
<td>Shrinkage (S) (µϵ)</td>
<td>AASHTO T160 ASTM C157</td>
<td>600 ≤ S &lt; 800</td>
<td>400 ≤ S &lt; 600</td>
<td>S &lt; 400</td>
</tr>
<tr>
<td>Chloride penetration (ChP) (C)</td>
<td>AASHTO T277 ASTM C1202</td>
<td>1,500 ≤ ChP &lt; 2,500</td>
<td>500 ≤ ChP &lt; 1,500</td>
<td>ChP ≤ 500</td>
</tr>
<tr>
<td>Freezing-and-thawing durability (F/T) (relative dynamic modulus of elasticity after 300 cycles)</td>
<td>Procedure A</td>
<td>70% ≤ F/T &lt; 80%</td>
<td>80% ≤ F/T &lt; 90%</td>
<td>90% ≤ F/T</td>
</tr>
</tbody>
</table>

*a* All tests to be performed on concrete samples moist or submersion cured for 56 days until otherwise specified.

*b* The 56-day strength is recommended.

*c* Shrinkage measurements are to start 28 days after moist curing and be taken for a drying period of 180 days.

### Table 2.2: Selective performance characteristic grades by Tepke and Tikalsky (2007)

<table>
<thead>
<tr>
<th>Performance characteristic</th>
<th>Test method</th>
<th>Grade 1</th>
<th>Grade 2</th>
<th>Grade 3</th>
</tr>
</thead>
<tbody>
<tr>
<td>Compressive strength (CS) (MPa)</td>
<td>AASHTO T22</td>
<td>24 ≤ CS ≤ 55 at 28 days</td>
<td>55 ≤ CS at 28 days</td>
<td>24 ≤ CS at early ages after 30 days moist curing and 7 days air drying</td>
</tr>
<tr>
<td>Shrinkage (S) (µϵ)</td>
<td>ASTM C157</td>
<td>S ≤ 600 at 56 days</td>
<td>S ≤ 400 at 56 days</td>
<td>S ≤ 200 at 56 days</td>
</tr>
<tr>
<td>Chloride penetration (ChP) (C)</td>
<td>AASHTO T277</td>
<td>ChP ≤ 4,000 at 56 days</td>
<td>ChP ≤ 1,500 at 56 days</td>
<td>ChP ≤ 800 at 56 days</td>
</tr>
<tr>
<td>Freezing-and-thawing durability (F/T) (relative modulus after 300 cycles)</td>
<td>AASHTO T161 Procedure A</td>
<td>60% ≤ F/T</td>
<td>80% ≤ F/T</td>
<td>90% ≤ F/T</td>
</tr>
</tbody>
</table>
### Table 2.3: Proposed performance criteria of CP materials by Zhu (2010)

<table>
<thead>
<tr>
<th>Performance characteristic</th>
<th>Test method</th>
<th>Performance criteria</th>
</tr>
</thead>
<tbody>
<tr>
<td>Compressive strength (CS) (MPa)</td>
<td>ASTM C39 modified</td>
<td>41.4 ≤ CS At 8 h (overnight cure) At 7 days (7-day cure)</td>
</tr>
<tr>
<td>Shrinkage&lt;sup&gt;a&lt;/sup&gt; (S) (crack age, days)</td>
<td>AASHTO PP 34 – 99 modified</td>
<td>20 &lt; S</td>
</tr>
<tr>
<td>Bond strength (BS) (MPa)</td>
<td>ASTM C882 modified</td>
<td>2.5 &lt; BS</td>
</tr>
<tr>
<td>Chloride penetration&lt;sup&gt;b&lt;/sup&gt; (ChP) (depth for percent chloride of 0.2% by mass of cement after 90-day ponding, mm)</td>
<td>ASTM C1543 modified</td>
<td>ChP &lt; 38</td>
</tr>
<tr>
<td>Freezing-and-thawing durability&lt;sup&gt;c&lt;/sup&gt; (F/T) (relative modulus after 300 cycles)</td>
<td>ASTM C666 Procedure A modified</td>
<td>Grade 1 Grade 2 Grade 3</td>
</tr>
<tr>
<td></td>
<td></td>
<td>70% ≤ F/T 80% ≤ F/T 90% ≤ F/T</td>
</tr>
</tbody>
</table>

<sup>a</sup> No S criterion need be specified if the CP material is not exposed to moisture, chloride salts, or soluble sulfate environments.

<sup>b</sup> No ChP criterion need be specified if the CP material is not exposed to chloride salts or soluble sulfate environments.

<sup>c</sup> Grades are defined as follows: (1) F/T grade should not be specified if the concrete is not exposed to freezing-and-thawing environments; (2) when the concrete is exposed to freezing and thawing but without exposing to deicing salts, F/T Grade 1 should be specified; and (3) when the concrete is exposed to freezing-and-thawing and deicing salts, F/T Grade 2 be specified for members not saturated during freezing and F/T Grade 3 for members saturated during freezing.
The above studies mainly focused on unreinforced female-to-female joints. These joints are generally designed for shear and tension. Due to the lack of reinforcement, they are usually weak in tension resulting from shrinkage, temperature change, and bending. Therefore these joints are usually used together with weld connectors. In such cases, the weld connectors act to keep the joints tight under tension, and the unreinforced portion takes the large portion of shear. Other reinforced joints also utilize post-tensioning or well-distributed reinforcement.

Porter et al. (2010) performed experimental studies on five types of full-scale female-to-female transverse connection details including two conventional details used by the Utah DOT and three modified details. One conventional detail used a combination of weld stud connections at a fixed spacing and grouted shear keys (see Figure 2.8a) and the other utilized straight post-tensioning located at the middle depth of the panel (see Figure 2.9a). Based on the two conventional connections, three modified connections were developed. In the modified welded connection, the two studs were replaced with reinforcing bars which had a longer anchor length (see Figure 2.8b). In the modified post-tensioned connection, long straight rods were replaced with curved bolt connections across one joint (see Figures 2.9b-2.9c). By doing so it is possible to replace a single panel independent of the rest of the whole bridge deck while providing adequate serviceability performance. In the two variations, one had a small curvature, and the other had a larger curvature. Non-shrink grout was used for all the joints and pockets. The joint was subjected to pure bending under the four-point flexural test. Specimens were loaded monotonically until failure. Results show that the traditional post-tensioned joint is good at crack control in terms of cracking load, followed by the weld rebar connection, and then the long curved bolt connection. As far as the ultimate capacity is concerned, the long curved bolt connection was the strongest. The weld rebar connection gave a higher capacity than
that of the weld stud connection due to the fact that the anchor length of the weld stud connection was not sufficient enough to yield the rebar. The welded connections do not have adequate negative moment capacity and are not suggested to be used in such occasions.

The use of well-distributed reinforcement within the joint will be discussed in the next section. It is also applied to the full-depth precast concrete panels. In fact many of the joints talked above, though developed for full-depth precast concrete panels, are also applied to other types of superstructures.

2.2.2 Joints connecting stemmed beams

According to Martin and Osborn (1983), grout keys for longitudinal joints are usually the best choice for this type of superstructure. Other joints such as tongue-in-groove joints often proved impractical.

Standards of joint details for such kinds of precast beams, as pointed out by Martin (1983), generally ‘appear to have been set on the basis of subjective evaluations and modified when performance was unsatisfactory.’ Various joint materials have been tried including sand-cement grouts, non-shrink grout, and epoxy grout. The sand-cement grouts usually gave low strength and high shrinkage due to the high water-to-cement ratios needed for good workability. The non-shrink grout used expansive gradients so that the grout expands during initial hardening and curing by either generating gas, oxidation of metals, formation of gypsum, etc. This expansion offset the subsequent shrinkage. The longitudinal joints were tightened transversely using various transverse ties including non-prestressed tie rods and bolts, lateral post-tensioning, weld plate connectors, and epoxy shear keys. These transverse ties are only required to provide enough horizontal restraint to keep the grout keys closed.
Figure 2.8: Welded connection details (Porter et al., 2010)

under the action of transverse bending moments, shrinkage, and temperature change, etc. Analysis shows that the non-prestressed tie rods and bolts are not sufficient to
keep the joint tight. Epoxy shear keys can give satisfactory joint performance only when the shear key interfaces are properly sandblasted. The weld plate connectors spaced usually from 4 to 6 ft along the longitudinal direction were widely used. This type of joint was reported to generally perform well. The main problem with this type of joint, according to Martin and Osborn (1983) and Stanton and Mattock (1986), is the grout key, which has more of an issue with poor operation rather than with concept (Stanton and Mattock, 1986). A typical longitudinal joint using alternating
weld plate connectors and grout joints as reported by Stanton and Mattock (1986) is shown in Figure 2.10. Transverse post-tensioning is also an effective way to keep the joint tight.

![Grouted shear key](image1)

(a) Grouted shear key

![Welded connectors—vertical view](image2)

(b) Welded connectors—vertical view

![Welded connectors—top view](image3)

(c) Welded connectors—top view

Figure 2.10: Typical flange connection detail reported by Stanton and Mattock (1986)

In Florida, transverse post-tensioning was applied to the longitudinal joints of double-tee beam bridges with spans up to 80 ft. The prestressing level to achieve a monolithic behavior in these bridges was explored (Shahawy, 1990). It was believed that under the effect of post-tensioning, it is not necessary to use the complicated keyed or match-cast joints using expensive epoxy adhesives. In the test, longitudinal V-joints were used and filled with non-shrink grout (see Figure 2.11) and transverse post-tensioning was applied close to the mid-depth of the flange. It was found that transverse post-tensioning with at least 150-psi effective prestress can lead to mono-
lithic behavior under a punching shear test. Fatigue characteristics of longitudinal V-joints under transverse prestress were also examined by applying a constant load range which was equivalent to an AASHTO design truck with 30 percent impact. Under an effective transverse prestress level of 150 psi, the 1:3.5 scaled model with two continuous spans underwent 2 million cycles and showed stabilized crack propagation characterized by crack width. Static load tests on real double-tee bridges built in 1987 were performed by Shahawy and Issa (1992) to examine the behavior of this type of joint under the effect of transverse post-tensioning. The effective prestress was more than 200 psi. Under both shear and moment loading, this transverse prestress level was found to be sufficient in achieving monolithic behavior and the punching shear behavior resembled that of CIP concrete slab on multibeams.

![V-joints used for double-tee beams in Florida](image)

**Figure 2.11:** V-joints used for double-tee beams in Florida

The weld plate connections, as well as prestressing strands, although not verified, may be negatively influenced by deicing salts under high traffic volume because these types of joints have limited moment-transfer capacity and are weak in crack control (Martin and Osborn, 1983). In comparison, well distributed reinforcement is better at crack control than widely distributed weld plate connectors, and could
provide more moment capacity. In this perspective, straight reinforcement may be the first that comes to mind. However, straight reinforcement requires a much wider joint due to the development length requirement. It is important to keep the joint as narrow as possible to accelerate bridge construction at the lowest cost. As such, other joint rebar configurations have been explored.

Li et al. (2009a) examined two joint rebar configurations for the 6-in. deep integral deck of a bulb tee beam superstructure. One configuration used a single layer staggered headed rebar (#5) placed in the middle depth of the deck (see Figure 2.12) and the other used welded wire reinforcement (WWR)(#5). In the headed rebar configuration, the headed bar has a 2 in-diameter and 0.5 in-thick circular welded head. In the middle of the lap length, a #5 bar with a 1.375 in-diameter circular head at each end was placed longitudinally both above and below the transverse headed bar. For this rebar configuration, two main factors were studied: rebar spacing from 4 in. to 6 in. and lap length from 2.5 in. to 6 in.. For the WWR configuration, only the spacing was studied, varying from 4 in. to 6 in. In order to focus on only the influence of rebar configuration, the joint was excluded and the specimen was cast monolithically. The slab specimen was simply supported and tested under four-point bending with the joint zone subjected to pure moment, under patch loading, until failure. The specimen was monitored for flexural capacity, ductility in the form of curvature and deflection, and failure modes, etc. The headed rebar configuration with a 6 in. lap length is sufficient to fully develop the headed bar, giving full moment capacity and significant ductility. Compared with the conventional welded connection, this rebar configuration is more efficient at crack control, moment capacity, and simplifying construction.

The same reinforcement detail as mentioned above was tested with the presence of the joint by Li et al. (2009b). The joint had a diamond shape (see Figure
Figure 2.12: Headed reinforcement connection studied by Li et al. (2009a)

2.13) and was filled with set45 HW with 60% extension of round pea gravel, which was selected based on the study of Zhu and Ma (2010) and additional tests. The joint interfaces were sandblasted. Both four-point pure flexure and three-point flexure-shear tests were conducted for static and fatigue performance of the joint. The fatigue test underwent 2 million cycles at a frequency of 4 Hz, exerting both positive moment demand and negative moment demand determined based on a finite element study. The influence of the fatigue test on the capacity and ductility was documented. It was found that the fatigue test had no influence on specimen capacity, but reduced the ductility under pure bending by inhibiting the plastic hinge development.

Although more advantageous than the weld plate connection in crack control, the single layer headed bar detail mentioned above still is not quite effective at controlling cracks due to the mid-depth position of the headed bar. This position is restricted by deck depth, concrete cover, and head size. Based on the study of Li et al. (2009b), Ma et al. (2012) examined another two rebar configurations: two-layer staggered headed bar with smaller heads and staggered U-bar with tight bends (see Figure 2.14). For the headed rebar configuration, in order to fit within
Figure 2.13: Headed reinforcement connection tested by Li (2010b)
the narrow flange depth (6.25 in) while maintaining the minimum concrete cover, a smaller-diameter (1.5 in) circular head was used for the headed bar. Two lacer bars were put in the middle of the lap length above and below the two layers of staggered headed bars. For the U-bar configuration, due to the restraint of the deck depth and cover requirement, the inside diameter of the bend was three times the bar diameter for the #5 bar, which is only half of that required by ACI. To avoid the breaking of the bar during bending and the crushing of the concrete within the tight bend, deformed wire reinforcement (DWR) and stainless steel were used, which are all very ductile. Two #4 lacer bars were placed at the bearing faces of the U-bar. Specimens were cast monolithically without joints so as to focus only on the rebar configuration. The specimen was subjected to four-point patch loading with the joint zone under pure bending, and was monotonically loaded until failure. Experimental results showed that the DWR U-bar connection provided more capacity and tighter crack width at the service load level as compared with the headed bar configuration. It is believed that the #5 DWR U-bar configuration with 4.5 in. spacing and 6 in. overlap length is a viable choice (Ma et al., 2012).

This same rebar configuration for the longitudinal joint was later examined in the presence of the diamond joint (see Figure 2.15) under static flexure-shear, static flexure, fatigue flexure-shear, and fatigue flexure tests (French et al., 2011). One over-night cure material set45 HW and one 7-day cure material HPC Mix1 based on the study of Zhu and Ma (2010) were selected to fill the joint. Before grouting, joint interfaces were sandblasted for better bonding performance. Joint performances were evaluated at both service load and strength load levels. The fatigue test underwent 2 million cycles at a frequency of 4 Hz, exerting both positive moment and negative moment demands determined based on a finite element study. The fatigue load was found to have little influence on joint behaviors at the service load level in terms
of the average curvature, deflection at the middle span, rebar strain, and relative displacement of interfaces. At the strength load level, the fatigue load was observed to reduce the loading capacity under flexure test for the overnight-cure material. The U-bar detail was confirmed to be a viable longitudinal connection between decked-bulb-Ts and full-depth precast deck panels.

Graybeal (2010a) explored the use of ultra-high performance concrete (UHPC) for both longitudinal and transverse diamond joints. Two joint reinforcement details were examined for the longitudinal joint including staggered headed mild-steel reinforcement (see Figure 2.16) and staggered straight mild-steel reinforcement (see Figure 2.17). The specimens were subjected to both static and fatigue loads, which covered 9 to 12 million cycles. Generally, the bond showed extremely promising performance under both static and fatigue loads. For the longitudinal joint, large flexural stresses did not necessarily result in interface crack. Monolithic behavior was
Figure 2.15: Specimen tested by French et al. (2011)
achieved. In addition, no rebar debonding was found for any joint. Even under a severe stress range, the straight mild-steel reinforcing bars failed due to metal fatigue. This means that UHPC allows the use of shorter anchorage of rebar, which leads to narrow joints. It was also pointed out that the good performance of the bond does not guarantee a leak-free precast panel.

2.2.3 Joints connecting box beams

For box beams, the early shear key is generally of a bulb shape located at the upper portion of the beam, filled with non-shrink grout, and tightened by transverse ties. Cracking and leaking with such shear keys are widely reported. Studies have been conducted on using high performance joint materials, changing the location of the shear key, increasing the depth of shear keys, widening shear keys, and increasing the post-tensioning force.

Gulyas et al. (1995) compared the performance of non-shrink grout and magnesium ammonium phosphate mortars as shear key materials under both component test and composite test. The magnesium ammonium phosphate mortar gave much better bond performances (higher capacity and usually substrate failure) in the composite tests than the non-shrink grout. This is believed to be related to the low shrinkage of the magnesium ammonium phosphate mortar.

Huckelbridge Jr. et al. (1995) reported that fracture of an upper-depth shear key, as shown in Figure 2.18, is common for box beam bridges. During an experimental study of a bridge before and after shear key/deck membrane replacement, even the new joints showed indications of joint fracture. If the fracture length takes a significant fraction of the girder span, the load transfer capacity could be substantially
impaired. It was also reported that tie bars seemed to have little or no impact on shear key performance.

Lall et al. (1998) compared the shear key performances of the partial shear-keys built before 1992 (see Figure 2.19a) and full-depth shear keys with more post-
tensioning after 1992 (see Figure 2.19b) based on a nationwide investigation. It was found that a full-depth shear key with more transverse post-tensioning can effectively control the frequency of occurrence of longitudinal reflective cracking in shear keys.
Miller et al. (1999) investigated the cracking behavior and load transfer ability of both upper-depth and mid-depth shear keys filled with either non-shrink grout or epoxy. The examined shear key detail, regardless of the location, is as that studied by Huckelbridge (1995) (see Figure 2.18). It was noticed that the upper-depth shear key with non-shrink grout cracked before loading due to temperature change. The mid-depth shear key filled with non-shrink grout was less sensitive to temperature change. Therefore it is suggested to use mid-depth shear key with the throat filled with water sealant to reduce cracking. The upper-depth shear key filled with epoxy did not crack. However, considering the large variation of thermal expansion coefficient of epoxy from season to season, cracks could be formed in the substrate due to this coefficient incompatibility with concrete. Other options to reduce shear key cracking include using full-depth shear key, post-tensioning, and materials with high bond strength such as magnesium ammonium phosphate mortars. As far as the impact of shear key condition on load transfer ability is concerned, it was found that even cracked a shear key is still effective in transferring vertical loads, which leads to the conclusion that it is leakage rather than load transfer that is the main concern caused by shear key cracking.
(a) Partial-depth shear key system used before 1992 (cited by Lall, 1998)

(b) Full-depth shear key system used after 1992 (Lall, 1998)

Figure 2.19: Shear key system studied by Lall (1998)
2.3 Double-Tee and NEXT-D

According to Stanton and Mattock (1986), double-tees are widely used on county, municipal or private roads; it is also used for highway bridges. According to an investigation of national experience with double-tee beams (Hag-Elsafi, 1998): double-tee beams have been used on short-to-medium-span bridges with low traffic volumes and less than 30 degree skew angles; the top flanges can be connected by longitudinal grouted keyways and weld connectors at fixed intervals, or by transverse post-tensioning; diaphragms are sometimes applied and post-tensioned for lateral stability. Impermeable water-proof membranes can be applied, covered by asphalt wearing surface for leakage protection.

In Florida, a double-tee structural system has been developed for interstate class highway applications (Csagoly and NICHAS, 1987). The precast double-tee units are joined together by a simple V-joint and transverse post-tensioning, without any mechanical shear connectors or overlay (see Figure 2.11). A minimum of 150-psi effective transverse prestress was applied for better crack control under punching shear. The fatigue behavior of the longitudinal joint was confirmed experimentally. Longitudinally, draped strand profiles were applied to the beam to increase its structural effectiveness. Debonding of strands was eliminated due to potential premature shear failure. Top strands (straight) close to the neutral axis were suggested to be used for all double-tees for highway bridges for better crack and camber control. A typical double-tee cross section used in Florida is shown in Figure 2.20.

The most significant features of double-tee beams are the simplicity in fabrication and the resulting low cost. As such, double-tees have been widely used for the replacement of old bridges. In New York State, double-tee beams with either cast-in-place composite deck or non-composite bituminous wearing surface were de-
signed to replace the currently used voided slab bridges for short span bridges (span length varies from 25 to 60 ft) (Hag-Elsafi, 1998). The proposed cross section and joint configuration for the non-composite beam are shown in Figure 2.21.

In the northeastern United States, a beam type that closely resembles a double-tee beam— northeast extreme tee (NEXT) beam— has been developed by PCINE as a substitute for box beams for medium span bridges (spans varies from 45 ft to 90 ft)(Culmo and Seraderian, 2010). Compared with the traditional double-tee beam, the NEXT beam has wider stems to handle the moment and shear demands. Compared with the box beam, the NEXT beam has the following advantages: 1. It is much simpler to be fabricated; 2. Under-bridge utilities can be easily supported between the stems, which eliminates the need for parapet attachment if box beam is to be used; 3. Specified bridge width and deck profile can be obtained due to the flexibility in adjusting the flange width. As far as the beam design is concerned, a fixed stem spacing and size are desired so that a single form can be used. Variation in the beam is achieved by adjusting the depth with fillers and the width with adjustable side forms. Since PCINE wants the beam to be as narrow as 8 ft, the stem spacing
was set to a maximum of 5 ft. According to the targeted beam spans, the maximum beam depth was set to 36 in. The stem width was determined based on the amount of strand, size of shear reinforcement, and desired concrete cover. The bottom of the stem width was finalized at 13 in. to accommodate 5 columns of strands and a no. 4 shear stirrup. The slope of the stem was 0.375 in./ft to provide enough draft for removing the beams from the forms. If vertical forms are used, a forming system comprising of complicated collapsible formwork is required, which is more
complicated and expensive. Considering shipping and handling, a partial-depth flange was determined to reduce the beam weight. However, a full-depth top flange is desired, which can be as thick as 8 in. to accommodate truck loads as well as concrete cover. In such cases, the beam is suitable for short-span secondary roads with low truck volumes. If longer spans are desired, light-weight concrete is an option. In addition, the longitudinal connection between the precast units needs to be carefully designed. Straight strands were proposed instead of draped strands to improve construction efficiency and save construction costs. A typical non-composite beam cross section proposed by PCI Northeast is shown in Figure 2.22.

![Figure 2.22: NEXT D Beam by PCI Northeast](image)

At the time when the NEXT beam was proposed, the SCDOT was searching for an alternative to the hollow core beam bridge for the replacement of CIP flat slab bridges, which constitute a major portion of short-span bridges in South Carolina. Currently, the precast hollow core bridge is used only on low traffic volume roads and has serious durability issues with both transverse and longitudinal joints. In a study by Deery (2010), the NEXT beam was identified and deemed as a promising alternative to the hollow core beam. The NEXT beam is deemed to be adaptable to
22 ft and 40 ft bridge spans with high traffic volume, and may be cost competitive with the hollow core beam bridge once formwork is available. In order to be efficient for short span lengths and width control, the beam width range was narrowed to 6 ft (NEXT-6) to 8 ft (NEXT-8) (see Figure 2.23a). Correspondingly the minimum beam depth was reduced to 20 in. The single layer of #5 headed rebar connection proposed by Li et al. (2009a) and the wide bulb joint shape were chosen for the longitudinal connection (see Figure 2.23b). An initial evaluation of the deck design forces revealed that the average ratio of positive moment and negative moment generated in the shear key was approximately 2:1 for the NEXT 6 and 6:1 for the NEXT 8. As such, the headed bar was suggested to be located one inch below the mid-depth of the shear key to accommodate the high positive moment demand. This rebar configuration later was found to be insufficient for crack control and was replaced by a #4 U-bar configuration (Nielson et al., 2013).

Before the NEXT beam can be put into service, the performance of the joint in the current configuration needs to be evaluated under both high moment and high shear demands. The fatigue behavior of the joint needs to be explored before this bridge can be confidently put under high traffic volume. Moreover, the live load distribution factors and transverse demand distribution of bridges constructed with NEXT-D beams need to be investigated to facilitate design.
Figure 2.23: Modified NEXT-D studied by Deery (2010)
Chapter 3

Performance Assessment of NEXT-D Shear Key

3.1 The Big Picture

Chapters 3 to 6 (including experimental, demand evaluation, and bridge design) are interdependent because of the shear key associated with the NEXT-D bridge. Indeed, the shear key influences, not only the behavior and performance of the bridge, but also the load demand placed on the bridge components and their subsequent designs. Thus it is difficult to discuss the performance assessment of the shear key without knowing the implications relative to analytical modeling. The specific link between the chapters is the stiffness matrix associated with the shear key. Information related to this matrix and the resulting performance prevents a linear presentation of the research which was conducted. To make the flow of this interaction more clear for the reader, an overview of the big picture is provided here.

The experiments in this chapter aim at fully understanding the performance of the shear key with respect to stiffness and strength in the static sense and durability
under fatigue. One of the unknowns is the magnitude of the fatigue load, which in the case studied is determined using a 3D finite element model of the NEXT-D bridge. However, this model required knowledge of the shear key stiffness which was provided by the static test results. To evaluate whether the fatigue load exerted is conservative or not, the stiffness obtained from the fatigue test will be compared with the stiffness from the static test. Refer to Figure 3.1 for a visual illustration of the performance assessment procedure. As far as the final design is concerned, the 3D bridge finite element model is used to determine both the transverse (deck) and longitudinal (beam) demands, which will provide a reference for further simplification of the beam design and deck design. In order to get a reasonable demand distribution, the shear key stiffness again is the key. This stiffness matrix will be determined based on the experimentally calibrated shear key finite element model. Therefore, this chapter and Chapter 4 will occasionally refer to the work presented in Chapters 5 and 6 while those chapters will refer to the work presented in this chapter and Chapter 4.

3.2 Introduction

The durability of transverse joints associated with adjacent precast concrete beams/slabs is a serious concern for state highway departments. The movement towards accelerated bridge construction (ABC) will only increase the number of precast concrete bridges built as existing bridges are replaced and new bridges are needed as the transportation routes are expanded. In South Carolina (SC), as well as most other states, the durability of longitudinal joints between adjacent prestressed hollow core concrete slabs compelled the South Carolina Department of Transportation (SC-DOT) to look for alternative designs for short-span bridges on low volume roads and
Figure 3.1: Performance assessment procedure
to see whether this bridge design is also suitable for use on high ADT roads in-place of cast-in-place concrete bridges or precast concrete bridges with a structural topping. According to an earlier investigation (Deery, 2010), the NEXT-D system developed by the Northeast Region of the Prestressed Concrete Institute (PCINE) was identified to be a promising alternative. This NEXT-D beam system is essentially a precast concrete double-tee beam (see Figure 3.2) with U-bars extending from the edges of the flanges into field cast shear keys. The original configuration used headed bars (see Figure 3.3) but was changed so that the shear key reinforcement could better control cracking.

![Figure 3.2: Modified NEXT-D examined in this study](image1)

![Figure 3.3: Original NEXT-D configuration with headed reinforcement](image2)

The detail of the shear key provides the required continuity between units and eliminates the need for additional transverse post-tensioning or structural topping over the precast concrete units. The NEXT-D beam system can be used to facilitate
accelerated bridge construction and is believed to be suitable for high ADT roads (Baur et al., 2010). However, before the NEXT-D beam system can be constructed, especially on the high ADT routes, several key issues still need to be resolved.

Additional testing to investigate the strength and stiffness of the shear key in the current configuration (i.e. shape and reinforcing details) needs to be conducted considering a number of different load demand scenarios. Previous tests on similar connections have focused on the behavior of the shear key under high moments. For the NEXT-D beam studied, one must recognize that large shear forces and relatively small bending moments may exist and this aspect has not been previously investigated. Graybeal (2010b) performed some studies on diamond-shaped shear keys using UHPC as the shear key grout material to investigate long-term fatigue damage. However, the shear key shape and reinforcing details in those studies do not directly reflect those under current consideration (see Figure 3.2).

While the literature suggests that the NEXT-D beam system appears ready for implementation, there are a few issues, such as fatigue issues, that need to be addressed before bridge engineers can have confidence in specifying this bridge system on high volume roads. An experimental research program was undertaken to study the structural behavior of the longitudinal shear key and its influence on the design of the bridge system. Experiments focus on both the capacity and load-deformation behavior of the shear key under static loading and also the reduction in strength and stiffness after an accumulation of many cycles of low level loading to address fatigue.

Given that the shear key is intended to create transverse continuity and that the shear key demand is a function of the stiffness of the shear key, a series of tests were proposed to look at different ratios of bending moment to shear force in the key. In order to understand the actual fatigue demand on the shear key, the shear key stiffness in the static test was considered in the demand determination for the
fatigue tests. Subsequently, the shear key stiffness in the fatigue tests can be used to validate the demand determination procedure. Two different commercially available cementitious premixes were used. One of the premixes was QUIKRETE Non-Shrink Precision Grout #1585 along with the addition of PVA fibers to control the formation of micro cracks. The other premix was Lafarge Ductal with the addition of either steel or PVA fibers and a high-range water reducer (super plasticizer). The results of the shear key testing along with additional analytical studies were used to determine an appropriate transverse distribution of applied loading and to validate the design of the shear key to meet the desired performance with respect to both strength and serviceability (see Figure 3.1).

This Chapter addresses the following aspects: experimental setup, shear key material selection, verification of shear key reinforcing details, and other test details.

3.3 Experimental Setup

The experimental setup includes the use of an existing steel reaction frame - modified for the testing of shear key specimens, static and fatigue load actuators, the control system, the design of a shear key specimen and data sensors deployed on each specimen and the data collection system.

3.3.1 Steel reaction frame

The steel reaction frame, shown in Figure 3.4, was originally built to test wall systems under both in-plane and out-of-plane loads. This frame was also designed to be self-reacting to eliminate any attachment to a foundation. Since the shear key specimens would be tested in a horizontal position, a new support system was designed and fabricated and attached to the four vertical columns of the reaction
frame. The upper beams were also modified to allow the large hydraulic actuators to hang from them. In Figure 3.4, the smaller 35-kip fatigue actuator is shown in proper position to apply a load to the shear key specimen. When a static test is conducted, both actuators are shifted to the left until the larger 160-kip actuator is in proper position to apply a load to a shear key specimen.

The actuators are connected to a feedback control system that allows for either a displacement- or load-controlled test to be conducted. The static tests were run using a displacement-controlled protocol, and the fatigue tests were run using a load-controlled protocol. However, the system was also setup to monitor abrupt changes in stiffness so that the onset of possible failure could be detected and shut the system down. While the investigators were interested in the ultimate capacity of the specimen, there was no benefit to destroying the specimen and risk damage to sensors, actuators, or reaction frame and more importantly the safety of the personnel conducting the tests.
3.3.2 Specimen dimensions and shear key configuration

The original shear key specimen was designed to be 92 in. in length, 48 in. in width and 8 in. in thickness with the shear key centered between two pieces of precast concrete. The shear key configuration, as proposed by PCINE, uses a single layer of headed bar extending from each precast piece into the shear key as illustrated in Figure 3.5. Since the #5 headed bars were on a staggered spacing, two #5 bars were tied to these headed bars to help facilitate tension load transfer from the headed bars in one piece to the headed bars in the other piece. In addition, five #4 bars were tied to the headed bars in each piece to serve as shrinkage and temperature steel and also hold the headed bars in proper position during casting of the specimens. The precast concrete had a 28-day design compressive strength of 6000 psi.

3.3.3 Sensor layout

Sensors were selected based upon the data that was needed to understand and quantify the behavior of the shear key specimen under either static or fatigue loading. The key information desired was the strain distribution through the thickness of the shear key, the deflection of the specimen and the relative rotation of the shear key with respect to the precast pieces and the possible opening of the interfaces between the shear key and precast concrete pieces. This data along with the applied loading of the specimen was collected at a regular interval during the testing of the specimen. Strain transducers where attached to the top and bottom surfaces of the specimen to measure bending strains, strain gages were adhered to selected bars extending into the shear keys (see Figure 3.6), LVDTs and string pots were used to measure vertical deflection of the specimen, relative rotations of the shear key and the opening of interfaces.
between each piece of precast and the shear key. The complete instrumentation diagram along with the chosen name for each sensor is shown in Figures 3.7 and 3.8.

3.3.4 Loading configuration

With the aim of exploring the shear key behavior under different moment to shear ratios, it was proposed that four ratios be tried including 43, 32, 22 and 12. The two extreme ratios would be tested first and the other two intermediate ratios would
only be tested if deemed necessary – they were not necessary. The different moment to shear ratios are realized by only changing the position of the right support, which is 43 in. away from the center of the shear key in the high moment test (HM) and 12 in. in the high shear test (HS) (see Figure 3.9). The applied loading was placed 8 in. off-center so that the shear key had a constant shear from face-to-face of the key. The bending span for the high moment configuration was set to 86 in. to ensure that there was not a bearing failure. The high shear configuration required the specimen to be cantilevered beyon

3.4 Primary Shear Key Material Selection

3.4.1 Selection criteria

When selecting shear key materials, four aspects are of primary concern.
Figure 3.7: Initial sensor layout (part 1)

Figure 3.8: Initial sensor layout (part 2)

Nomenclature:

1: Top BDIs (same for bottom BDIs):
- BDI-TB: BDI top back
- BDI-TC: BDI top center
- BDI-TF: BDI top front
- BDI-TL: BDI top left
- BDI-TR: BDI top right

2: LVDTs:
- LVDT-TF: LVDT top front
- LVDT-TB: LVDT top back
- LVDT-BF: LVDT bottom front
- LVDT-BB: LVDT bottom back
- LVDT-V: LVDT vertical
1. A shear key material with high early strength is desired to facilitate accelerated bridge construction.

2. The durability of the shear key material should be exceptional and have the ability to bond to the precast concrete and create a bond strength that exceeds the tensile capacity of the precast concrete.

3. The shear key material is expected to possess required workability so that long shear keys can be formed in a single pour and eliminate cold joints with the shear key material.

4. The cost of the shear key material, including placement costs, should be tolerable. The impact of a higher unit cost of high performing shear key material compared to typical concretes is minimized by the relatively small amount of shear key material needed to construct a bridge. In fact, the size of the shear key will likely get smaller as the strength parameters of the shear key material increases.

Nowadays, high early strength with good workability is no longer a problem by the use of water reducing admixtures in the shear key material mix design. The biggest concern is the durability of the shear key material during service, which com-
bined with a best possible shear key profile, plays a significant role in improving the durability of the shear key. This durability issue is initiated by cracking at the bond or within the key, which leads to rebar corrosion when exposed to water and deicing salts. This corrosion reduces the performance of the shear key over time. Hence, when choosing a shear key material, the critical issue is to control the crack propagation at the bond and in the shear key itself. To realize this, a material that possesses high bonding properties, toughness, and dimensional stability is desired. Considering the fact that all cementitious materials are brittle and prone to crack, it is necessary to add supplementary reinforcement like fibers in the shear key material to control the crack propagation within the shear key itself.

### 3.4.2 Primary material selection

Based on the previous successful examples of similar studies (Graybeal, 2010b), the original plan for the shear key material was to use Lafarge Ductal, an ultra-high performance concrete (UHPC) with micro fibers. Typically the specification calls for the use of steel fibers in structural applications and PVA fibers in architectural (non-structural) applications. The reason for this selection of Ductal is that it is well known that the use of standard non-shrink grouts has not traditionally performed well in the proposed shear key application. Based on work performed by FHWA researchers, this UHPC shows extreme promise for meeting both strength and serviceability requirements of the shear key. Furthermore, the original objective of this research was to develop a complete design of a bridge and neither the budget nor schedule allowed for much development of shear key materials. Instead, the researchers needed to pick from existing materials with the possibility of minimal modifications. Ductal was believed to have all of the desired attributes and would likely be an acceptable shear
key material and lower the risk of this research project not developing a plausible solution.

Although the South Carolina Department of Transportation (SCDOT) was concerned about the cost of Ductal, the bigger issues were the specification of a proprietary product for the shear key material and that the steel fibers supplied by Lafarge for the Ductal mix design were manufactured from steel wire drawn outside of the United States. The “Buy American” provision of the 1982 Surface transportation Assistance Act limits the amount of foreign produced steel that can be used in the construction of a bridge. The amount of steel fiber in the shear key material would typically exceed the Buy American limits and thus would require a waiver for each bridge constructed by the SCDOT. Since the SCDOT did not want to be in a position to continually request waivers, alternate mix designs were investigated — namely a typical non-shrink grout with PVA fibers added to the mix and a UHPC using PVA fibers instead of steel fibers.

A material study using an “off-the-shelf” non-shrink precision grout was considered for an alternative. Quikrete Non-Shrink Precision Grout with high strength and non-shrink properties was selected for this purpose. The Nycon-PVA-RFS400 micro fiber was selected for this investigation because it has dimensions (0.006 inches in diameter by 0.6875 inches in length), which are similar to the steel fibers dimensions (0.008 inches in diameter by 0.5 inches in length) used in Ductal. The Nycon-PVA-RFS400 micro fibers are touted to possess superior crack control properties and excellent tensile and molecular bond strength. The micro fibers are not intended to increase overall strength of the grout but rather control the micro cracking.

A control mix without fibers and three different fiber-to-grout ratios were explored using standard ASTM compression and split cylinder tests using 3 in. x 6 in. cylinders (ASTM, 2010, 2011). The four mixes are outlined in Table 3.1. The
Table 3.1: Grout and PVA fiber ratios considered

<table>
<thead>
<tr>
<th>ID</th>
<th>Description</th>
<th>Volume Percentage, %</th>
</tr>
</thead>
<tbody>
<tr>
<td>G</td>
<td>Grout without fibers</td>
<td>0.000</td>
</tr>
<tr>
<td>GF0.5</td>
<td>Grout with fibers (0.5 ounces / 50-lb bag)</td>
<td>0.085</td>
</tr>
<tr>
<td>GF2</td>
<td>Grout with fibers (2 ounces / 50-lb bag)</td>
<td>0.340</td>
</tr>
<tr>
<td>GF12</td>
<td>Grout with fibers (12 ounces / 50-lb bag)</td>
<td>2.000</td>
</tr>
</tbody>
</table>

GF0.5 and GF2 mix designs represent a typical range of fiber content as indicated by the fiber manufacturer (i.e. 1 to 8 lbs/yd$^3$ of mix). The fourth mix represents a volumetric ratio that is identical to the ratio used in the Ductal material (i.e. 243 pounds of steel fiber/yd$^3$ of mix and 45 pounds of PVA fiber/yd$^3$ of mix). The water volume recommended for a 50-lb bag of Quikrete Non-shrink Precision Grout ranges from 3 to 6 quarts depending on the flowability needed and recognizing that the addition of more water lowers the compressive strength of the mix. Based on some experimentation, five quarts of water per 50-bag was selected so as to achieve a workable mix while obtaining a desired compressive strength.

To ensure that fibers were distributed uniformly in the grout, for GF12 cylinders, a drill mounted paint/grout paddle mixer and a bucket were used instead of a drum mixer, which was used for the other three groups of cylinders. After an approximate five-minute mixing process, the mix was scooped into the molds, rodded and tapped according to ASTM C192 (ASTM, 2007a). They were then put into the curing room immediately. Considering the fast application requirement of the shear key, the cylinders were water cured in the curing room for either two or three days with only the first day in molds. After curing, the cylinders were stored at room temperature until tested.
Cylinder tests included a compression test, a splitting tension test, and a direct tension test. The first two tests (see Figure 3.10) were performed according to ASTM C39 and ASTM C496, respectively. Table 3.2 provides a summary of the average strength values obtained from each test where each test has a sample size of three to six specimens. A few basic trends can be identified from the results. For example, the curing duration did have a notable effect on the compressive strength of the grout. The compressive strength is also adversely affected by the addition of fiber on the order of 6 to 10 percent, while the splitting tensile strength is not affected greatly until large volume ratios of fibers were used. The two percent by volume (GF12) mix design demonstrated a 13.6 percent increase in splitting tensile strength over the unreinforced grout. The individual specimen values for the GF12 and concrete are given in Table 3.3 and Table 3.4.

![Figure 3.10: Cylinder tests](image)

For the direct tension test, steel fixtures were epoxied to both ends of a cylinder. After curing of the epoxy, the cylinder was tested using a universal testing
Table 3.2: Average compressive and tensile strengths of concrete and grout materials

<table>
<thead>
<tr>
<th>ID</th>
<th>Compressive Strength (psi)</th>
<th>Splitting Tensile Strength (psi)</th>
<th>Curing (days)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>ASTM C39</td>
<td>ASTM C496</td>
<td></td>
</tr>
<tr>
<td>Concrete-28 day</td>
<td>6930</td>
<td></td>
<td></td>
</tr>
<tr>
<td>G-14 day</td>
<td>10340</td>
<td>1210</td>
<td>2</td>
</tr>
<tr>
<td>GF0.5-14 day</td>
<td>9380</td>
<td>1120</td>
<td>2</td>
</tr>
<tr>
<td>GF2.0-14 day</td>
<td>9560</td>
<td>1280</td>
<td>2</td>
</tr>
<tr>
<td>G-14 day</td>
<td>10960</td>
<td>1360</td>
<td>3</td>
</tr>
<tr>
<td>GF12.0-14 day</td>
<td>10230</td>
<td>1545</td>
<td>3</td>
</tr>
</tbody>
</table>

Table 3.3: Test results of GF12 cylinders (3in × 6in)

<table>
<thead>
<tr>
<th>Cylinder</th>
<th>Compressive Strength (psi)</th>
<th>Splitting Tensile Strength (psi)</th>
<th>( f'_{cr} ) (14-day)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>3-day</td>
<td>7-day</td>
<td>14-day</td>
</tr>
<tr>
<td>1(^2)</td>
<td>7060</td>
<td>9410</td>
<td>10200</td>
</tr>
<tr>
<td>2</td>
<td>7480</td>
<td>9000</td>
<td>10430</td>
</tr>
<tr>
<td>3</td>
<td>7630</td>
<td>8570</td>
<td>9130</td>
</tr>
<tr>
<td>4</td>
<td>7250</td>
<td>8710</td>
<td>10370</td>
</tr>
<tr>
<td>5</td>
<td>6850</td>
<td>9680</td>
<td>10760</td>
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<tr>
<td>6</td>
<td>7530</td>
<td>9680</td>
<td>10480</td>
</tr>
<tr>
<td><strong>Mean</strong></td>
<td><strong>7300</strong></td>
<td><strong>9180</strong></td>
<td><strong>10230</strong></td>
</tr>
</tbody>
</table>

Note
1. The design compression strength \( f'_{cr} \) is calculated using \( f'_{cr} = 1.10f'_{c} + 700(\text{psi}) \) in accordance with ACI 318 – 11Table 5.3.2.2(ACI, 2011)
2. Cylinders 1-3 belong to a different batch from cylinders 4-6

Machine (UTM) by applying a tensile force to the cylinder through the steel end fixtures. A loading rate ranging between 0.020 in/min - 0.026 in/min was applied to keep the stress level application rate within acceptable limits. Although this direct tension test is theoretically applicable, it was difficult to align/level the end plates to produce a pure tensile force in the cylinder. These tests did provide an indication of the benefit of high dosage of fiber reinforcement. The ductility of each mix design
Table 3.4: Test results of concrete cylinders (4in × 8in)

<table>
<thead>
<tr>
<th>Cylinder</th>
<th>Compressive Strength (psi)</th>
<th>( f'_{c} ) (28 day)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>3 day</td>
<td>7 day</td>
</tr>
<tr>
<td>1</td>
<td>5360</td>
<td>5930</td>
</tr>
<tr>
<td>2</td>
<td>5460</td>
<td>6080</td>
</tr>
<tr>
<td>3</td>
<td>6090</td>
<td>6160</td>
</tr>
<tr>
<td>Mean ( f'_{cr} )</td>
<td>5640</td>
<td>6060</td>
</tr>
</tbody>
</table>

under direct tension was explored and is demonstrated in Figure 3.11. Since mitigating crack propagation under fatigue loadings is the purpose of the fibers, post-crack behavior (i.e. ductility) is a preliminary screening tool for the different mix designs. Ductility is taken as the ratio of the specimen extension divided by the specimen extension at ultimate load. As seen in Figure 3.11, the highly reinforced material (GF12) is the only material that exhibited the ability to sustain any load beyond the ultimate region. While fatigue testing is going to be required to identify if the crack control is met, these static results indicate that only one material mix should be investigated further.

In addition to the tests mentioned above, a bond strength test between concrete and GF0.5 was also performed using the same direct tension method with the purpose of getting a better understanding about the bond strength at the interface between the shear key and the slab. Bond test results suggest the bond may be more sensitive to tension than the concrete itself is.

Later a smaller diameter and shorter length PVA fiber Nycon-PVA-RECS15 (0.001496 in. in diameter x 0.375 in. in length) was selected for testing. This smaller fiber was selected to be able to get more fibers to cross any given crack for the same volumetric ratio of fiber. Like the Nycon-PVA-RFS400, the Nycon-PVA-RECS15
can also improve the ductility when the same volume percentage is used. Because the fiber is smaller, the mix would contain more fibers per unit volume and thus more uniform distribution in the new mix design, more water was added to account for the increasing surface area of the new fibers. Therefore the new mix design was 50 pounds of Quikrete Non-Shrink Precision Grout, 12 ounces of Nycon-PVA-RECS15 and 6 quarts of water.
3.5 Primary Reinforcing Detail Verification

After determining the materials to be used, the deck was then checked according to AASHTO LRFD Bridge Design Specifications for moment capacity ($\phi M_n$), interface shear transfer (shear friction), cracking control under service loads and minimum reinforcement requirements. This was verified with a single full-scale test of the system. In the original design detail proposed by PCINE, the position of the headed rebar (#5@6 in. o.c. in one layer) failed to satisfy the cracking requirement (AASHTO 5.7.3.4-1) under service loads. Since crack control is of great importance for this study, especially at the interface, a modification was proposed to use two layers of reinforcing steel instead of one. This change will allow the steel to be placed more closely to the free surfaces and thus become more effective in the control of crack width. Because of this, it was proposed that a switch from headed bars to a U-bar be considered. The U-bar allows for the centroid of the bar to be placed closer to the free-surfaces while maintaining the requisite cover as opposed to the headed bar.

Considering both the strength and serviceability requirements, a small range of reinforcement arrangements was examined. The use of #5 bars does not satisfy the minimum bend radius requirement, and therefore #4 bars were considered. It was decided to use a reinforcement schedule of #4 U-shaped bars at 8 in. o.c. (shown in Figure 3.12-3.15) for further testing under both static and fatigue loads.

3.6 Experiment Matrix Development

After the shear key material and reinforcing details were determined, the first group of specimen tests using the Quikrete Non-Shrink Precision Grout and PVA
Figure 3.12: Proposed detail for NEXT-D bridge joint - part 1

fiber mix as the shear key material was carried out. First, a static test was conducted to determine the stiffness of the specimen and the results were used to determine the
Figure 3.13: Proposed detail for NEXT-D bridge joint - part 2

magnitude of the fatigue load in a subsequent test. Due to the additional one quart of water added in the mix design for workability, the shear key material compressive strength was about 7500 psi during the day of static testing, which was quite similar to that of the concrete. The first crack in the static test happened at the interface when the bending moment across the width of the specimen reached 121 kip-in. The fatigue test showed cracks at the shear key interface after about 5000 cycles under a fatigue load level of 8.7 kips (equals 180 k-in of internal moment). A pond test conducted after 10 million fatigue load cycles showed an immediate seepage through the shear key interfaces. This fatigue load level used was later found to be higher than that required — the process is explained later in this chapter — and another fatigue test was later performed at a load of 5.3 kips (110 k-in of internal moment). As a consequence of the elevated fatigue load, the results of the fatigue test and pond

Note: Up to 2 inches of additional concrete may be added on top of the slab to accommodate any requisite grinding.

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test may not have direct application, but the early cracking at the interface in the static test did cause some concern in this preliminary test. The direct tensile bond test also showed bond failure. It was observed that the fiber did help control the crack propagation of the shear key, but as expected did not help improve the bond at the interface.

Due to the concerns of using Quikrete Non-Shrink Precision Grout, the next set of tests focused on using Lafarge’s Ductal with a steel fiber in the mix (JS1000) to see if there would be an improvement in the shear key performance. The test results using Ductal with steel fibers showed a significant improvement in bond cracking strength, which was 277 kip-in, 129 percent higher than that of the Quikrete mix.
hereafter just referred to as grout. After 10 million cycles of a fatigue load higher than that required, although there were slight cracks in one of the precast concrete pieces, there was no cracking along the interfaces. The subsequent 2 million cycles of loading during the ponding test showed no seepage through the interfaces. This performance clearly satisfied the design criteria.

Despite the superior performance of the UHPC with steel fibers compared to traditional non-shrink grout with PVA fibers, a mix design using UHPC with PVA fibers was tested to avoid the previously mentioned concern of using steel fibers in the mix. Since the UHPC mix with steel fibers exceeded the level of performance required, it was believed that a mix design using Ductal with PVA fibers (JS2000)
Table 3.5: Experiment matrix for static tests

<table>
<thead>
<tr>
<th>Shear Key Mixture</th>
<th>Moment to Shear Ratio</th>
<th>Specimen ID</th>
</tr>
</thead>
<tbody>
<tr>
<td>Quikrete Non-Shrink Precision Grout</td>
<td>43 (HM)</td>
<td>STA-01</td>
</tr>
<tr>
<td>+ Nycon-PVA-RECS15</td>
<td></td>
<td>MONO-01</td>
</tr>
<tr>
<td></td>
<td></td>
<td>MONO-01(redo)</td>
</tr>
<tr>
<td>Lafarge UHPC</td>
<td>43 (HM)</td>
<td>STA-02</td>
</tr>
<tr>
<td>+ steel fiber</td>
<td></td>
<td>STA-03</td>
</tr>
<tr>
<td></td>
<td></td>
<td>STA-04</td>
</tr>
<tr>
<td></td>
<td></td>
<td>MONO-03</td>
</tr>
<tr>
<td>Lafarge UHPC</td>
<td>43 (HM)</td>
<td>STA-05</td>
</tr>
<tr>
<td>+ Kuraray PVA</td>
<td>12 (HS)</td>
<td>STA-06</td>
</tr>
<tr>
<td></td>
<td></td>
<td>MONO-04</td>
</tr>
</tbody>
</table>

would provide a lower, but acceptable, level of performance. Therefore another six specimens were fabricated using Ductal with PVA fibers as the shear key material. Three specimens were tested using a high moment demand in the shear key — two static and one fatigue. The fatigue tests are also labeled as mono because following each fatigue test a monotonic static test was performed on each specimen. The remaining three specimens were tested using a high shear demand in the shear key — two static and one fatigue.

Described above is the chronological history of shear key material selection. This experiment matrix is illustrated in Table 3.5. Next, detailed information from specimen casting to testing will be presented both horizontally between different groups of specimens, and vertically from static test to fatigue test within each group.

### 3.7 Shear Key Casting and Material Properties

The casting and curing of the precast concrete pieces was provided by Metromont located in Greenville, SC (see Figure 3.16). The specified 28-day design compres-
sive strength of concrete was 6 ksi. After delivery of the precast concrete pieces to Clemson, the shear keys were cast at the Wind and Structural Engineering Research Facility. The next section will focus on the following aspects: mixture proportions of shear key material, casting and curing of shear key and cylinders, and material properties. Specimen testing will be discussed in the next subsection.

![Slab casting at Metromont](image)

Figure 3.16: Slab casting at Metromont

### 3.7.1 Mixture proportion

The material proportions for each shear key material combination are listed in Table 3.6. For the group of Quikrete with PVA specimens, the water amount was determined through some experiments, as mentioned before, so that a balance can be achieved between workability and compressive strength. For the two groups of UHPC mixes, the material proportions were provided by Lafarge. An extremely low water to cement ratio was possible by using a high range water reducer (HRWR). A typical UHPC mixture proportion is listed in Table 3.7 (Graybeal, 2006). In the mixture, the largest granular particle is fine sand with a dimension between 150 and 600 \( \mu m \). The second largest particle is Portland cement with an average diameter of 15 \( \mu m \), followed by crushed quartz, which has an average diameter of 10 \( \mu m \). The smallest
Table 3.6: Mixture proportions of shear key material

<table>
<thead>
<tr>
<th>ID</th>
<th>Material Combination</th>
<th>Fiber Dimension (in.) (diameter × length)</th>
<th>Mix Design Ratio by Weight (Pre-mix:Water:Fiber:HRWR)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Grout with PVA fiber</td>
<td>0.001496 × 0.375</td>
<td>50.00 : 12.52 : 0.75 : 0.00</td>
</tr>
<tr>
<td>2</td>
<td>UHPC with steel fiber</td>
<td>0.00800 × 0.500</td>
<td>50.00 : 2.96 : 3.55 : 0.68</td>
</tr>
<tr>
<td>3</td>
<td>UHPC with PVA fiber</td>
<td>0.007874 × 0.750</td>
<td>50.00 : 3.53 : 0.87 : 0.68</td>
</tr>
</tbody>
</table>

Table 3.7: Typical proportions of UHPC (per yd³ of UHPC) (Graybeal, 2006)

<table>
<thead>
<tr>
<th>Material</th>
<th>Amount (lb.)</th>
<th>Percent by Weight</th>
</tr>
</thead>
<tbody>
<tr>
<td>Portland Cement</td>
<td>1200.0</td>
<td>28.5</td>
</tr>
<tr>
<td>Fine Sand</td>
<td>1720.0</td>
<td>40.8</td>
</tr>
<tr>
<td>Silica Fume</td>
<td>390.0</td>
<td>9.3</td>
</tr>
<tr>
<td>Ground Quartz</td>
<td>355.0</td>
<td>8.4</td>
</tr>
<tr>
<td>Super plasticizer</td>
<td>51.8</td>
<td>1.2</td>
</tr>
<tr>
<td>Accelerator</td>
<td>50.5</td>
<td>1.2</td>
</tr>
<tr>
<td>Steel Fibers</td>
<td>263.0</td>
<td>6.2</td>
</tr>
<tr>
<td>Water</td>
<td>184.0</td>
<td>4.4</td>
</tr>
</tbody>
</table>

particle is silica fume. The large quantity of fine sand can help reduce the quantity of cement. Dimensionally speaking, steel fiber is the largest material in the matrix. The dimension and quantity of steel fibers are determined in a way so that the steel fibers can effectively control cracking, and increase the tensile capacity and toughness of the material (Graybeal and Hartmann, 2003). In this typical mixture, two percent by weight of steel fibers with a diameter of 0.008 in. and a length of 0.5 in. are used.

3.7.2 Preparation and shear key material mixing

Within a couple of days of casting, the shear key interface of the precast concrete pieces were sand blasted at the casting yard to roughen the surface for improved bond between the shear key material and the precast concrete. Prior to
placing the shear key, the interface was washed to clean the surface as shown in Figure 3.17a. Also prior to the casting, strain gauges were attached to the rebar and the lead wires were carefully routed through the side faces of the shear key (see Figure 3.17b). A couple of hours before shear key casting, shear key interfaces were rinsed with water using a sprayer and kept wet before casting to control the absorption of water from the shear key material into the precast during the casting of the shear key. The mixing of grout with PVA fibers was quite conventional, and took about five minutes to mix and maintain workability for about ten minutes. Compared to conventional concrete, mixing UHPC requires increased energy input. Therefore a high-energy mixer was used to mix the Ductal (see Figure 3.17c). On mixing days with elevated ambient air temperatures, ice was used rather than water to keep the mix cool during mixing.

3.7.3 Workability

In the two UHPC mixes, one reason for the high compressive strength is a very low water to premix ratio. To create acceptable workability and flow, a high range water reducer is added to the mix. This can impact the compressive strength along with the workability. Therefore an optimum amount of HRWR needs to be used for good workability. There are several distinct phase changes in the UHPC material during the mixing. Shown in Figure 3.18a is the UHPC material near the completion of the mixing. At this point in time, the material is very sticky, but flows when placed into the formwork. To measure the flowability of the mix, a flow table test similar to that described in ASTM C1437 was used to measure the rheological properties of the UHPC (see Figure 3.18b) (ASTM, 2007b). When casting the shear key, there was no need to vibrate or even rod the material during or after individual lifts.
3.7.4 Curing

Curing is very important for enhancing material properties for cementitious materials. Since UHPC has a very low water to cement ratio, it is very important to seal the top surface of the uncured UHPC with an impermeable layer immediately after casting to avoid evaporation of the water from the surface layer. If sealing is delayed too long, the surface layer will not have enough water for hydration, which will lead to self-desiccation, and subsequent autogenous shrinking, cracking, and poor long-term durability. In the case studied, immediately after casting, specimens were
sealed using a plastic film for three days (see Figure 3.19). Similar to the specimens, cylinders were cured in molds which were sealed for three days and then cured out of the molds until testing of the cylinders.
3.7.5 Cylinder tests

Based on the properties desired, the shear key materials were tested for compressive strength (ASTM C39), split tensile strength (ASTM C496), and bond performance. In the bond tests, the cylinders with a flat bond surface were subjected to direct tension, and the cylinders with a sloped bond surface were subjected to compression (slant shear test). For each test, the cylinders were tested at certain ages like 4-day, 7-day, 14-day, 28-day, and also on the initiation of either a static or fatigue test of a shear key specimen. Since UHPC has a very high compressive strength, testing a 6 in. by 12 in. cylinder would require a very high capacity test machine. According to Graybeal (2006) and Graybeal and Davis (2008), a decreased cylinder size of 3 in. by 6 in., and an increased loading rate of 150 psi/sec are acceptable. In this research, only 3 in. by 6 in. cylinders were used. Considering there were not many cylinders to be tested and it would be better to keep the loading rate the same for both the UHPC mix and the grout mix, the loading rate was kept within the range specified by ASTM C39, which is 200 lbs./sec to 300 lbs./sec for a 3 in. x 6 in. cylinder.

In the direct tensile bond test (see Figure 3.20), the ends of the cylinders were epoxied to steel end plates that could be attached to the base platen and crosshead of a UTM machine. A displacement rate of 0.026 in./min was applied to keep the strain rate within the acceptable range. Knowing that there are many drawbacks with the direct tensile test like the alignment issue and the epoxy issue, this test is mainly for a bonding performance (failure modes) study. In addition to the tests mentioned above, pull-off tests according to ASTM C1583 were performed at the same ages to test the bond between the UHPC with PVA mix with the precast concrete (see Figure 3.20) (ASTM, 2004).
3.7.6 Cylinder test results

The compressive strengths from cylinder tests, as illustrated in Figure 3.21, show that the UHPC with steel group had the highest compressive strength at the same cylinder age, followed by the UHPC with PVA group, and then the grout group. This relationship is mainly influenced by the water to premix ratio. The 4-day and 28-day compressive strengths and splitting tensile strengths for each material combination are listed in Table 3.8. Cylinder compressive strengths during the static and fatigue tests are listed in Table 3.9 and will be later referenced in the subsection of experimental result analysis.

For the UHPC groups, all the bond tests including direct tensile test, slant shear test, and pull-off test show that most of the specimens failed in the concrete (see Figures 3.22-3.23). The corresponding average pull-off strengths of both the composite cylinder and the pure concrete substrate are listed in Table 3.10. A general trend is that at the same age, the average pull-off tensile strength of the concrete
Table 3.8: Cylinder test results

<table>
<thead>
<tr>
<th>Mixture ID</th>
<th>Compressive Strength (psi)</th>
<th>Splitting Tensile Strength (psi)</th>
<th>4−day</th>
<th>28−day</th>
<th>4−day</th>
<th>28−day</th>
</tr>
</thead>
<tbody>
<tr>
<td>UHPC with steel</td>
<td>16290</td>
<td>26970</td>
<td></td>
<td></td>
<td>2665</td>
<td></td>
</tr>
<tr>
<td>UHPC with PVA</td>
<td>13820</td>
<td>21070</td>
<td></td>
<td></td>
<td>1735</td>
<td></td>
</tr>
<tr>
<td>Grout with PVA</td>
<td>N/A</td>
<td>7400</td>
<td></td>
<td></td>
<td>N/A</td>
<td>1270</td>
</tr>
</tbody>
</table>

Table 3.9: Compressive strengths during tests

<table>
<thead>
<tr>
<th>Specimen ID</th>
<th>Precast (psi)</th>
<th>Shear Key (psi)</th>
</tr>
</thead>
<tbody>
<tr>
<td>STA-01</td>
<td>7500</td>
<td>7500</td>
</tr>
<tr>
<td>STA-02</td>
<td>8280</td>
<td>25490</td>
</tr>
<tr>
<td>STA-03</td>
<td>9530</td>
<td>15190</td>
</tr>
<tr>
<td>STA-04</td>
<td>9680</td>
<td>18510</td>
</tr>
<tr>
<td>STA-05 and 06</td>
<td>8730</td>
<td>22330</td>
</tr>
<tr>
<td>MONO-02</td>
<td>8070</td>
<td>26970</td>
</tr>
<tr>
<td>MONO-03</td>
<td>6530</td>
<td>21070</td>
</tr>
<tr>
<td>MONO-04</td>
<td>6570</td>
<td>N/A</td>
</tr>
<tr>
<td>MONO-01(redo)</td>
<td>6280</td>
<td>7670</td>
</tr>
</tbody>
</table>

substrate is much higher than that of the concrete in the bond test. The concrete subjected to pull-off test is from a wasted specimen, which was put outside for several months. This difference in tensile strengths could result from the long-time vibration when drilling the composite cylinder, which weakened the bottom concrete. For the grout group, the typical failure is at the bond in both the direct tensile test and slant shear test (see Figure 3.22c and Figure 3.22d). For the UHPC groups, the slant shear test results indicate that although the cylinders failed in the concrete, their strengths are slightly higher compared with those of pure concrete (see Figure 3.24), which may result from the restraint of the UHPC material.
Figure 3.21: Cylinder compressive strengths during specimen tests

Table 3.10: Tensile strengths during pull-off tests of UHPC with PVA combination

<table>
<thead>
<tr>
<th>Shear key age (day)</th>
<th>6</th>
<th>14</th>
<th>28</th>
<th>57</th>
</tr>
</thead>
<tbody>
<tr>
<td>Composite cylinder (psi)</td>
<td>415</td>
<td>297</td>
<td>380</td>
<td>365</td>
</tr>
<tr>
<td>Concrete substrate (psi)</td>
<td>500</td>
<td>393</td>
<td>440</td>
<td>420</td>
</tr>
</tbody>
</table>

3.8 Specimen Testing

For each group of specimens in the experiment matrix mentioned in Table 3.5, both static tests and fatigue tests were conducted. For each specimen, the concrete casting date, shear key casting date, and specimen testing date are summarized in Table 3.11 for later reference. The results from the static test are not only used in the fatigue load determination, but also used in the calibration of the shear key finite element model. The results from a fatigue test can highlight the performance of the shear key joint durability, and also validate the fatigue load exerted on it.
The slab is simply supported on two steel rods covered by a dense plastic tube that are aligned parallel to the shear key. In the high moment test, the slab has a span of 86 in., with the right support located 43 in. from the centerline of the shear key. In the high shear test, the slab span is 55 in., with the right support located 12 in. from the shear key centerline. The load strip is a square steel tube that runs the full width of the specimen and is located 8 in. left of the centerline of the shear key. A 160-kip actuator was used for static tests and a 35-kip actuator was used for the fatigue tests. Both the static and fatigue tests were controlled using the Multipurpose
Figure 3.23: Pull-off test results for the UHPC with PVA combination

Figure 3.24: Restraining effect of UHPC
Table 3.11: Specimen casting date and testing date

<table>
<thead>
<tr>
<th>Specimen ID</th>
<th>Precast</th>
<th>Shear Key</th>
<th>Specimen Testing Day</th>
</tr>
</thead>
<tbody>
<tr>
<td>STA-01</td>
<td>01/25/12</td>
<td>01/31/12</td>
<td>02/17/12</td>
</tr>
<tr>
<td>FAT-01</td>
<td>01/25/12</td>
<td>01/31/12</td>
<td>03/01 – 03/26/12</td>
</tr>
<tr>
<td>STA-02</td>
<td>04/11/12</td>
<td>04/27/12</td>
<td>05/15/12</td>
</tr>
<tr>
<td>FAT-02</td>
<td>04/11/12</td>
<td>04/27/12</td>
<td>05/23 – 06/16/12</td>
</tr>
<tr>
<td>STA-03</td>
<td>03/29/12</td>
<td>07/26/12</td>
<td>08/01/12</td>
</tr>
<tr>
<td>STA-04</td>
<td>03/29/12</td>
<td>07/26/12</td>
<td>08/09/12</td>
</tr>
<tr>
<td>STA-05</td>
<td>04/16/12</td>
<td>07/26/12</td>
<td>09/21/12</td>
</tr>
<tr>
<td>STA-06</td>
<td>04/16/12</td>
<td>07/26/12</td>
<td>09/27/12</td>
</tr>
<tr>
<td>FAT-03</td>
<td>06/12/12</td>
<td>07/26/12</td>
<td>08/17 – 09/10/12</td>
</tr>
<tr>
<td>FAT-04</td>
<td>06/12/12</td>
<td>07/26/12</td>
<td>10/11 – 11/02/12</td>
</tr>
<tr>
<td>FAT-01(redo)</td>
<td>09/17/12</td>
<td>10/16/12</td>
<td>11/17 – 12/11/12</td>
</tr>
</tbody>
</table>

Software (MPT). Data from all the specimen sensors, including strain transducers, strain gauges, string pots and LVDTs were acquired using LabVIEW 8.2.

Before specimen testing, all the sensors were calibrated. The typical sensor layout is shown in Figure 3.25 and Figure 3.26. The scale factor obtained for each sensor would later be applied to the experimental data. After sensor calibration, for each group of specimens, the static test was conducted first, the results of which were processed for the shear key rotational stiffness and vertical stiffness needed in the later fatigue demand determination procedure. Before each fatigue test, the fatigue actuator was tuned to ensure the feedback signal reflected accurately the command signal. The details of the tests will be discussed in the following paragraphs.

3.8.1 Static test

About two hours were required to complete a high moment test and 4.5 hours to complete a high shear test. The static tests are composed of several load levels.
Figure 3.25: Final sensor layout (part 1)

Figure 3.26: Final sensor layout (part 2)

Nomenclature - strain gauges:
- SG-LBT/B: SG-left back top/bottom
- SG-LCT/B: SG-left center top/bottom
- SG-RCT/B: SG-right center top/bottom
- SG-RBT/B: SG-right back top/bottom
At each level, the specimen was loaded and unloaded twice until the final load level. In the second cycle at each load level, before unloading, the load was held at the specified load level, during which time crack developments were marked. There could be a drop of about 0.2 kips to 0.3 kips during the holding phase. To keep the load strip in contact with the slab, the bottom load limit is set to be 0.1 kips, rather than a complete unloading of the specimen. The test was stopped when a large deformation happened under a small load increment. The test was under displacement control, with a displacement rate of 0.1 in/min, and the unloading rate of 0.5 in/min. The rate is determined in a way such that the strain rate is acceptable and the test can be finished within a reasonable time frame. Data was acquired throughout the whole process. The sampling rate was specified to provide enough data to capture the response without being overwhelmed with large data files. Refer to Figure 3.27 for a typical loading protocol.

![Figure 3.27: Typical loading protocol for static test](image)

Figure 3.27: Typical loading protocol for static test
3.8.2 Fatigue test

3.8.2.1 Test description

In order to capture the performance of the shear key during its expected service life, the fatigue test was determined to need 10 million cycles. The specimen was tested under a sine-wave load, the frequency of which was determined considering both the system response and the time available. For the high moment test, the frequency applied was 5 Hz, and for the high shear test, it was found that 6 Hz could be used. Generally a fatigue test took 22 to 25 days to complete the 10 million cycles. After building a water pond reservoir above the shear key and the surrounding area, another 2 million cycles of load were applied to the specimen. During these 2 million cycles the shear key was monitored for leakage through the shear key and the interface. After the pond test, the specimen was loaded monotonically until a large deformation was observed under a slight load increment. The whole process of applying the monotonic loading only took a few minutes. The monotonic test was similar to the static test with respect to loading rate and data sampling rate. Different from the static test and monotonic test, the fatigue test was under load control. The MTS software was configured to acquire data (actuator load and actuator extension) at a rate of 10 samples / cycle for a consecutive 2 seconds after every 5000 cycles. The data from the other sensors were sampled at approximately the same rate in LabVIEW but roughly on a daily base for a short period of time. Refer to Figure 3.28 for a typical loading protocol.

3.8.2.2 Fatigue load determination

At the time of fatigue tests, the full stiffness matrix for a shear key frame element had not been obtained. From the static test results, only the shear key
rotational and translational stiffness were available. With these two stiffness terms and other terms estimated, the input parameters for a shear key frame element in the bridge FE model that will be talked about in Chapter 5 was determined. The stiffness from the experimental data are elastic-cracked stiffness. To be consistent, the post-cracked stiffness is applied to the deck concrete by assigning a value of 0.35 to both the modifier for transverse bending and the modifier for transverse bending induced torsion. This is because the longitudinal cracks in the deck will not only affect the transverse bending stiffness, but also the torsional stiffness.

The bridge model used for determining fatigue demands is NEXT-8 with a span length of 40 ft. The selection was made based on the study of Funcik (2011), which concluded that the NEXT-8 with 40 ft-span length gave larger transverse demands than NEXT-6 with the same span length. The fatigue load was the design truck specified in LRFD Art. 3.6.1.2.2, (AASHTO, 2012) but with a constant spacing of 30 ft. between the two 32 kip axles. The load was modeled as patch loads (see Section

Figure 3.28: Typical loading protocol for fatigue test
5.2.2.4) and exerted between the inner faces of the parapets in order to obtain the critical demands in the shear key. The critical positive moment demand was then used to calculate the unfactored external force in the high moment test, and the critical shear demand was used to determine the unfactored external force in the high shear test.

The final load was calculated by multiplying a load factor of 0.75 for the fatigue II limit state, and an impact factor of 1.33 for the first load level, and 1.75 for the second load level. The impact factor used for each load level was determined based on the following reasons. According to AASHTO LRFD Table 3.6.2.1-1, for deck joints, the impact factor is 1.75 for all the limit states. It is understood that the intent of this provision is to address the impact upon transverse expansion joints. However, in the case studied, the shear key is a joint that is longitudinal to the bridge centerline and will not likely experience the same impact that a transverse joint will. Therefore, the 1.75 factor is checked and considered to be a conservative upper limit. This factor was used for the second load level. For all other components, AASHTO LRFD Table 3.6.2.1-1 specifies an impact factor of 1.15 for the fatigue limit state. Again, considering that the shear key is a joint, the impact factor should be higher than 1.15. Therefore to be conservative, an impact factor of 1.33 was applied for the first load level. The calculated load levels for each specimen are listed in Table 3.12, and the applied load levels are listed in Table 3.13.

3.8.2.3 Fatigue load evaluation

With the full shear key stiffness matrix determined later for each material combination (see Section 5.3.4) under high moment test, the ‘correct’ fatigue demand ranges were determined for FAT-02, FAT-03, and FAT-01 (redo), with the upper boundary based on the pre-cracking stiffness, and the lower boundary based on the
Table 3.12: Fatigue load determination on shear key based on shear key stiffness

<table>
<thead>
<tr>
<th>ID</th>
<th>Stiffness from Static Test</th>
<th>Stiffness in FE model</th>
<th>Fatigue Load</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>$K_t$ (k/in)</td>
<td>$K_r$ (k-in/rad)</td>
<td>$K_t$ (k/in)</td>
</tr>
<tr>
<td>FAT-01</td>
<td>376.5</td>
<td>83879.0</td>
<td>56.5</td>
</tr>
<tr>
<td>FAT-02</td>
<td>575.9</td>
<td>581473.5</td>
<td>86.4</td>
</tr>
<tr>
<td>FAT-03</td>
<td>420.0</td>
<td>100000.0</td>
<td>63.0</td>
</tr>
<tr>
<td>FAT-04</td>
<td>550.0</td>
<td>117890.0</td>
<td>82.5</td>
</tr>
</tbody>
</table>

Table 3.13: Fatigue load applied in the test

<table>
<thead>
<tr>
<th>Specimen ID</th>
<th>Load level Applied (Cycle/million)</th>
</tr>
</thead>
<tbody>
<tr>
<td>FAT-01</td>
<td>8.7(10.6)</td>
</tr>
<tr>
<td>FAT-02</td>
<td>8.7(5.9) 9.7(4.1)</td>
</tr>
<tr>
<td>FAT-03</td>
<td>4.9(1.8) 6.5(0.4) 5.4(0.4) 5.6(4.2) 7.3(3.3)</td>
</tr>
<tr>
<td>FAT-04</td>
<td>8.1(5.0) 10.6(5.0)</td>
</tr>
<tr>
<td>FAT-01(redo)</td>
<td>5.3(5.0) 7.0(5.0)</td>
</tr>
</tbody>
</table>

post-yield stiffness (see Figure 3.29a-c). These two bounding stiffness matrix were obtained as follows. For each material combination, the stiffness matrix in Section 5.3.4 was obtained based on the calibration of the elastic-cracked joint rotational stiffness. This is the reference matrix. From the static test results, the pre-cracking and post-yielding joint rotational stiffness are also known for each material combination. Each term in the reference matrix was then scaled up based on the pre-cracking and post-yielding joint rotational stiffness into two stiffness matrix, representing the two boundaries.

For FAT-04 that is under high shear test, the joint FE model was not calibrated because the shear effect on the joint rotational stiffness in the experiment cannot
be neglected; while in the FE model, the joint was under pure bending. To be conservative, the fatigue demand of FAT-04 was evaluated based on the full joint stiffness of STA-02 (the UHPC with steel combination) (see Figure 3.29d).

In Figure 3.29, the two close dotted lines represent the two interfaces of a shear key. Comparing with the demand range on the shear key for each material combination, the fatigue demands applied as indicated by the horizontal solid lines for FAT-02 and FAT-04 are conservative. For FAT-01 and FAT-03, the demands are all within the range.

Figure 3.29: Fatigue load evaluation
Chapter 4

Experimental Result Analysis

4.1 Result Analysis—Performance at Strength Level

This section will focus on the analysis of the results from the performance at strength level (static test and monotonic test). Analyzed specimen performances at the strength level include crack propagation, failure mechanism and final capacity, stiffness degradation, toughness and ductility.

4.1.1 Crack propagation during strength test

4.1.1.1 Before test

Before any live load was exerted, cracks were observed on the top and at the bottom of the shear key, and sometimes along the interfaces of the shear keys (Figure 4.1). This phenomenon happened for all the shear key material combinations except for the UHPC with steel fiber combination. The cracks are deemed to result from drying shrinkage and the curing procedure applied. Cracks within the shear key were usually transversal (orthogonal to the load strip), indicating that there is
tension longitudinally, and this tension is possibly due to the volume reduction in the longitudinal direction.

(a) Cracks on the top of the shear key (b) Cracks at the bottom of the shear key

Figure 4.1: Cracks due to shrinkage before tests

4.1.1.2 Bottom view

Since the load was uniformly distributed along the width of the shear key specimen, the specimen behaved as a one-way slab during the tests. In the static tests, the first crack usually occurred in the precast close to the load strip where maximum moment existed. The first crack occurred under a load level between 8 and 12 kips, or under a maximum moment between 166 and 250 kip-in. for the high moment tests. For high shear tests, this load level is between 12 and 16 kips, or a maximum moment between 153 and 204 kip-in. At the beginning, the direction of the crack is longitudinal, which is orthogonal to the direction of flexural tension. As the load level increased, the cracking zone spread. New cracks formed alongside the first crack, and old cracks propagated. When the load level was high enough, transverse cracks were formed under the U-bars, which were spaced 8 in. on-center. A grid-like pattern in the precast was formed in this way in most of the strength
tests (see Figure 4.2). The longitudinal cracks resulted from flexural tension, while the transverse cracks directly under the U-bar were formed due to bond splitting. Such cracks did not appear during fatigue tests because the service loads were not big enough to result in high bond stress. There was no regular cracking pattern in the shear key, which was possibly due to the presence of fibers. Since the direction of fibers can make a big difference in tensile strength of the specimen, it can also influence the direction of cracks. There was no visible crack in the shear key made of UHPC and steel fiber at any stage during the monotonic test.

Figure 4.2: Cracking pattern after strength test-bottom view (STA-04)

4.1.1.3 Side view

On the side of the slab, a typical crack propagation in the precast is first the cracks grew vertically due to flexural tension and then they changed direction towards the load strip due to shear as they approached the top edge of the slab (see Figure 4.3). This type of crack is referred to as flexural shear crack. The inclined crack is fairly pronounced in the high shear tests. Within the shear key, due to its special geometry and the existence of fibers, cracks sometimes propagated along the interface of the key. The cracks also sometimes started from the bond and then switched their
direction either towards the key or the precast while sometimes the cracks grew into the bond from elsewhere. Again due to the influence of fibers, there was no regular pattern in the shear key.

![Figure 4.3: Cracking pattern after strength test-side view](image.jpg)

(a) High moment test (STA-04)  (b) High shear test (STA-05)

4.1.1.4 Bond crack

During the static tests, the first interface crack happened at the left bond at a moment about 121 kip-in, 277 kip-in, 183 kip-in, 247 kip-in, 285 kip-in, and 285 kip-in for specimens STA-01 to STA-06, respectively (see Table 6.5 for test specimen designations). These cracking moments were obtained from the moment-curvature curves based on LVDT readings and will be discussed later in this chapter. It can be observed from Figure 4.4 that STA-01 and STA-03 gave the lowest bond cracking strength. This is due to the fact that these two specimens cracked first at the bond interface during the tests. For other specimens, bond interface cracked at higher moment levels than the concrete cracking strength. The cracking moments for STA-02, STA-04, STA-05, and STA-06 are quite close. At the similar joint ages, STA-02 (UHPC with steel combination) gave the highest bond cracking strength, STA-
01 (grout with PVA combination gave the lowest, and STA-04 (UHPC with PVA combination) gave a bond cracking strength in-between. Compared with STA-04, the higher bond strength of STA-02 could result from the lower water volume or simply its older shear key age at test. For the UHPC with PVA combinations, the development of cracking strength with time is observed. There is a distinct improvement in bond cracking strength from age 6 to age 14 of the joint material, and a slow increase from age 14 to age 57. If the bond cracking strength at age 57 is taken as the full strength, the bond cracking strength achieves 64.2 percent at age 6, and 86.7 percent at age 14. Compared with the compressive strength, while there is a 21.9 percent of increase in compressive strength from STA-03 to STA-04, there is a 46.8 percent of increase in bond cracking strength. This indicates that the joint age between 6 day and 14 day is an important transition period to the bond cracking strength development for UHPC combinations. This aspect should be considered in ABC when the bridge was open to the traffic 4 days after the joint was cast. A selective open to light weight vehicles would be preferred in case that the heavy vehicles like trucks may crack the bond of joint at an early age.

4.1.2 Failure mechanism and final capacity

4.1.2.1 Failure mechanism

For all the specimens tested, the final failure mode was ductile regardless of whether it is high moment test or high shear test. The failure mode is a flexural failure for the high moment test, while for the high shear test, it is considered to be a mixture of a flexural failure and a shear failure. This conclusion is made by observing the cracks from the high shear tests, in which the cracks due to shear were quite wide and close to the top of the slab. Failure is caused by the widening of cracks
especially under the load strip and at the left bond, which led to increasing strain of rebar until they all yielded, at which time, large deflection occurred when the load nearly stopped increasing.

4.1.2.2 Critical cross section identification

In order to determine the final moment and shear capacity, it is important to identify the critical section. For the current test configuration, there are two possible critical cross sections, one of which is under the load strip which has the largest moment, and the other at the left shear key interface. Two reasons can explain why the left shear key interface can be critical. First, depending on the strength and interface properties of concrete slab, the properties of shear key material, the age of the shear key when tested, and the shrinkage of the shear key, the bond can be the weakest part in the whole system. Second, in contrast to the multiple cracks formed under the load strip, there was only one crack at the interface once it was
formed (see Figure 4.5). Since multiple cracks dissipate energy much better than a single crack does, with the same amount of energy, a single crack can get much wider than multiple cracks. A wider crack can lead to larger strain in the reinforcing steel; therefore the bond interface is a potential critical cross section. It should be noted that in no case was the shear key weaker than the bond, which can also be seen the cylinder bond test in direct tension. Hence the possibility that there may be a critical cross section in the shear key is excluded. Based on direct observation and rebar strain, it is concluded that the specimens with the grout material combination in the high moment test, and the specimens in the high shear tests failed at the left bond, and others failed in the precast close to the load strip (refer to Table 4.1 for details). Keep in mind that all specimens were loaded past the design loads up to failure. This simply describes the mode of failure and is not indicative of insufficient capacities.

![Figure 4.5: Multiple cracks under load strip](image)

4.1.2.3 Capacity determination

After determining the critical cross section, the moment and shear capacities at the cross section were calculated, accounting for the dead load of the concrete slab,
Table 4.1: Specimen capacity summary

<table>
<thead>
<tr>
<th>Shear Key Mixture and Loading Condition</th>
<th>Specimen ID</th>
<th>Critical Cross Section</th>
<th>Max. Moment kip-in</th>
<th>Max. Shear kip</th>
</tr>
</thead>
<tbody>
<tr>
<td>Grout with PVA (HM)</td>
<td>STA-01</td>
<td>Left bond</td>
<td>763.5</td>
<td>15.6</td>
</tr>
<tr>
<td></td>
<td>MONO-01</td>
<td>Left bond</td>
<td>690.6</td>
<td>14.0</td>
</tr>
<tr>
<td></td>
<td>MONO-01(redo)</td>
<td>Left bond</td>
<td>674.8</td>
<td>13.7</td>
</tr>
<tr>
<td>UHPC with steel (HM)</td>
<td>STA-02</td>
<td>Under load strip</td>
<td>742.5</td>
<td>20.7</td>
</tr>
<tr>
<td></td>
<td>MONO-02</td>
<td>Under load strip</td>
<td>745.4</td>
<td>20.8</td>
</tr>
<tr>
<td>UHPC with PVA (HM)</td>
<td>STA-03</td>
<td>Under load strip</td>
<td>736.7</td>
<td>20.7</td>
</tr>
<tr>
<td></td>
<td>STA-04</td>
<td>Under load strip</td>
<td>738.0</td>
<td>20.7</td>
</tr>
<tr>
<td></td>
<td>MONO-03</td>
<td>Under load strip</td>
<td>727.0</td>
<td>20.4</td>
</tr>
<tr>
<td>UHPC with PVA (HS)</td>
<td>STA-05</td>
<td>Left bond</td>
<td>650.0</td>
<td>40.9</td>
</tr>
<tr>
<td></td>
<td>STA-06</td>
<td>Left bond</td>
<td>634.1</td>
<td>40.2</td>
</tr>
<tr>
<td></td>
<td>MONO-04</td>
<td>Left bond</td>
<td>543.8</td>
<td>34.6</td>
</tr>
</tbody>
</table>

which was assumed to have a uniform density of 150 lb/ft\(^3\). It should be noted that for most of the specimens, final capacities were not achieved, which can be observed from the load-displacement curve. Tests were stopped when large deformation occurred with a slight increase of load. The maximum capacities of all the slabs at the corresponding critical cross sections are tabulated in Table 4.1. The capacities of all the 11 slabs determined from experiments will later be compared with the shear-moment interaction diagram based on modified compression field theory (Collins and Mitchell, 2001). For specimens that failed at the joint interface, rebar strain indicated that both layers of rebar at the failure joint interface all yielded, which means the anchorage length of the U-bar is sufficient to achieve full-capacity joint.

4.1.2.4 Shear-moment interaction diagram

The shear-moment interaction diagram for a cross section in the concrete slab is determined based on the modified compression field theory by Collins and Mitchell (Collins and Mitchell, 2001), which can be applied to beam-like elements that conform
to the assumption that plane section remains plane after loading (AASHTO LRFD 5.8.1). Compared with the classical compression field theory, the modified theory takes into account of the concrete principal tensile stress after cracking, which is deemed to be more realistic when explaining the shear failure mechanism. In order to calculate the shear capacity of a cross section, combined equilibrium, compatibility, and constitutive models need to be applied. Due to the complexity involved in the calculation, design aids were provided by Collins and Mitchell to reduce the effort. And these aids are available in AASHTO LRFD Appendix B5. According to this theory, for a slab cross section without either transverse reinforcement or prestressed reinforcement, the shear capacity of a cross section is provided only by the nominal shear strength of the concrete. And the procedure to determine this capacity can be stated as follows:

**Step 1:** Determine the crack spacing parameter $S_{xe}$ using the formula:

$$S_{xe} = S_x \frac{1.38}{a_g + 0.63} \leq 80 \text{ in}$$

(4.1)

where $a_g$ is the maximum aggregate size (in.), and 0.75 in. is used in this case, $S_x$ is the minimum of the effective shear depth and the maximum distance between layers of longitudinal crack control reinforcement, which in this case is 4 in. The reinforcement area in each layer should not be less than $0.003d_eS_x$. The minimum shear depth $d_e$ is defined as the distance, measured perpendicular to the neutral axis, between the resultants of the tensile and compressive forces due to flexure, but it needs not be taken less than the greater of $0.9d_e$ or $0.72h$, where $d_e$ is the distance from the extreme compressive fiber to the centroid of the tensile force in the tension reinforcement (4.75 in. in this case), and $h$ is
the height of the specimen (8 in. in this case). Therefore \( d_v \) is calculated to be 5.76 in., \( S_x \) is taken to be 4 in., and \( S_{xe} \) is determined to be 4 in..

**Step 2:** Select a range of longitudinal strain \( \epsilon_x \), for instance from 0 to 2000 microstrain and go to AASHTO LRFD Table B5.2 – 2. With the \( S_{xe} \) determined in step 1, a range of \( \theta \) and \( \beta \) can be calculated corresponding to the values of \( S_{xe} \) selected, where \( \theta \) is the angle of inclination of diagonal compressive stresses, and \( \beta \) is a factor indicating ability of diagonally cracked concrete to transmit tension and shear.

**Step 3:** The nominal shear capacity \( V_n \) of the cross section can be calculated according to AASHTO LRFD 5.8.3.3:

\[
V_n = 0.0316 \beta \sqrt{f'_c b_v d_v}
\]  

(4.2)

where \( f'_c \) is the specified compressive strength of concrete (ksi) (6 ksi in this case), and \( b_v \) is the effective web width taken as the minimum web width within the depth \( d_v \) (40 in in this case).

**Step 4:** With several shear capacity values being determined, the corresponding moment capacity can also be calculated according to AASHTO LRFD Table B5.2 – 2:

\[
\epsilon_x = \frac{M_u}{d_v} + 0.5|V_u| \cot \theta
\]

(4.3)

where \( M_u \) is the factored moment and should not be less than \( V_u d_v \), \( V_u \) is the factored shear force, \( E_s \) is the Young’s modulus of nonprestressed steel, \( \epsilon_x \) is the normal strain and \( A_s \) is the area of nonprestressed steel on the flexural tension side of the member at the section under consideration. The resistance factor is 95.
chosen to be unity. Several combinations of $M_u$ and $V_u$ calculated following the above procedure can give a shear failure line.

**Step 5:** The yielding of longitudinal reinforcement before failure can reduce the shear capacity of the slab. Therefore when determining the moment failure line, the minimum longitudinal reinforcement requirement should be considered. The formula according to AASHTO LRFD 5.8.3.5−1 is simplified for the case studied as:

\[ A_s f_y = \frac{M_n}{d_v} + V_n \cot \theta \]  

(4.4)

With other parameters all known, $M_n$ can be plotted against $V_n$. Since there is no transverse or prestressed reinforcement, the shear capacity is zero at pure moment. In this way a moment failure line is generated (see Figure 4.6).

It should be noted that the maximum moment calculated from the modified compression field theory is much larger than that calculated according to AASHTO LRFD 5.7.3.2 due to different moment arms $d_v$ being used. Although $d_v$ is defined as the distance between the resultant tensile force and the resultant compressive force, it has a minimum limit according to AASHTO LRFD 5.8.2.9, which explains why the moment capacity calculated is larger. At the ultimate state, both layers of rebar are in tension. Therefore half of the shear-induced axial tension is resisted by all the rebar in this case, and the other half is resisted by the compressive zone. When using the formula from AASHTO, the resistance factor is unity, so that it gives the maximum resistance. Moreover, this shear-moment diagram is obtained for the concrete, not for the bond. For some of the slabs that failed at the bond, a diagram with less resistance is supposed to be obtained. This is because if the bond is stronger than the concrete, the slab cannot fail at the bond under the same demand, not to mention that the bond had a smaller demand than the concrete in the current loading configuration.
4.1.2.5 Discussion

For the eleven slabs tested, the maximum shear and moment capacities (considering dead load) they subjected to during the experiments all exceeded what are predicted from the shear moment diagram (Figure 4.6). Even after 10 million cycles, there is still a large reserve capacity. It should be noted that these specimens were tested under a laboratory environment. In real cases, the rebar may become corroded during service, which can reduce the ductility and also the capacity of the slab. The specimens that failed close to the load strip (UHPC with PVA/steel - HM) have close maximum capacities. Fatigue load did not have notable influence on the shear and moment capacities of these specimens. For the specimens failed at the left bond
(grout with PVA - HM, and UHPC with PVA - HS), fatigue tests had a negative impact on these specimens’ capacities.

4.1.3 Stiffness degradation at the strength level

4.1.3.1 Significance

A stiffness comparison between static tests and the corresponding monotonic test is provided in Figures 4.7 and 4.8, which include the rotational stiffness of the shear key and the vertical stiffness under the load strip. A comparison like this can directly tell the stiffness changes from static test to monotonic test, and the property variation from specimen to specimen.

4.1.3.2 Determination

In Figure 4.7, the rotational stiffness comparison between the static tests and the corresponding monotonic test is presented by plotting the moment at the center of shear key against the rotation of the shear key. This rotation is calculated based on LVDT data by subtracting the top LVDT reading (in compression) from the corresponding bottom LVDT reading (in tension), the result of which is divided by the vertical distance between these two LVDTs. The vertical stiffness under the load strip is presented in Figure 4.8 by plotting the load exerted by the load cell against the distance measured from the left string pot, which is close to the load strip.

4.1.3.3 Moment-rotation curve

Generally speaking, the moment-rotation curve in the post-fatigue monotonic test agrees well with the corresponding curves in the static tests, especially for the specimens that failed in the precast concrete (UHPC with PVA/steel -HM). A signif-
significant difference between the curves in the static test with that in the monotonic test is that the static specimens, since they were in a virgin state at the beginning of the test, underwent a large deformation when the bond cracked (i.e. a sudden horizontal shift in the moment-rotation curve), while most of the monotonic specimens did not exhibit this phenomenon.

As far as stiffness at the fatigue load level is concerned, except for MONO-03 (UHPC with PVA -HM), there is a clear drop of rotational stiffness for other monotonic specimens. The high rotational stiffness of MONO-03 within the fatigue load level and the sudden horizontal shift in its moment-rotation curve at the load
Figure 4.8: Translational stiffness degradation under load strip

level of about 310 kip-in indicates that the bond of MONO-03 was sound enough to resist tension after the 10 million cycles’ fatigue test. This high stiffness at the fatigue load level means that the fatigue load exerted was not large enough to crack the bond. Other specimens under monotonic tests do not have this indication of bond cracking shown in their monotonic moment-rotation curves. Later analysis showed that the shear key interfaces for these specimens already cracked at the end of the fatigue tests.
4.1.3.4 Force-displacement curve

Generally the force-displacement curves in the monotonic tests agrees well with the those in the static tests. One factor that can result in the degradation of vertical stiffness under the load strip, is the reduction of effective moment of inertia of concrete due to the crack development. Depending on the compressive strength of concrete (which is related to the tensile strength of concrete) and fatigue load magnitude, the degree of concrete cracking at the end of the fatigue test differs among the tested specimens. Another factor that affects the magnitude of the vertical stiffness under the load strip is the shear key rotational stiffness.

For MONO-02, due to the high fatigue load applied, there were a lot of cracks in the precast concrete at the end of the fatigue test. Also due to the bond cracks at the end of the fatigue test, the initial shear key rotational stiffness was smaller than that of STA-02 (see Figure 4.7b). This drop of effective moment of inertia in the precast concrete and the joint rotational stiffness resulted in a distinct initial stiffness degradation (Figure 4.8b). For MONO-03, the monotonic curve agrees well with the static curves. This is due to the fact that on the one hand, there were only a few cracks in the slab after fatigue test and therefore the effective moment of inertia was not reduced much, and on the other hand, the initial joint rotational stiffness was slightly higher than those in the static tests (see Figure 4.7c). The vertical stiffness degradation resulted from a reduction of effective moment of inertia is compensated by the restraining effect of the shear key. For MONO-04, there were very few cracks developed after the fatigue test, but since there was a drop of shear key rotational stiffness due to bond cracking, the vertical stiffness kept degrading compared with those in the static tests.
4.1.3.5 Variation from specimen to specimen

The main factors that lead to the variations of specimen properties are the strengths of concrete and shear key. In the case of STA-03 and STA-04, both the precast and the shear key were cast on the same dates. The only difference between these two specimens is STA-03 was tested 6 days after the shear key was cast, and STA-04 14 days after the shear key was cast. The concrete compressive strengths were close between these two specimens, while there was a 22 percent increase of shear key material strength from STA-03 to STA-04, which resulted in a 35 percent increase in bond cracking strength from the former to the latter (see Figure 4.7). For STA-05 and STA-06, both the precast elements and the shear keys were cast on the same dates. The former was tested 57 days after the shear key was cast, and the later 63 days after that. The strengths of concrete and shear key between these two specimens during the test days were similar. And we see there is not much variation in the force-deformation curves of these two specimens (see Figure 4.7 and 4.8).

4.1.4 Toughness and ductility

4.1.4.1 Significance

Toughness or the work of fracture, is the energy required to crack a cross section of a material (Gordon, 1991). It is a measure of the material’s capacity to resist crack propagation. It is related to the area under the stress-strain curve. The more ductile a material is, the tougher it is. Concrete has low work of fracture and is sensitive to cracks. Compared with concrete, reinforcing steel is much more ductile and has much higher work of fracture. As such it is necessary to reinforce concrete with steel for tension. The significance of ductility in a structural system can be reflected from the fact that large deformations can happen due to ductility before
the structure collapse. For an indeterminate structure, sufficient ductility leads to demand redistribution and permits the formation of plastic hinges, and therefore increases the capacity of the structure. Hence the larger the ductility, the more safety margin there is.

For specimens that failed at the joint interface (grout with PVA - HM, and UHPC with PVA - HS), fatigue tests reduced the toughness of these specimens (see Figure 4.7). For specimens that failed in the precast (UHPC with PVA - HM, and UHPC with steel - HM), the UHPC with steel combination had less toughness compared with other specimens. While the high stiffness of UHPC with steel combination increased the stiffness of the surrounding precast concrete slab, it reduced the overall toughness of the specimen. In real bridges, this high joint stiffness can also attract more demands to both the joint and the surrounding precast deck. This increased demand with reduced toughness could be a concern with respect to the critical crack length, which is proportional to toughness of the material, and inversely proportional to the square of the stress the material is subjected to. This could result in big cracks in the precast concrete deck and degrade the durability of the deck especially if the subsequent rebar corrosion is considered.

4.1.4.2 Quantification

Ductility can be quantified using ductility index, which is the ratio of the ultimate deformation to the yield deformation. Usually curvature and displacement are used for calculating ductility index (see Equation 4.5 and DispDuctility.

\[ \mu = \frac{\psi_u}{\psi_y} \]  

(4.5)

where \( \psi_u \) is the ultimate curvature, and \( \psi_y \) the yield curvature;
\[ \mu = \frac{D_u}{D_y} \]  (4.6)

where \(D_u\) is the ultimate displacement, and \(D_y\) the yield displacement.

For specimens that failed at the bond (grout with PVA-HM and UHPC with PVA-HS), the ductility indices are calculated using Equation 4.5. For specimens that failed close to the load strip, the ductility indices are determined using Equation 4.6. The curvature is determined by subtracting the top LVDT reading (compression) from the corresponding bottom LVDT reading (tension), the result of which is divided by the distance between the two LVDTs (10.25 in.) and the width of the shear key (8 in). Simply put, the curvature is calculated by dividing the rotation of shear key by the width of the shear key. Strictly speaking, the curvature calculated this way is not the curvature at the failed bond, but an averaged curvature over the width of the shear key, which is mainly caused by cracks at the two interfaces.

The moment-curvature relationship mentioned above is typically composed of three phases: before bond cracking, after bond cracking but before rebar yielding, and after rebar yielding (see Figure 4.7). In the first phase, the moment-curvature curve is close to linear. In the second phase, non-linearity happens due to bond cracking, and the slope of the curve becomes flatter than that in the first phase. During the last phase, the curve was approaching a flat line due to rebar yielding. Note that the curve is not flat at the yielding point like that in an idealized stress-strain curve of a bare bar when yielding happens. Rather it is similar to the average stress-strain curve of an embedded bar (see Figure 4.9), the strain of which is monitored over a distance that covers at least several cracks. The increasing mean stress after mean yielding strain is due to the tension stiffening effect of concrete. Since the LVDT monitored the whole width of the shear key, within which several cracks were covered, the shape of the
moment-curvature curve should be similar to the stress-strain curve of an embedded bar.

The yield curvature was determined by identifying the yield moment, which is the moment that causes the first rebar to yield. In the case studied, since not all of the rebar at the left bond were monitored, the minimum moment that yielded the rebar monitored is taken as the yield moment. The ductility index calculated using this value therefore tends to be conservative. The ultimate curvature is determined as largest curvature in the moment-curvature curve. Since most of the specimens did not achieve its ultimate capacity as discussed before, the ductility index calculated is representative of demand ductility and not capacity ductility. Remember that demand ductility is always less than capacity ductility and can serve as a conservative estimate of the capacity ductility. Similar to $\psi_y$ and $\psi_u$, $D_y$ and $D_u$ were determined correspondingly for specimens that failed at a cross section close to the load strip. Since the yield moments at the corresponding failed cross sections are not available from any sensors, the yield moments based on rebar strain gauge readings were used.

![Figure 4.9: Stiffening effect of concrete on embedded bar (Adapted from Hsu (1993))](image)
The ductility indices calculated are listed in Table 4.2 for all the specimens. For the specimens subjected to high moment demands, the ductility indices generally vary from 5.5 to 8 except for MONO-02, which has the minimum ductility index of 2.78. This low value results from the high yield displacement. The ductility indices of the high shear specimens vary between 8.8 and 11.7, which are higher than those of the grout combination in the high moment test which also failed at the bond. This higher ductility index comes from a smaller ultimate curvature and even smaller yield curvature due to the high shear effect. For each material combination, it is also observed that the ductility index dropped from the static tests to the corresponding monotonic test. At first glance, this seems to indicate that fatigue test plays a part in it. Rearranging these specimens, as shown in Table 4.3, shows that this drop is caused by the increase of $\psi_y$ or $D_y$. For the specimens that failed at the bond, since the yielding curvature can be calculated as the ratio of yielding strain of the bottom rebar to the distance between the bottom rebar and the neutral axis, a larger yielding curvature means this distance is shorter and the compressive zone is larger when the first rebar yields. The larger the compressive zone, the lower compressive strength of the bond. This is confirmed by the capacities from the strength tests of these specimens that failed at the bond (grout with PVA-HM and UHPC with PVA-HS).

A suggested improvement to the calculation of ductility is to calculate the rotational capacity of a member at a plastic hinge (Skogman et al., 1988). This rotational capacity is calculated as:

$$\theta_p = \psi_u d = \epsilon_{cu} \frac{d}{c}$$

(4.7)

in which $d$ is the effective depth at each moment concentration, $c$ the distance from the extreme compression fiber to the neutral axis, and $\epsilon_{cu}$ the limit strain at the
Table 4.2: Specimen ductility index

<table>
<thead>
<tr>
<th>Shear Key Mixture and Loading Condition</th>
<th>Specimen ID</th>
<th>$\psi_y$ (or $D_y$)</th>
<th>$\psi_u$ (or $D_u$)</th>
<th>$\mu$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Grout with PVA (HM)</td>
<td>STA-01</td>
<td>$0.6498 \times 10^{-3}$</td>
<td>$0.0051$</td>
<td>7.81</td>
</tr>
<tr>
<td></td>
<td>MONO-01</td>
<td>$N/A$</td>
<td>$N/A$</td>
<td>$N/A$</td>
</tr>
<tr>
<td></td>
<td>MONO-01(redo)</td>
<td>$0.7301 \times 10^{-3}$</td>
<td>$0.0041$</td>
<td>5.57</td>
</tr>
<tr>
<td>UHPC with steel (HM)</td>
<td>STA-02</td>
<td>$0.2436$</td>
<td>$1.3475$</td>
<td>5.53</td>
</tr>
<tr>
<td></td>
<td>MONO-02</td>
<td>$0.6519$</td>
<td>$1.8141$</td>
<td>2.78</td>
</tr>
<tr>
<td>UHPC with PVA (HM)</td>
<td>STA-03</td>
<td>$0.4036$</td>
<td>$2.3437$</td>
<td>5.81</td>
</tr>
<tr>
<td></td>
<td>STA-04</td>
<td>$0.3549$</td>
<td>$2.8291$</td>
<td>7.97</td>
</tr>
<tr>
<td></td>
<td>MONO-03</td>
<td>$0.4346$</td>
<td>$2.4899$</td>
<td>5.73</td>
</tr>
<tr>
<td>UHPC with PVA (HS)</td>
<td>STA-05</td>
<td>$0.2256 \times 10^{-3}$</td>
<td>$0.0025$</td>
<td>11.06</td>
</tr>
<tr>
<td></td>
<td>STA-06</td>
<td>$0.2553 \times 10^{-3}$</td>
<td>$0.0030$</td>
<td>11.67</td>
</tr>
<tr>
<td></td>
<td>MONO-04</td>
<td>$0.2763 \times 10^{-3}$</td>
<td>$0.0024$</td>
<td>8.83</td>
</tr>
</tbody>
</table>

Table 4.3: Relationship between ductility index and maximum moment

<table>
<thead>
<tr>
<th>Shear Key Mixture and Loading Condition</th>
<th>Specimen ID</th>
<th>$\psi_y$ (or $D_y$)</th>
<th>$\mu$</th>
<th>Max. Moment (kip-in)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Grout with PVA (HM)</td>
<td>STA-01</td>
<td>$0.6498 \times 10^{-3}$</td>
<td>7.81</td>
<td>763.5</td>
</tr>
<tr>
<td></td>
<td>MONO-01(redo)</td>
<td>$0.7301 \times 10^{-3}$</td>
<td>5.57</td>
<td>674.8</td>
</tr>
<tr>
<td>UHPC with steel (HM)</td>
<td>STA-02</td>
<td>$0.2436$</td>
<td>5.53</td>
<td>742.5</td>
</tr>
<tr>
<td></td>
<td>MONO-02</td>
<td>$0.6519$</td>
<td>2.78</td>
<td>745.4</td>
</tr>
<tr>
<td>UHPC with PVA (HM)</td>
<td>STA-03</td>
<td>$0.3549$</td>
<td>7.97</td>
<td>738.0</td>
</tr>
<tr>
<td></td>
<td>STA-04</td>
<td>$0.4036$</td>
<td>5.81</td>
<td>736.7</td>
</tr>
<tr>
<td></td>
<td>MONO-03</td>
<td>$0.4346$</td>
<td>5.73</td>
<td>727.0</td>
</tr>
<tr>
<td>UHPC with PVA (HS)</td>
<td>STA-05</td>
<td>$0.2256 \times 10^{-3}$</td>
<td>11.06</td>
<td>650.0</td>
</tr>
<tr>
<td></td>
<td>STA-06</td>
<td>$0.2553 \times 10^{-3}$</td>
<td>11.67</td>
<td>634.1</td>
</tr>
<tr>
<td></td>
<td>MONO-04</td>
<td>$0.2763 \times 10^{-3}$</td>
<td>8.83</td>
<td>543.8</td>
</tr>
</tbody>
</table>
extreme compression fiber. It should be noted that

$$\psi_u = \frac{\epsilon_{cu}}{c} \tag{4.8}$$

where $d$ is recommended to be the total spreading length of the plastic hinge at each moment concentration (Sawyer, 1964). The minimum ductility calculated in this way is assured in older versions of AASHTO LFRD (prior to 2005) by limiting the maximum value of $\frac{d}{d_e}$ to be less than 0.42, in which $d_e$ is the effective depth from the extreme compression fiber to the centroid of the tensile force in the tensile reinforcement. In the versions after that, however, this limit is eliminated and replaced by reducing the factored resistance of prestressed and nonprestressed sections if the tensile steel quantity increases under the condition that the net strain in the extreme tensile steel is less than 0.005 (AASHTO LRFD 5.5.4.2.1). Only when there is also a corresponding increase in compression steel, can the factored resistance of the section be increased. This new specification accounts for the decreasing ductility due to the increasing over-strength. For the slab tested, at ultimate flexural capacity, the tensile strain in the extreme steel fiber is 0.023, much larger than 0.005, if the design 28-day compressive strength of concrete 6 ksi is used. Therefore ductility in this respect is satisfied and there should be no reduction of factored flexural resistance of the section.

### 4.2 Result Analysis—Performance at Service Level

At the service level, specimen performance is evaluated from the following aspects: reserve capacity and fatigue life, stiffness degradation, and bond performance evaluation.
4.2.1 Reserve capacity and fatigue life

4.2.1.1 Significance

Reserve capacity is an indicator of the remaining fatigue life of a certain component of a structure. In a reinforced concrete structure, rebar is the main tension resisting element, and the reserve capacity of rebar determines the remaining life of a certain component. The strain of rebar selected should be at a critical cross section where the rebar strain is the maximum. For a reinforced concrete member without cracks, the strain of rebar varies from section to section depending upon the loading configuration. After cracking, the strain at the crack is much larger than that protected by concrete due to tension stiffening effect of concrete (Hsu, 1993). This effect is especially obvious at low strains, which is what happened during the fatigue test. Therefore when determining the reserve capacity or fatigue life of rebar, sensors should be installed in sections that have critical strains. In the case studied, strain gauges were installed in the shear key close to the bond which is sensitive to cracks from past experience. Therefore the remaining life of the shear key can be known.

4.2.1.2 Reserve capacity

Following 10 million cycles’ fatigue test and 2 million cycles’ pond test, the specimen was loaded monotonically until large deformation occurred with slight increase of load. From the monotonic test, rebar strains were plotted against the applied monotonic load at a level that is a little bit larger than the corresponding fatigue load in order to show the maximum rebar strain at the maximum fatigue load level, marked by a horizontal line (see Figure 4.10). It is evident in the plots that FAT-03 gave minimum maximum strain at the final fatigue load level compared with other specimens. Its maximum rebar strain is $85 \times 10^{-6}$, 4.1 percent of the yield strain.
(2070 × 10⁻⁶) of the reinforcing steel. This low strain of FAT-03 is due to the fact that the bond was still sound enough to resist tension at the end of the fatigue test. The grout combination FAT-01 (redo) gave the largest maximum strain (550 × 10⁻⁶), which is about 26.6 percent of the yield strain of rebar. Taken the maximum tensile strain at the extreme tensile fiber of the bond as the ratio of 7.5 (factor of tensile strength) to 57000 (factor of Young’s modulus), the maximum tensile strain of the bond is 131 × 10⁻⁶. The rebar strain of FAT-01 (redo) at the end of the fatigue test is much larger than the maximum tensile strain of the bond, indicating that the bond was cracked at the end of the fatigue test. FAT-02 (UHPC with steel -HM) and FAT-04 (UHPC with PVA -HS) had similar maximum rebar tensile strains at the end of the fatigue test —279 × 10⁻⁶ and 272 × 10⁻⁶ respectively, larger than the maximum tensile strain of the bond, indicating bond cracking at the end of the fatigue tests.

### 4.2.1.3 Fatigue life

Bridge slabs in AASHTO are not designed for fatigue limit states. According to ACI Committee 215 (ACI, 1974), beams subjected to repetitive loads under the service state should be checked for the possibility of fatigue distress so that adequate performance under service can be assured. Fatigue tests of reinforcing steel below 10 million cycles seem to indicate that reinforcing steel has a stress endurance limit below which its fatigue life is infinite. Of all the factors that can influence the fatigue life of steel, the stress range, which is the difference between the maximum stress and the minimum stress, is deemed to be the most critical one. In AASHTO [A5.5.3.2], the fatigue threshold for straight reinforcement without a cross weld in the high-stress region is taken as:

\[
(\Delta F)_{TH} = 24 - 0.33f_{min}
\]  

(4.9)
Figure 4.10: Rebar strain at the maximum fatigue load level

where $f_{\min}$ is the minimum stress. The units used for stress are all ksi. In the slabs tested, the minimum load level is 0.1 kips, which is small enough to consider $f_{\min}$ to be equal to zero. Then the final limit range would be 24 ksi. The maximum stress ranges during the fatigue tests are 8.1 ksi, 2.4 ksi, 7.9 ksi, and 15.9 ksi for FAT-02, FAT-03, FAT-04, and FAT-01(redo), respectively. It can be seen that under 10 million cycles’ fatigue test, FAT-02, FAT-03, and FAT-04 are still far from getting distressed from fatigue.

It should be noted however, the strain of the rebar at the critical bond may not be the controlling strain as far as the fatigue life of the rebar is concerned. Other
possible critical locations include the bends within the shear key, and the critical section in the concrete slab considering the shear key and the bond may be much stronger than the concrete. For the slab tested, the rebar strain at the cross section within the precast under the load strip, where the plastic hinge may form, could control the fatigue life. For the bridge, this controlling strain could be within the precast concrete that is adjacent to the joint due to the large demand attracted to it by the shear key. However as far as bond performance is concerned, the stress range of rebar close to the bond is good enough as an indicator.

4.2.2 Stiffness degradation

4.2.2.1 Significance

In an indeterminate structure like the modified NEXT-D bridge studied, the stiffness ratios among various components determine the demands to be exerted on each component. During the service life of a bridge, all the components will deteriorate gradually, which can cause subsequent stiffness degradation. Since the stiffness degradation differs from component to component, a relative stiffness change will occur, resulting in demand redistribution. As far as the superstructure is concerned (the beams and the shear keys), the precast stem and the cast-in-place shear key can be much stiffer than the precast slab, and therefore can attract much larger demands. During service, the prestressed stem is assumed to not crack under tension, therefore its stiffness will remain at a constant level, while the stiffness of the shear key may drop due to material cracking. Therefore the relative stiffness of the shear key decreases, resulting in less demands on it and more demands on the stem.
4.2.2.2 Stiffness calculation

From the fatigue test, the shear key rotational stiffness was calculated by dividing the moment at the center cross section of the shear key by the average shear key rotation from LVDT readings, which is consistent with the way that stiffness was calculated in the static test. The vertical stiffness of shear key was not calculated due to the fact that the displacement measured by vertical LVDT was so small during the fatigue test that the resolution of the sensor accounts for a large percent of it, and therefore the data from the sensor was not reliable. Instead, the vertical stiffness under the load strip was calculated by dividing the applied load by the global displacement readings from the left string pot. Although the left string pot was not directly under the load strip, the distance between them was so small that the displacement it measured is considered to be similar enough to the actual displacement under the load strip. In all of the stiffness calculations for the fatigue test, it is the range of displacement and range of load that were used rather than the absolute value. When dealing with the data, the range of each cycle was calculated, added together, and then averaged over the number of cycles. In this way the effect of extreme data can be reduced. Except for FAT-01 which had no LVDTs on top of the slab, the rotational stiffness degradation for each specimen was plotted against each other (see Figure 4.11).

4.2.2.3 General trend

Shown in the stiffness degradation plots (see Figure 4.11a), of all the specimens, FAT-02 has the highest initial rotational stiffness of about $8.99 \times 10^5$ kip-in/rad. During the first load level of 8.7 kips from 0 to about 6 million cycles, its stiffness kept dropping. After that, a higher load level of 9.7 kips was exerted, accompanied by
a big drop of the rotational stiffness, which stabilized at about $3.18 \times 10^5$ kip-in/rad from 6.7 million cycles to 10 million cycles. Compared with that of FAT-02, the stiffness of FAT-04 was quite stable ($4.72 \times 10^5$ kip-in/rad) during the first load level of 8.1 kips from 0 to 5 million cycles. At the beginning of the second load level of 10.6 kips, its stiffness dropped and stabilized at about $2.02 \times 10^5$ kip-in/rad from 6 million cycles to the end. Compared with the two specimens mentioned above, FAT-03 did not have much change in its rotational stiffness. The biggest drop (from $6.06 \times 10^5$ kip-in/rad to $4.73 \times 10^5$ kip-in/rad) happened at the beginning of the second load level at the end of 2 million cycles, when the load was increased from 4.9 to 6.5 kips. During the third and fourth load levels (from 2.4 to 6.9 million cycles), the stiffness got stabilized at a value of about $4.41 \times 10^5$ kip-in/rad. At the beginning of the final load level of 7.3 kips, the stiffness dropped and stabilized at the value of about $4.38 \times 10^5$ kip-in/rad until about 9.5 million cycles, where the stiffness dropped again to the value of $4.24 \times 10^5$ kip-in/rad due to a longitudinal crack in the shear key. With the smallest initial stiffness of $1.06 \times 10^5$ kip-in/rad, FAT-01(redo) had a very little change in its rotational stiffness throughout the test except at the beginning of the sixth million cycles when the load was increased from 5.3 to 7 kips. Its final stabilized stiffness is $0.77 \times 10^5$ kip-in/rad.

The initial rotational stiffness for the shear keys made with UHPC material combinations (FAT-02 to FAT-04) varies from $4.7 \times 10^5$ to $9.0 \times 10^5$ kip-in/rad. The combination of UHPC with steel fiber gave the highest initial rotational shear key stiffness as that showed in the static tests. For FAT-03 and FAT-04, although they had the same shear key material combination, the initial stiffness of FAT-04 ($4.72 \times 10^5$ kip-in/rad) is 71.9 percent of that of FAT-03 ($6.06 \times 10^5$ kip-in/rad). Since the precast strengths of the two specimens during the test day were close, their effects on the stiffness through influencing the soundness of the bond can be neglected.
The interface of concrete slab can also be neglected. The remaining major factors include the strength and shrinkage of the shear key material, and the age at which the specimen was tested. The shear key cylinder tests during the corresponding test day showed that the average compressive strength for FAT-04 was 18370 psi, 87.2 percent of that for FAT-03 (21070 psi). Remember the same situation happened for STA-03 and STA-04, where STA-03 had a shear key compressive strength about 82 percent of that of STA-04, its bond cracking strength was only 74 percent of that of STA-04. The influence of the other two factors can be combined together. Since FAT-03 and FAT-04 were tested 22 and 77 days, respectively, after the shear key was cast, the shear key material property of FAT-04 can be influenced more by creep and shrinkage than that of FAT-03. For instance, due to the increase in strain caused by creep, Young’s modulus of shear key is reduced under long-term repetitive loads (Barker and Puckett, 2007), which will cause the reduction of stiffness of the shear key.

As far as vertical stiffness degradation (see Figure 4.11b) is concerned, the high shear test, due to its different support condition experiences a displacement...
under the load strip which is smaller than those in the high moment tests. This explains its higher stiffness compared with the specimens in the high moment tests. For the three specimens subjected to a high moment-to-shear ratio (FAT-02, FAT-03, and FAT-01(redo)), the general trends of vertical stiffness degradation are similar to those of rotational stiffness degradation. The stiffness of FAT-02 is higher than that of FAT-03, but this difference is not as pronounced as that in shear key rotational stiffness. This stiffness difference is influenced by the rotational stiffness of the shear key and effective moment of inertia of concrete of these two specimens.

Finally, due to the resolution of sensors, the stiffness values used above and in the plots do not represent the exact value. However, compared with the displacements measured, the resolutions of sensors account for only a small percent. Generally speaking, the stiffness values presented are within a tolerable range and the general trends of stiffness degradation are reliable.

4.2.2.4 Stiffness evaluation

Due to the load factor and impact factor applied when calculating the fatigue demand, the exerted demand is larger than the live load demand from the simulation. As such it is expected that the stabilized stiffness during the fatigue test is less than the stiffness used to obtain the demands. For FAT-02 and FAT-03, the stabilized rotational stiffness during the fatigue tests were $3.1 \times 10^5$ kip-in/rad and $4.4 \times 10^5$ kip-in/rad respectively (see Figure 4.11). This is equivalent to about 53.3 and 440 percent of the values used to predict the demand. The higher stiffness for FAT-03 is due to the fact that there was no bond cracking during the fatigue test. For FAT-01(redo), the stabilized rotational stiffness of $0.77 \times 10^5$ kip-in/rad is about 92 percent of that applied in demand determination.
4.2.3 Bond performance evaluation

4.2.3.1 Leakage-based

The leakage of water along the shear key interface during a pond test is a direct evidence of interface cracking both on the top and at the bottom of the interface. During the pond test, there was leakage along the shear key interfaces of FAT-01. However, since a larger than required fatigue load was applied on FAT-01, it is not prudent to say the bond is unsound from this evidence. Later a make-up specimen FAT-01(redo) showed no leakage during the pond test, which indicates that there are no interconnected channels to help convey the water from the top to the bottom. Since the joint is subjected to positive moment only, a possible scenario is that there is no bond crack at the compression side of the joint, but there could be at the tensile side of the joint.

4.2.3.2 Sensor information-based

To further detect the existence of any bond cracks, sensor readings are needed. One feature that can prove interface cracking is the joint rotational stiffness degradation during the fatigue test. From Figure 4.11a, it is seen that there were drops of shear key rotational stiffness for all of the specimens during the fatigue tests. The stiffness degradation are not as pronounced in FAT-01 (redo) and FAT-03 as in FAT-02 and FAT-04. Therefore none of the bonds of these specimens were intact during the fatigue tests, but the cracking degree of the bonds differ among the specimens.

Based on the maximum rebar strain a the fatigue load level at the end of the fatigue test (see Figure 4.10), it is evident that FAT-02, FAT-04, and FAT-01(redo) all had bond cracked at the end of the fatigue tests.
For FAT-02 and FAT-04, the strain measurement at the bottom of the shear key shows there were localized stress relief in these specimens (see Figure 4.12). The bottom shear key interface is under tension, and therefore much more sensitive to cracks compared to the top interface. If there is any crack along the bottom edge of the bond, there will be stress relieved from the adjacent shear key materials. If the interface crack is deep enough, there will be sufficient adjacent shear key material to relieve stress that the BDIs located at the bottom of the shear key can detect this relief. Such evidence is sufficient to prove the interface cracking unless the cracks occur in the shear key adjacent to the strain transducer but not in between the measurement points of the transducer. Cracks that happened between the measurement points of the strain transducer could increase the strain, which may overcome the drop of the strain due to interface cracking. For FAT-02 and FAT-04, there were no cracks observed in the shear key besides the measuring points. This confirmed the bond cracking of these two specimens.

Based on these features, it is concluded that there were interface cracks for FAT-02, FAT-04 and FAT-01(redo) at the end of the fatigue test. For FAT-03, based on the degradation of shear key rotational stiffness and no indication of stress relief at the bottom of the shear key, it is inferred that there were slight interface cracking during the fatigue test. Besides, the bond was sound enough to resist tension as indicated in the monotonic moment-rotation curve (see Figure 4.7). For FAT-01(redo), although there were not much drop of shear key rotational stiffness nor stress relief at the bottom of the shear key, the rebar strain at the end of the fatigue test revealed the interface cracks.
4.3 Shear Key Material Determination

As mentioned in Section 3.4.1, when selecting shear key materials, it is important that the shear key material possesses high early strength with good workability to satisfy the needs of accelerated bridge construction, high durability both at the bond and within the key itself to solve the joint durability issue, and reasonable cost. The durability requires that the material should possess high toughness (work of fracture) to control crack propagation both at the bond and within the material. Also the material should have good dimensional stability. Considering that in a 40 ft. span bridge, considerable shrinkage in the longitudinal direction could happen - thus giving rise to cracks and reducing the long-term durability. Based on the previous discussion from specimen casting to testing, an evaluation of the performance of each shear key material combination is summarized as follows:
4.3.1 Early strength and workability of shear key materials

Shown in the cylinder tests, the UHPC combinations all had 4-day strength above 13000 psi. Due to the addition of HRWR, the UHPC groups had very good workability that it can compact itself when casting specimens. For the grout with PVA combination, the 7-day compressive strength is 5310 psi, and the 28-day compressive strength is 7405 psi. For this combination, there are obvious trade-offs between workability, toughness, and compressive strength. The addition of fiber although can increase the toughness, it requires a higher amount of water to improve workability, which, however, decreases compressive strength.

4.3.2 Bond performance

Cylinder bond tests showed that in both direct tensile and compression-shear bond tests, the typical failure mode for the grout with PVA combination was at the bond interface, and for the UHPC combinations was in the precast concrete. Static test showed that at similar joint ages, the grout with PVA combination had the lowest bond cracking strength, and the specimen cracked first at the joint interface during the test; while UHPC with steel combination gave the highest bond cracking strength. For the UHPC with PVA combinations, there was a notable increase in bond cracking strength from joint age 6 day to 14 day, which is a critical period for the development of bond strength. After this period, the bond cracking strength increased slowly. Pond test showed no leakage for all the joint material combinations because there were not interconnected channels, but this does not exclude the scenario that there are cracks on the tensile side of the joint. Combined sensor readings, including the shear key rotational stiffness degradation during the fatigue test, maximum rebar strain at the fatigue level at the end of the fatigue test, and strain relief at the
bottom of the shear key, indicated joint interface cracking for FAT-02 (UHPC with steel -HM), FAT-04 (UHPC with PVA -HS), and FAT-01(redo) (Grout with PVA -HM). For FAT-03 (UHPC with PVA -HM), the interface was sound enough at the end of the fatigue test to resist tension.

4.3.3 Stiffness, durability, and toughness

The UHPC with steel combination, due to its high stiffness, attracted a corresponding large fatigue demand on itself and the precast adjacent to it that cause a lot of cracks in the precast concrete. The grout group, by comparison, attracted a smaller demand which resulted in only a few cracks in the adjacent precast. The UHPC with PVA group, with a stiffness in-between those of the above two groups, attracted a moderate moment that caused a few cracks in the precast. For specimens that failed at the joint interface (grout with PVA -HM and UHPC with PVA -HS), fatigue test reduced joint toughness. Test results also showed that a durable bond of the UHPC combinations is accompanied by a high stiffness of the joint material, which can increase the global stiffness and reduce the overall toughness of the specimen, as shown in the case of the UHPC with steel combination. Concerning stiffness, durability, and ductility, the UHPC with PVA combination is a better choice compared with other material combinations.

4.3.4 Specimen capacity

All the material combinations showed higher strengths than the capacities predicted in the shear-moment interaction diagram. For the specimens that failed at the interface (grout with PVA -HM and UHPC with PVA -HS), fatigue test reduced
the capacities of these specimens. For specimens that failed in the precast concrete (UHPC combinations under high moment test), there was no such influence.

4.3.5 Dimensional stability

Shrinkage cracks were found both on the top and at the bottom of the shear key for the UHPC with PVA and grout with PVA groups. The shrinkage crack is considered to result from the curing method applied and less internal restraint from the PVA fiber. Larfarge indicates that no indication of such cracks has been seen for the in-field use of UHPC with PVA (JS2000).

4.3.6 Rebar stress

After 10 million cycles, the rebar in the UHPC groups are far from getting distressed, even in the case of FAT-02 and FAT-04, which had a slight interface cracking under conservative fatigue loads. For the grout with PVA combination (FAT-01(redo)), the stress range of rebar at the fatigue load level is the largest compared with other material combinations.

4.3.7 Cost

Among the three combinations, the grout with PVA is the most economical, and the UHPC combination is most expensive with a cost than can be four times as high as the grout. These relative costs are purely for the base material. However, if the full-advantage of UHPC is realized then the shear key details may be optimized - thus reducing overall costs.
4.4 Closure

The experimental phase of this research provided information needed in determining the expected demand in a shear key of a constructed bridge. The data from the static testing was used to determine the magnitude of the load applied during the fatigue testing and it was also used to calibrate a finite element model of the shear key for more accurate demand distribution in the bridge. Based on the calibrated shear key stiffness matrix that will be talked about in the next Chapter, the fatigue loads applied for each material combination were evaluated.

From what has been discussed above, the UHPC with PVA fiber reinforcement is recommended as the most appropriate shear key material for the NEXT-D beam bridge. Basically it satisfies all the requirements for the application studied. The cost is a deterrent but considering the relatively small amount of material, it is feasible within the context of an entire bridge project.

Several important conclusions from the experimental results include:

- For specimens that failed at the joint interface (grout with PVA -HM and UHPC with PVA -HS), fatigue tests reduced both the capacity and toughness of the bond. For these specimens, both layers of rebar close to the left joint interface yielded during the strength tests. This implies that the anchorage length of the U-bar is sufficient to achieve a full-capacity joint. For specimens that failed within the precast concrete, there were no distinct reduction of ductility and capacity due to fatigue tests.

- The durable bond of the UHPC with steel combination is accompanied by a high stiffness of the joint material, which increases also the stiffness of the surrounding precast concrete deck, and reduces the overall toughness of the specimen. Concerning both durability and toughness, the UHPC with PVA
combination is favored compared with other material combinations. For this material combination, the joint age from 6 day to 14 day is a critical period for bond strength development. A selective open to traffic should be considered during this period in the real bridge.

In the subsequent chapters, it is necessary to determine the live load distribution factors and transverse demand distribution of the modified NEXT-D beam bridge using finite element analysis, which requires the input of stiffness matrix from the shear key finite element model. Before that, the model needs to be calibrated first using the experimental data, which will be presented in the next chapter.
Chapter 5

Demand Quantification for
NEXT-D Beam

5.1 Scope and Objectives

As far as the NEXT-D bridge design is concerned, the load distribution formulas of LRFD for beam design, as they are given in the AASHTO Bridge Design Specifications (AASHTO, 2012), do not directly apply as the beam has a stem spacing of 3 ft. Furthermore, the AASHTO strip method for calculating demands placed on the deck does not consider the varying deck spans which are typical to the NEXT-D bridge. Therefore, the refined analysis method as described in LRFD Article 4.6.3 is required in order to determine the appropriate load distribution factors and deck demands for the NEXT-D bridges. In a previous study (Funcik, 2011), a three dimensional finite element (FE) modeling of the NEXT-D bridges was carried out using the commercial analysis package SAP2000 (CSI, 2011). Base models were built for NEXT-D bridges with a span length of 40 ft, and a beam spacing of either 6 ft (NEXT-6) or 8 ft (NEXT-8).
In the beginning of this Chapter, the study of Funcik (2011) was summarized including some important modeling details of the base model and the basic findings. This base model study utilized the joint stiffness matrix from the FE model study of Flores-Duron (2011).

Since the FE model is sensitive to the modeling assumptions, an initial study was performed to assess the requisite level of accuracy within the base model by Funcik (2011). Two types of base models were created, one using frame and shell elements while the other used solid elements. This screening study provided modeling guidance for the other analytical bridge models used in this research. After this initial model validation, deeper study of demand distribution on the base model was carried out. Since the AASHTO strip width formulas do not apply to the NEXT-D bridges with varying deck spans, appropriate strip widths for different types of live loads were determined for the NEXT-D bridges.

In the bridge FE model, the stiffness of the shear key plays a significant role in the determination of demand distribution. Appropriate modeling the shear key stiffness required experimental data and also a detailed shear key model which is discussed later in this chapter. In the shear key FE model, the shear key stiffness was calibrated based on experimental data. This model aims at determining a stiffness matrix for the shear key, which will be fed into the bridge FE model.

The primary objective of this study is to provide reliable shear key stiffness for the bridge FE model for short-span NEXT-D bridges (between 22 ft and 40 ft) to facilitate the later bridge design discussed in the next chapter.
5.2 Summary of the Base Model Study

5.2.1 Bridge Geometry

As per the request of the SCDOT, this project is to focus on bridge spans of 22 to 40 ft. The use of a six-foot-wide NEXT-D section and an eight-foot-wide NEXT-D section were investigated because these widths are similar to the original NEXT-D beam width proposed by PCINE, and they make multiples of common road widths in South Carolina. In the base model study by Funcik (2011), only the 40-ft span bridges for NEXT-6 and NEXT-8 were constructed initially. The bridge model is supported six inches in from each end which is considered to be the center of bearing. For both NEXT-6 and NEXT-8, the width of the bridge is 47 ft and 4 in. The NEXT-6 bridge model consists of eight NEXT-D beams and seven shear keys, and the NEXT-8 bridge model consists of six NEXT-D beams and five shear keys. Refer to Figures 5.1 and 5.2 for geometry details of base bridge models.

5.2.2 Modeling Approach

Finite element modeling is sensitive to the model inputs, so it is important to establish certain criteria in order to obtain reasonable results. For this project, two types of finite element models were built using SAP2000 (CSI, 2011) for the same bridge. One model used eight-node solid elements for the NEXT-D sections and parapets and the other model used four-node shell elements for the bridge deck and frame elements for the stems and parapets. In the shell model, the slab was connected to the stems and the parapets using rigid links. In both models, the shear key was represented by a frame element, the input of which is to be determined based
on the calibrated shear key finite element model which will be discussed in Section 5.3.
5.2.2.1 Shear Key Modeling

The appropriate modeling of the shear key with a correct stiffness plays a significant role in demand determination of the NEXT-D bridges. In the base model, the shear key frame element is spaced 6 in. apart to provide adequate connectivity between the beams while trying to control the overall size and runtime of the model. This element spacing also allows for the investigation of shear key demand distribution in the longitudinal direction. The stiffness of the shear key frame element was obtained based on an original study by Flores Duron (Flores-Duron, 2011) before any experiment was carried out. The stiffness matrix is displayed in Figure 5.3, in which 1-3 denote translational degrees of freedom and 4-6 denote rotational degrees of freedom. Refer to Figure 5.4 for shear key local axes.

\[
\begin{pmatrix}
1 & 2 & 3 & 4 & 5 & 6 \\
1 & 1201 & 0 & 0 & 0 & 0 \\
2 & 220 & 0 & 0 & 0 & 513 \\
3 & 817 & 0 & 1905 & 0 \\
4 & 381 & 0 & 0 \\
5 & Sym. & 21929 & 0 \\
6 & & & & & 5905
\end{pmatrix}
\]

Figure 5.3: Frame element stiffness matrix for a six-inch section of shear key in units of kip, in, and rad (Flores-Duron, 2011)

In order to achieve the requisite stiffness matrix in Figure 5.3, the stiffness matrix formulation considering shear deformations for a 2D beam was utilized (see Figure 5.5) to identify the parameter input of the shear key frame element. In the formulation matrix shown in Figure 5.5, \(E\) stands for modulus of elasticity, \(I\) stands for moment of inertia, \(f_s\) is the shape factor, \(G\) is the shear modulus, \(A\) is the cross sectional area, and \(L\) is the length of the element. Axial stiffness is equal to \(\frac{AE}{L}\) and torsional stiffness is equal to \(\frac{JG}{L}\) where \(J\) is the torsional constant. In the matrix,
Figure 5.4: Shear key in local and global coordinate systems

\[ k_{w/shear} = \frac{EI}{L^3(1 + \beta_s)} \left( \begin{array}{cccc} 12 & 6L & -12 & 6L \\ 6L & L^2(4 + \beta_s) & -6L & L^2(2 - \beta_s) \\ -12 & -6L & 12 & -6L \\ 6L & L^2(2 - \beta_s) & -6L & L^2(4 + \beta_s) \end{array} \right) \]

\[ \beta_s = \frac{12EI_s}{GAL^2} \]

Figure 5.5: Element stiffness matrix for beam elements with inclusion of shear deformations

the shear modulus \( G \) is obtained based on the predefined Young’s modulus \( E \) and Poisson’s ratio, leaving \( I, f_s, A, \) and \( L \) the only unknown parameters, which are calculated using an excel chart (see Appendix A). This 2D formulation matrix can be applied to the 3D problem by considering cross section properties in the orthogonal directions.

Based on the formulation, the effective length of the shear key frame element was determined to be 4.66 in. \( \left( \frac{2 \times 513 \text{kip/ rad}}{220 \text{kip/in}} = 4.66 \text{ in.} \right) \), which does not necessarily fit well in the 8 in. gap left in the model for the shear key. This problem was solved.
through the use of equal constraints, i.e., one end of the shear key frame element was attached to the shell (or solid) element in one beam, and the other end was constrained to the shell/solid element in the adjacent beam using equal constraints for all the degree of freedoms.

The section properties of the shear key frame element determined this way include cross sectional area \( (A) \), torsional constant \( (J) \), moment of inertia about both axes \( (I_2, I_3) \), and shear area in both directions. The reliability of this method was checked by creating a very simple model of a 4.66 in. long frame element that was fixed at one end and free at the other (see Figure 5.6). The free end was constrained with a fixed node that was 3.44 in. away from it using equal constraints, therefore creating an 8 in. spacing between the fixed nodes. Unit displacements and rotations were applied to both the fixed end of the frame and the fixed node, and the reactions show that all the desired stiffness terms are obtained.

![Figure 5.6: Shear key stiffness verification model](image)
5.2.2.2 Parapets

The parapet was modeled with the dimensions given in Figure 5.7. In the shell model, the parapet was modeled as a frame element using the section designer feature of SAP2000. The parapet is connected to the deck using rigid links. The links allow the centroid of the parapet to be located properly in space relative to the rest of the bridge. The compressive strength of the parapet was preliminarily assigned to be 6 ksi for this initial study but all final analyses were done using the SCDOT recommended 4 ksi.

![Figure 5.7: Parapet dimensions (SCDOT 2008)](image)

5.2.2.3 Restraints

In order to ensure a symmetric response and avoid Poisson effect induced stresses at the supports for the bridge, special attention was paid to the restraints
placed on the bridge. In the model, at the location of the support, which is 6 in. from the end, all of the nodes at the bottom of the stems are restrained in the z (vertical) direction. At one end of the bridge, the node on the far side of the bridge is restrained in all the three translational directions. The node on the diagonally opposite corner of the bridge is restrained in the x (transverse) direction in order to keep the bridge from rotating about the z-axis. Refer to Figures 5.8 and 5.9 for the restraint details of the NEXT-8 solid model and shell models separately.

Figure 5.8: Restraints for solid model

5.2.2.4 Bridge live loads

To appropriately assess the behavior of the NEXT-D bridge it is important to apply loads which are typical to the bridge. As such, the live loads prescribed in the bridge design specifications were used. The design loads for decks are as specified in LRFD Article 3.6.1.3.3: when the deck is spanned mainly transversely, either the design truck of Article 3.6.1.2.2 (known as the HS20 truck) or the design tandem
3.6.1.2.3 needs to be applied (AASHTO, 2012). The amplification of the wheel loads from centrifugal and braking forces can be ignored. The design truck (Figure 5.10) consists of an 8-kip front axle, a 32-kip rear axle on the tractor, and a 32-kip axle on the trailer. The spacing between axles on the tractor is 14 ft and the spacing between the rear axle of the tractor and the trailer axle is a minimum of 14 ft but not more than 30 ft. The axle loads are split evenly between the driver’s side and passenger’s side of the truck and the tires on an axle are spaced 6-ft apart. The spacing used should maximize the demand of the design. The design tandem (Figure 5.11) consists of two 25-kip axles with 6 ft between each tire on an axle and 4 ft between axles.

According to LRFD Article 3.6.1.2.5, the contact area of wheel loads may be modeled as rectangular with a width of 20 in. and a length of 10 in. The force of the tire is to be uniformly distributed over the contact area. In the case studied, the wheel loads were applied as patch loads with widths between 14 and 15 in. and a length of 12 in. The dimensions of the wheel load were driven by the dimensions of the shell and solid elements of the deck. The load modeled is actually conservative in the sense that the patch load area does not include the spreading length of the deck depth.
5.2.3 Basic findings

The bridge FE model was validated by using both solid modeling and shell modeling, the influence lines of transverse demands from which were close to each
other except those for the exterior beams due to the different connection between parapets and the deck. Since the modeling of this connection in the shell model is much closer to the reality, the shell model is selected.

AASHTO’s strip method for deck design does not consider the varying deck spans which is common for the NEXT-D bridges. The strip width for NEXT-D bridges is determined based on the geometry of the AASHTO live loads. For the three live load cases studied (design tandem, single 32-kip axle of the design truck, and two 32-kip axles of the design truck spaced 14 ft apart), the recommended strip width is 10 ft for design tandem, 14 ft for the single-axle case, and 28 ft for the two-axle case. Using the strip width recommended above, even if more than one truck is in a lane at a time, the bridge will be ensured to have enough capacity to function without failure.

The normalized shear key transverse live load demands show that the critical demands are from the design tandem for both NEXT-6 and NEXT-8. It is also observed that the normalized transverse demands on a one-foot strip width for NEXT-8 are more critical than those for NEXT-6. This difference is particularly distinct for the positive moment demand due to the longer deck span of the NEXT-8.

5.3 Calibration of Shear Key Finite Element Model

In order to get the demand distribution for NEXT-D beam bridge, the shear key stiffness is essential. Here the stiffness matrix (Figure 5.12) is assumed to be symmetric with the following terms K11, K22, K33, K66, and K26 that are expected to be determined from the shear key FE models subjected to axial tension in X (local 1) direction, shear in Y (local 2) direction, shear in Z (local 3) direction, and bending in Z (local 6) direction respectively. The local axis was defined before but
is repeated here for convenience (see Figure 5.13). In the matrix, there is one term that is available from experimental data (K66). The idea is to calibrate this term against experimental data and use the calibrated model to get K26 and other terms from a detailed FE model of the shear key. Some of the stiffness terms should be elastic-cracked stiffness, obtained after cracking occurs but before the steel yields at the fatigue load level. However this may not apply to other stiffness terms, for which the load level may not be large enough to cause interface cracking to happen. In other words, the same load that causes interface crack in bending or axial tension may not cause cracking in shear.

\[
\begin{pmatrix}
K_{11} & 0 & 0 & 0 & 0 & 0 \\
0 & K_{22} & 0 & 0 & 0 & K_{26} \\
0 & 0 & K_{33} & 0 & K_{35} & 0 \\
K_{35} & 0 & 0 & K_{44} & 0 & 0 \\
\text{Sym.} & K_{55} & 0 & 0 & K_{55} & 0 \\
0 & 0 & 0 & 0 & 0 & K_{66}
\end{pmatrix}
\]

Figure 5.12: Shear key stiffness matrix

Figure 5.13: Shear key in local and global systems
5.3.1 Model Introduction

5.3.1.1 Geometry

The original shear key finite element models were built by Flores-Duron (2011) in ANSYS 12 (Release, 2009), which had a #5 headed reinforcement at 6 in. on center. As the configuration of the rebar was switched to #4 U-bar at 8 in. on center and dimensions of the shear key were changed, new models were built. The final shear key configuration adopted for the NEXT-D bridge is illustrated in Figure 5.14. The new shear key model is 4 in. in the longitudinal direction, which is the center to center distance of the staggered U-bar within the shear key. The model is 8 in. tall representing the depth of the deck. A beam flange length of 12 in. was modeled on each side of the shear key. Refer to Figure 5.15 for the finite element model geometry.

![Figure 5.14: Shear key configuration](image)

5.3.1.2 Material models

In the FE model, the solid 65 element with cracking capability is used to model concrete and UHPC with PVA fiber, and the solid 185 element with plasticity, stress stiffening, and large strain capabilities is used to model the U-bar.

The stress-strain behavior proposed by Graybeal (2007) is used to model the UHPC. This behavior was originally proposed to be representative of UHPC rein-
forced with steel fiber. Although PVA fiber provides less strength than does steel fiber, its effectiveness in controlling crack formation and propagation is like the steel fiber. Considering that in the shear key FE model to be calibrated the load to be exerted only needs to be sufficient to obtain the elastic-cracked stiffness, the linear stress-strain behavior is deemed sufficient for UHPC with PVA.

The linear stress-strain behavior proposed by Graybeal (2007) is based on a series of test conducted on UHPC with steel fiber. The stress-strain relationship of the cylinders tested 8 weeks after casting shows a 5 percent of divergence from linear behavior up to about 70 percent of the compressive strength (18.1 ksi). And the cylinders tested 2 weeks after casting show the similar stress-strain behavior up to about 50 percent of the compressive strength (16 ksi).

In the current study, the compressive strength of UHPC with PVA fiber is 18 ksi at the age of 2 weeks and 22.3 ksi at the age of 8 weeks. According to the formula proposed by Graybeal (Eq 5.1), the secant Young’s modulus of UHPC with PVA fiber with respect to strain at the peak stress is about 6200 ksi at the age of 2 weeks and
6900 ksi at the age of 8 weeks. Results show that before a divergence of 5 percent from linear behavior occurs, the Young’s modulus is much higher than that predicted by the formula. The values calculated above are increased by 10 percent, which gives a range from 6820 ksi to 7590 ksi. Taking the average value, the Young’s modulus of the shear key material is set to be 7200 ksi. The Poisson’s ratio is assigned 0.19 as that for UHPC with steel fiber (Graybeal, 2007).

\[ E_c = 46200\sqrt{f'_c} \]  \hspace{1cm} (5.1)

For concrete, the compressive strength at the age of slab test is 9600 psi. It should be noted that this comes from a design mix with a specified strength of 6500 psi. Since calibration of the model is desired, the actual strength and not the design strength is used. Considering the load level to be applied, a linear elastic model is used. The Young’s modulus for concrete is set to be 5600 ksi according to the formula from ACI 8.5.1 (Eq 5.2) (ACI, 2011) which is nominally identical to the AASHTO equation for normal weight concrete as specified in section 5.4.2.4 (AASHTO, 2012). Both cracking and crushing of UHPC and concrete are disabled. The reasons are as follows. First, before reinforcement yields, crushing of concrete and UHPC is not supposed to happen. Second, although cracking is expected in the model, its existence causes convergence issues. Third, by disabling concrete and UHPC cracking, only the bond is permitted to crack. The lumped crack is assumed to have the equivalent effect on the shear key stiffness as that of the distributed cracks. For steel, a bilinear elastic-plastic model from Hsu (1993) is applied, in which the initial Young’s modulus is 29000 ksi, and the Young’s modulus after yielding is 2.9 ksi. Enhanced strain formulation for the solid 185 element is applied to prevent shear locking in all the FE models by setting KEYOPT (2) = 2. A summary of the above
material properties and material models are displayed in Table 5.1 and Figure 5.16.

\[ E_c = 57000 \sqrt{f_c'} \quad (5.2) \]

Figure 5.16: Material model

5.3.1.3 Contact elements

5.3.1.3.1 Key options  Surface-to-surface contact pairs (conta 174 and targe170) are used to model the contact between UHPC with steel, concrete with steel, and concrete with UHPC. The contacts in the study are all asymmetric, which means all contact elements are on one surface and all target elements on the other surface. Since a target surface can penetrate the contact surface, when assigning the target and
contact surfaces, the target surface assigned to the more rigid material. Therefore in
the contact with steel, steel is the target surface, and in the contact between concrete
and UHPC, UHPC is the target surface. Pure penalty method is selected (KEYOPT
(2) = 1) when modeling contact, which uses springs to model the contact behaviors in
both normal and tangential directions. At each iteration, except in the very first one,
the normal contact stiffness (KN) is updated automatically depending on the current
mean stress of the underlying elements and the allowable penetration (FTOLN). The
tangential contact stiffness (KT) is updated automatically at each iteration based on
the current contact pressure, friction coefficient (Mu), and allowable slippage (SLTO).
This automatic update of stiffness based on the mean stress of each element is obtained
by setting KEYOPT (10) = 5. Bonded (always) contact (KEYOPT (12) = 5) is
chosen so that there is no sliding or separation between different materials. An initial
‘perfect’ contact status is created which has no initial penetration, gap, or initial force
in between the contact elements by setting KEYOPT (5) = 3 and KEYOPT (9) =
1 under the condition of bonded (always) contact. Contact is detected within the
pinball region of the Gaussian points. This pinball region is defined by setting the
parameter of PINB, which is the radius centering on the Gaussian point. To help
convergence, an aggressive refinement of stiffness is chosen by setting KEYOPT (6)
= 2.

5.3.1.3.2 Real constants The contact element and target element are associated
together by sharing the same real constant set as described next. Based on the
problem studied and the element key options selected, six real constants are singled
out for further study. They are the normal penalty stiffness factor FKN, penetration
tolerance factor FTOLN, tangent penalty stiffness factor FTN, allowable elastic slip
SLTO, pinball region PINB, and contact opening stiffness FKOP. The normal penalty
stiffness factor FKN is used to modify the default normal contact stiffness, which is
determined by material properties, element size, and the total number of degrees
of freedom in the model. The usual factor range of FKN is from 0.01 to 1. Since
ANSYS 12, this factor is never influenced by plasticity of any material model defined.
Therefore in the case studied, even in the presence of the plasticity of steel material
model, this factor will not be reduced. An ideal stiffness should be one that is neither
too high to cause convergence issue, nor too low to cause too much penetration. In the
case of bending, a smaller value would be more appropriate. In the bending model,
both FKN for the contact between concrete and grout, and the contact with steel will
need to be calibrated. The factor FTOLN is used to specify the penetration range. As
long as the penetration is within this range, compatibility is satisfied. In the model,
this factor is set to the default value 0.1. The tangential penalty stiffness factor FKT is
used to modify the tangential contact stiffness, the default value of which corresponds
to the default value of FKT = 1. The tangential contact stiffness KT is proportional
to the penalty stiffness factor FKT, contact pressure, and friction coefficient. In the
bonded (always) contact, the value of friction coefficient is default to 1. The allowable
elastic slip SLTO sets the maximum sliding distance upon each update of tangential
contact stiffness at each iteration. Larger values of FTOLN sand SLTO although
can help convergence, but may compromise accuracy. In the bending model, both
FKT and SLTO are set to default values. The contact opening stiffness FKOP is the
stiffness factor applied when contact opens for the bonded-always contact. A positive
value of FKOP is a scalar that when multiplied by the closed contact stiffness gives the
contact opening stiffness. The value of PINB is determined by considering the target
surface, contact surface behavior, and deformation setting. In the model studied,
the contact is bonded (always), and large deformation is turned on (NLGEOM, ON),
therefore the default value of PINB is 0.5. However, considering that there are several convex regions of the U-bar, a smaller value 0.1 is used for PINB.

5.3.1.4 Debonding model

Debonding happens either as separation in the normal direction or the sliding in the tangential direction or both. When the normal contact stress exceeds the maximum normal contact stress specified, separation occurs. Similarly, when the tangential contact stress exceeds the maximum tangential contact stress, sliding occurs. Debonding under bonded-always contact and pure penalty method is modeled using cohesive zone model (czm) (Alfano and Crisfield, 2001). Two bilinear material models can be used to define czm, one is by specifying tractions and separations and the other is tractions and critical fracture energies. In this study the first model is used. It should be noted that the area covered by the bilinear traction-separation curve gives the critical fracture energy. In this sense, these two bilinear material models are equivalent. There are three types of debonding behaviors. Mode I debonding (see Figure 5.17) is separation-dominated, which means the slip constitutive behavior follows the separation constitutive behavior. Debonding completes when the normal contact stress reaches zero. In this mode, the maximum tensile stress (sigma) and the separation gap at the completion of debonding (u) need to be specified. Mode II debonding is slip dominated. The parameters required for this mode include the maximum tangential stress and the slippage at the completion of debonding. Mode III debonding is a mixture of Mode I and Mode II. In the current study, the Mode I debonding model is chosen for the contact between concrete and grout and the Mode II debonding model is chosen for the contact with steel.

In the FE model, the interface between the UHPC and the concrete is assumed to be the location of the crack (debonding). In the real case, however, since the bond
is stronger than concrete, cracking actually occurs in the concrete. Therefore, in the model, the maximum normal contact stress should come from the properties of concrete. In a previous study (Julander, 2009), the bond is weaker than the concrete, and this value is set to be 23 percent of the tensile stress of concrete. In the current study, the direct tensile stress from concrete cylinder test at the comparable age is about 400 psi. Considering the material variability and inherent defects in the direct tension tests, a range from 200 psi to 600 psi is chosen. This value, together with the separation gap at the completion of debonding in the czm will be determined later during the calibration process.

For the contact between UHPC with steel reinforcement, Perry and Royce (2010) performed direct tensile pull-off tests on rebar with various diameters (0.511 in., 0.629 in. and 0.748 in.) which were either in epoxy coated or black steel bars. The UHPC used had steel fibers in it and the compressive strength at 28 days was

![Figure 5.17: Mode I debonding model](image_url)
20 ksi. The failure behavior for all the tests was rebar failure. In the current study, #4 deformed bar is used, which is comparable to one type of rebar used in Perry’s study. The influence of PVA fiber in UHPC on the pull-out behavior is neglected. As such sliding between rebar and UHPC is deemed to never happen, and the values of sliding stress and sliding distance at the completion of debonding are set to 60 ksi and 2 in.

For the contact between concrete and steel, Tastani and Pantazopoulou (2002) carried out direct tensile pull-off tests between rebar ($f_y = 60$ ksi, diameter = 0.55 in, clear cover = 1.7 in) and normal strength concrete with a compressive strength of 4.5 ksi at test. The bond stress - rebar slip relationship (see Figure 5.18) showed a maximum debonding stress of about 550 psi and the tangential slipping distance at the completion of debonding approaching to 1 in. According to Tastani’s study, the eccentric pullout test results in much larger bond stress than that in the direct tensile pull-off test due to increasing friction caused by rebar curvature. Since a higher strength concrete and rebar with smaller diameter (0.5 in.) are used in the current study, a higher value of maximum debonding stress 600 psi is applied, and a value of 1 in. is assigned to the tangential slip distance at the completion of debonding. For the bending model, these two values could be larger.

For each debonding model, damping is used to help convergence. Its value should be smaller than the smallest time step so that the separation or sliding stress can be correctly identified. In the model, a value of 0.00001 is assigned to damping. A summary of the parameters used in the FE model are listed in Table 5.2.

5.3.1.5 Meshing

Meshing is an important process in finite element modeling in the sense that the results and computational time are sensitive to it. A finer mesh can improve
the results but may also increase the computational time considerably. For the model considered, ANSYS recommends the use of brick elements for solid 65, which is simple to be implemented in the beam. For the shear key, however, due to the curvature of the U-bar, this requirement is not easy to be achieved. Nevertheless, efforts were made to divide the volume of the shear key into smaller volumes to facilitate the mesh using brick elements. The region of the U-bar curvature is specially addressed by using elements with smaller sizes. The divided volume is shown in Figure 5.19 and the final mesh is shown in Figure 5.20.

5.3.1.6 Boundary conditions and applied displacements

The shear key FE model is used to determine the translational stiffness of the shear key in the global X, Y, and Z directions and the bending stiffness around the global Z direction. As such four finite element models were created with different bounding conditions aimed at determining the required stiffness terms for the stiffness matrix of the shear key.
Table 5.2: Parameter summary in the shear key FE model

<table>
<thead>
<tr>
<th>Parameters</th>
<th>Value</th>
<th>Note</th>
</tr>
</thead>
<tbody>
<tr>
<td>FKN</td>
<td>0.01-1</td>
<td></td>
</tr>
<tr>
<td>FTOLN</td>
<td>0.1</td>
<td>default</td>
</tr>
<tr>
<td>FKT</td>
<td>1</td>
<td>default</td>
</tr>
<tr>
<td>Mu</td>
<td>1</td>
<td>default for bonded (always) contact</td>
</tr>
<tr>
<td>SLTO</td>
<td></td>
<td>default</td>
</tr>
<tr>
<td>PINB</td>
<td>0.1</td>
<td></td>
</tr>
<tr>
<td>FKOP</td>
<td>0-1</td>
<td></td>
</tr>
<tr>
<td>damping</td>
<td>0.00001</td>
<td>Smaller than the smallest time step</td>
</tr>
</tbody>
</table>

Normal contact stress at the completion of debonding: 200 psi-600 psi to be calibrated.

Tangential contact stress at the completion of debonding: 60000 psi contact between UHPC and steel.

Tangential contact stress at the completion of debonding: 600 psi contact between concrete and steel.

Figure 5.19: Volume division for mesh using brick elements

5.3.1.6.1 FEM-RZ This model is used to determine the shear key bending stiffness around the global Z axis. For this analysis, the nodes located on the bottom corners of the exterior left face of the left beam flange and the exterior right face of
the right beam flange are restricted in the Y direction. Additionally, translation along the X axis (transversal direction) is restricted on the bottom nodes of the exterior left face of the left beam to create a simply supported condition. External displacement is applied to the line of nodes located 1.5 in. from each side of the shear key to create a pure bending situation. Finally, translation along the Z axis (longitudinal direction of the bridge) was restricted on the front and back faces of the model so that there would be no longitudinal deformation. This reason behind this is that in the real bridge, a 4-in.-wide section longitudinally should not cause longitudinal deformations due to the restraints given. Indeed this could be termed a plane strain problem. Refer to Figure 5.21 for the detailed boundary conditions and applied displacements.

5.3.1.6.2 FEM-X This model is to determine the translational stiffness in the transverse direction, which is in alignment with global X axis in the FE model. For this analysis, the nodes located on the exterior left face of the left beam flange were
restricted in the X direction and external displacement in the X direction was applied to the nodes on the exterior right face of the right beam. Additionally, translation along the Y axis (vertical direction) was restricted on the bottom nodes of the exterior face of the left beam to create a fixed support condition. Finally, translation along the Z axis (longitudinal direction of the bridge) was restricted on the front and back faces of the model for the same reason as mentioned in Section 5.3.1.6.1. Refer to Figure 5.22 for the detailed boundary conditions and applied displacements.
5.3.1.6.3 FEM-Y  This model is to determine the translational stiffness in the vertical direction, which is in alignment with global Y axis in the FE model. For this analysis, the bottom nodes and top nodes of the left beam flange were restricted along the Y axis and external displacement in the Y direction was applied to the bottom and top nodes of the right beam. Additionally, translation along the X axis (transverse direction of the bridge) was restricted on the bottom nodes located on the exterior left face of the left beam to create a simply supported condition. Finally, translation along the Z axis (longitudinal direction of the bridge) was restricted on the front and back faces of the model. Refer to Figure 5.23 for the detailed boundary conditions and applied displacements.

5.3.1.6.4 FEM-Z  This model is to determine the translational stiffness in the longitudinal direction, which is in alignment with global Z axis in the FE model. For this analysis, the nodes located on the exterior front and back faces of the left beam flange were restricted in the Z direction and the external displacement in the Z direction was applied to the nodes on the front and back faces of the right beam flange. Additionally, translation along the X axis (transverse direction of the bridge)
and the Y axis (vertical direction) was restricted on the bottom corner nodes of the left and right beam. Finally, considering that in the real bridge the rotation of the shear key along Y axis would not occur, translation along the X axis was restricted for all the nodes inside the shear key. Also nodes on the front and back faces of the shear key are coupled together to prevent the relative movement. Refer to Figure 5.24 for the detailed boundary conditions and applied displacements.

5.3.1.7 Response Monitoring

5.3.1.7.1 FEM-RZ To get the bending stiffness, both bending moment and the rotation in the shear key are needed. In the FE model, the shear key is in pure bending. Bending moment in the shear key is obtained by using the reaction force multiplied by the moment arm. As far as shear key rotation is concerned, in the experiment, top and bottom LVDTs were put across the shear key and cracks in the adjacent concrete to determine the shear key rotation. A similar method is used in the FE model where displacement response is acquired at the interface points of the bottom and on the top edges on the concrete slab rather than the shear key (see Figure 5.25). After all, it is the stiffness of the shear key with adjacent cracked concrete that matters when determining demands rather than the shear key itself.
5.3.1.7.2 **FEM-X/Y/Z** For these three FE models, the displacement responses are monitored at the shear key interface nodes on the beams for the determination of the average relative movement in the X/Y/Z direction. The reaction forces are collected to calculate the force on the shear key in the X/Y/Z direction. In this way the load-displacement curves of the shear key are obtained.

**5.3.2 Feature Selection**

The shear key FE model in bending is calibrated against the moment-rotation curve from the experiment labeled STA-04. A conversion is made of the curve to change from a 40-in.-wide slab in the experiment to the 4-in.-wide shear key in the model (see Figure 5.26). Here in the curve, the elastic-cracked stiffness is only one value. In the real case however, within the fatigue load level, the elastic-cracked rotational stiffness changes its magnitude depending upon the level of cracking in the concrete adjacent to the shear key. In another words, the elastic-cracked stiffness is actually within a range, the upper bound of which is the pre-cracking stiffness and the lower bound is the stiffness when the first steel yields. This influence of this
range of shear key stiffness on the measured load demands is considered later in a sensitivity study. During the calibration, both the pre-cracking behavior and post-cracking behavior are used as features. It is expected that through this process, all the sensitive parameters can be calibrated.

![Converted moment-rotation curve](image)

Figure 5.26: Converted moment-rotation curve

### 5.3.3 Parameter Calibration

The calibration underwent two phases. These were the calibration of the pre-cracking behavior and of the post-cracking behavior of the moment-rotation curve. Models are calibrated against the specimens under high moment test. For the specimen in the high shear test, the shear effect on the moment-rotation curve of the joint cannot be neglected.
### Table 5.3: Correlation coefficients

<table>
<thead>
<tr>
<th></th>
<th>FKN(CG)</th>
<th>E_conc</th>
<th>E_UHPC</th>
<th>FKN_CS</th>
<th>Pre-cracking stiffness</th>
</tr>
</thead>
<tbody>
<tr>
<td>FKN(CG)</td>
<td>1</td>
<td>-0.0813</td>
<td>-0.1578</td>
<td>0.1488</td>
<td>0.8839</td>
</tr>
<tr>
<td>E_conc</td>
<td>-0.0813</td>
<td>1</td>
<td>0.5440</td>
<td>-0.0332</td>
<td>0.3298</td>
</tr>
<tr>
<td>E_UHPC</td>
<td>-0.1578</td>
<td>0.5440</td>
<td>1</td>
<td>0.0410</td>
<td>0.2058</td>
</tr>
<tr>
<td>FKN_CS</td>
<td>0.1488</td>
<td>-0.0332</td>
<td>0.0410</td>
<td>1</td>
<td>0.2222</td>
</tr>
<tr>
<td>Pre-cracking stiffness</td>
<td>0.8839</td>
<td>0.3298</td>
<td>0.2058</td>
<td>0.2222</td>
<td>1</td>
</tr>
</tbody>
</table>

### 5.3.3.1 Pre-cracking behavior

For the pre-cracking stiffness, an intensive sensitivity study shows that the pre-cracking stiffness is sensitive to only the following parameters: normal penalty stiffness factor between concrete and grout FKN(CG), normal penalty stiffness factor for the contact with steel FKN_CS, Young’s modulus of concrete, and Young’s modulus of UHPC. A full-factorial sensitivity study of the above mentioned parameters was carried out with FKN(CG) ranging from 0.025 to 0.08, E_conc from 5000 ksi to 6600 ksi, E_UHPC from 5600 ksi to 8000 ksi, and FKN_CS from 0.001 to 1 (Figure 5.27). The correlation between these parameters and the pre-cracking stiffness (Table 5.3) shows that within the ranges specified, the pre-cracking stiffness is most sensitive to FKN(CG), followed by E_conc, while it is relatively insensitive to E_UHPC and FKN_CS. This implies that the pre-cracking stiffness is controlled by the weak material. With E_UHPC at a value of 7200 ksi, and E_conc at a value of 5600 ksi, FKN(CG) is calibrated to be 0.048. It is also observed that with other parameters at their nominal values, a five percent divergence from the pre-cracking stiffness can either result from E_conc ranging from 5200 ksi to 6000 ksi, or E_UHPC ranging from 6000 ksi to 8400 ksi, implying that the material variability from batch to batch has insignificant impact on the pre-cracking stiffness.
Figure 5.27: Sensitivity study of pre-cracking behavior

5.3.3.2 Post-cracking behavior

With all the other parameters at their nominal values or default values, the post-cracking behavior is found out to be quite sensitive to FKN_CS, fracture energy (sigma and gap at the completion of debonding), and relatively insensitive to the stiffness factor after cracking FKOP. A combination of FKN_CS = 0.001, sigma = 230 psi, gap = 0.0013 and FKOP = 0.5 gives a post-cracking behavior that agrees fairly well with the experimental data. Since the magnitude of elastic-cracked stiffness varies depending upon the seriousness of the interface cracks, a displacement study and full cycle study are carried out. Under a displacement of 0.017in, the model is unloaded and loaded to the same displacement. The calibrated moment-rotation curve is shown in Figure 5.28a. A small deviation exists between the post-cracking curves, which is mainly due to the displacement exerted. A larger displacement could bring these two curves closer, but at a significant cost of computational time. Since the post-cracking stiffness determined this way from the FE model is close to that
from the experiment, the calibration is deemed to be sufficient. As such the stiffness term K66 is obtained by fitting the curve to a straight line. Based on the bending model, the stiffness term of K26 (the force in y direction caused by a unit rotation) is also determined. It is also found out that a 5 percent divergence from the pre-cracking stiffness does not have much influence on the elastic-cracked stiffness (see Figure 5.28b).

For other material combinations under high moment tests, the calibrated moment-rotation curves for a shear key which is 4 in long in the longitudinal direction are shown in Figure 5.29.

![Calibrated moment-rotation curve of STA-04](image1)

![Sensitivity to material variability](image2)

Figure 5.28: Calibrated moment-rotation curve of STA-04

### 5.3.4 Stiffness Matrix Determination

This calibrated bending model is then used to get stiffness terms from other models including the model subjected to axial tension in the direction of X and the models subjected to shear in both Y and Z directions. For each model, several studies are conducted including a displacement study, support study, sensitivity to
other parameters other than those calibrated above, and sometimes full-cycle study. As in the bending model, the stiffness in these other models is not sensitive to other parameters. For the model subjected to axial tension, cracking is deemed to occur under fatigue loads and the elastic-cracked stiffness is used. For the other two models subjected to shear, the pre-cracking stiffness is used. The load-displacement curve for each model is displayed in Figure 5.30, and the final stiffness matrix for each material combination is given in Figures 5.31-5.33. Considering the later use of these stiffness matrix for a 6-in-long shear key in the bridge FE model, the stiffness terms given are all for the 6-in-long shear key.

5.4 Closure

This chapter mainly talks about the method that is used for demand evaluation through the use of a bridge FE model and a shear key FE model. The shear key FE model provides the shear key stiffness matrix calibrated through experimental data, which is then converted and fed into the bridge FE model for further demand determin-
Figure 5.30: Load-displacement curve for stiffness determination

\[ \begin{bmatrix}
1 & 4651 & 0 & 0 & 0 & 0 & 0 \\
2 & 1478 & 0 & 0 & 0 & 52.4 & 0 \\
3 & 9170 & 0 & \cdots & 0 & \cdots & 0 \\
4 & \cdots & 0 & 0 & \cdots & 0 & \cdots \\
5 & Sym. & \cdots & 0 & \cdots & \cdots & 0 \\
6 & & & & & 14.4 & \end{bmatrix} \]

Figure 5.31: Joint (6 in-long) stiffness matrix for grout with PVA combination (STA-01) \(10^5\) N, m, and rad)
nation. This method requires an appropriate modeling of the bridge FE model, and of equal importance, a reasonable stiffness matrix obtained through the calibration of the shear key FE model.

The shear key FE model is based on the assumptions that the lumped crack at the interface has the same effect on stiffness as distributed cracks. In the model, the bond is stronger than the concrete so that the debonding properties are controlled by the concrete. The stiffness matrix is obtained depending on the parameters from the calibrated shear key FE model subjected to bending, which is calibrated against the moment-rotation curve from the experiment. The shear key stiffness is more sensitive to the rigidity of the weak material (concrete) than the strong material (UHPC with PVA).
There are many factors that can influence the stiffness matrix. The material properties could change and the concrete and the UHPC can undergo creep and shrinkage. The general formula for Young’s modulus may not be suitable for a particular case. The debonding model or certain parameters suitable for the bending FE model may not be appropriate for the shear FE model. The lumped crack may exert different effects on the stiffness than the distributed cracks. Moreover, the elastic-cracked stiffness covers a range rather than a single value. Talking about all of these factors, however, does not discourage the use of FE model. Rather, the model is valuable in the sense that it provides a reference for the stiffness to be used, and it is robust in the sense that out of the so many parameters, only a few of them have significant influences on the stiffness. In addition, the stiffness is not influenced that much by material properties. Varying $E_{\text{conc}}$ from 5200 ksi to 6000 ksi, or $E_{\text{UHPC}}$ from 6000 ksi to 8400 ksi results in pre-cracking stiffness within five percent divergence and little change of elastic-cracked stiffness. After all, the objective of obtaining a shear key stiffness matrix is to determine the demands on the NEXT-D beam bridge. Therefore rather than figuring out all the uncertainties involved, a sensitivity study of the shear key stiffness’s influence on the demand distribution is more direct and effective, which will be talked about in the next chapter.
Chapter 6

NEXT-D Bridge Design

A highway bridge is designed on a component basis. In this project, the following components of a typical NEXT-D bridge are designed: parapet, overhang, beam and deck.

6.1 Parapet and Overhang Design

The parapet designed is a New Jersey type of parapet. It is designed to the load level TL4 (LRFD Article 13.7.2) under the Extreme Event II limit state based on yield line analysis. When performing yield line analysis, the critical length of yield line failure pattern and the nominal resistance are determined using formulas in LRFD Article A13.3.1 with the following assumptions.

1. The parapet is of uniform thickness;
2. The overhang is strong enough to force the yield line pattern to remain within the parapet;
3. The parapet wall must be long enough so that the yield line pattern can happen;
4. The positive and negative wall resisting moments are equal.
A yield line analysis for parapets with variable thicknesses was conducted by Calloway Calloway (1993). The LRFD method is four percent conservative according to Calloway and at the same time more computationally convenient. During the design, it is found that the end zone impact causes more demands than those resulting from the impact within the wall segment. As such, the middle zone and end zone were designed separately. The current parapet design in SCDOT using #5@12in throughout the entire length of the parapet is designed to the end impact condition and thus produces a conservative design in the middle zone. The deck overhang must be designed to have a larger capacity than the parapet it supports. Therefore, the overdesign in the middle zone of the parapet also causes a more stringent design to be used for the design of the overhang. The benefit, however, is that it is more construction friendly and the same reinforcement design can be used throughout the overhang. Finally this conventional design, as per the SCDOT’s request, is used.

The basic concept for the deck overhang design is that the overhang must be stronger than the parapet; so that the damage will remain in the parapet as much as possible which is more easily repaired than the overhang. The standard overhang width for NEXT-8 is 2.5 ft., and for NEXT-6 is 1.5 ft., which does not satisfy the minimum requirement (2 ft.-3 in.) specified in SCDOT Bridge Design Manual 12.2.5.5 to accommodate drainage (SCDOT, 2006). As such, the overhang width is set to be 2.5 ft. for both NEXT-6 and NEXT-8 and any width section in between. For the overhang design, the Extreme Event II limit state controls. The overhang is designed considering not only the negative moment resulting from the transverse collision load on the parapet, but also the axial tension caused by the collision load. The collision load effects on the overhang are calculated by assuming that the collision loads spread at a slope of 45 degree from the top of the parapet to the interface. Therefore when the impact happens within the wall, the spreading length of the collision load at the
interface is two times the height of the parapet; and when the impact happens at the end of the wall, the spreading length is the height of the parapet. When calculating the axial tensile capacity of the overhang, the transverse rebar are assumed to all yield.

6.2 Beam Analysis and Design

6.2.1 Refined analysis using 3D finite element model

LRFD Article 4.6.2.2.2b specifies that for stemmed beams with shear keys, if the stem spacing is less than 4 ft. or more than 10 ft., a refined analysis needs to be performed to determine the live load flexural moment for interior beams (AASHTO, 2012). For both the NEXT-6 and NEXT-8, the stem spacing is 3 ft.; therefore a refined analysis is needed. The requirement however does not necessarily mean that the AASHTO live load distribution formulas do not apply to the NEXT-D Beam Bridge, rather it means that the formulas were developed without considering the above ranges. Three dimensional finite element models of 40 ft. span NEXT-6 and NEXT-8 bridges were built using SAP2000 by Funcik, (2011) as discussed in the previous chapter. It should be noted that those models are base models. There are a few updates about this model concerning geometry, material properties, and component stiffness as follows.

6.2.2 Geometry and material properties

The overhang length for both the NEXT-6 and NEXT-8 is updated to 2.5 ft. measured from the centerline of the exterior stem of the exterior beam to the edge of the overhang. As such, the width of the NEXT-8 Bridge remains 48 ft., while
the width of the NEXT-6 Bridge is expanded to 50 ft. According to LRFD Article 4.6.3.1, the structurally continuous parapet, acting compositely with the deck, can be considered to be structurally active at service and fatigue limit states (AASHTO, 2012). Therefore the parapets are present in the model when determining the load distribution factors and demands to ascertain their influence. However, to be consistent with the requirements of the SCDOT Bridge Design Manual Section 14.1.1.2 the parapet was not included in the capacity calculation of the bridge. The stem depth to be designed for both the NEXT-6 and NEXT-8, after a discussion with SCDOT, is determined to be 13 in. giving an overall section depth of 21 in. In the bridge FE model, this depth used remains 12 in. as determined by Deery (2010) to be the minimum feasible depth. To account for the effect of stem depth on demands, a sensitivity study is carried out later.

Young’s modulus, used for the parapet, is 3600 ksi which corresponds to a design compressive strength of 4000 psi. For the precast beam, Young’s modulus is 5600 ksi corresponding to a compressive strength of 9600 psi according to the experimental data. Notice that the design compressive strength for the precast at service is 6500 psi, which results in a smaller Young’s modulus. However, this difference in Young’s modulus should not be a concern for live load distribution factor determination. In addition to the 40 ft. span, NEXT-D bridges with span lengths of 30 ft. and 22 ft. are also modeled and designed. To be clear, the design strength used for the actual capacity design of the beams later in this chapter is 6500 psi. The 9600 psi actual strength of this 6500 psi design strength was to capture the actual behavior for demand determination sensitivity.
6.2.3 Component stiffness

Relative stiffness among components is important for demand determination. For the shear key stiffness, the matrix obtained from the shear key FE models is used as a target. Before this matrix is used, a few modifications are made. First, the matrix is based on a 4 in. wide shear key FE model. It is converted to be used for the 6 in. spaced shear key frame elements by multiplying a factor of 1.5 assuming that all the stiffness terms are proportional to the shear key width. Second, there is a term missing in the matrix, which is the rotational stiffness K44. Since this term is necessary, the torsional constant of the shear key frame element is assumed. Considering that it is much easier for the shear key to bend than to rotate, a value which is ten times that of the bending stiffness is assigned to the torsional stiffness. A sensitivity study using the design tandem load shows that the transverse demands are insensitive to the change of this term. For the NEXT-6, by amplifying or decreasing this term 100 times, the change of maximum positive moment is within 1.5 percent, and the change of critical negative moment is within 6 percent. For the NEXT-8, the percentages are within 1 percent for maximum positive moment, and 1.5 percent for critical negative moment. Based on the updated stiffness matrix, as given in Figure 6.1, the input parameters are determined for the shear key frame element. Refer to Appendix A for the detailed calculation shown in a spreadsheet. Since the calculated length of the frame element is smaller than 8 in., equal constraints are applied to both ends of the shear key frame element, which gives the desired stiffness terms in the matrix.
6.3 Live Load Distribution Factor

6.3.1 Definition

The live load distribution factor (LDF) is defined as “the critical load actions (either moment or shear) under either a single design truck or multiple design trucks spaced transversely based on refined analysis, multiplied by multiple presence factors specified in AASHTO LRFD Table 3.6.1.1.2-1, the result of which is then divided by the corresponding load actions obtained from beam line analysis under a single design truck”. The current live load distribution factor formulas in the LRFD Design Specifications are based on design trucks and already include multiple presence factors except for those calculated based on the lever rule method. For the NEXT-D beam bridges with varying span lengths, if the load distribution factors are close to that based on AASHTO LRFD formulas, the beam design would be much more efficient by using a commercial bridge design software rather than performing refined analyses.

6.3.2 Beam line analysis

The design truck as described in LRFD Article 3.6.1.2.2, with a distance of 14 ft. between the two 32 kip axles, is applied to both the beam line analysis and the
bridge model for maximum load effects. When performing a beam line analysis, the critical demand is determined following the guidance below:

1. The maximum shear due to moving concentrated loads occurs at one support when one of the loads is at the support.

2. The maximum bending moment produced by moving concentrated loads occurs under one of the loads when that load is as far from one support as the center of gravity of all the moving loads on the beam is from the other support. The critical moment happens under the load that is closest to the center of gravity of all the moving loads on the beam.

Therefore, for shear, the rear 32 kip axle is positioned on the support. The critical load configuration for moment depends on the span length. For the 40 ft. span the three-axle truck dominates — see Figure 6.2; for the 30 ft. span the two—32 kip axle condition controls (Figure 6.3); and for the 22 ft. span, the single 32 kip axle condition controls when the axle is placed in the middle of the span. The critical demands based on beam line analysis are listed in Appendix B.

![Figure 6.2: Critical load position for the 40 ft. span in beam line analysis](image)
6.3.3 Critical demands based on 3D FE model

Since there are three design lanes (LRFD Art. 3.6.1.1.1) for the bridges under design, a maximum number of three design trucks are positioned side by side on the bridge FE model. Wheel loads are modeled as patch loads as described in Chapter 7, which is conservative in the sense that the spreading length (depth of the deck) is not considered, resulting in a more focused load. The center of any wheel load of any design truck is placed no closer than 2 ft. from the face of the parapet as specified in LRFD Article 3.6.1.3. The critical load position for critical shear and positive moment are as described in the beam line analysis, so are the corresponding critical cross sections. In the FE model, the section cut command is used to define a critical cross section in the beam, which includes the frame joints of the stem and the shell joints of the deck. For the exterior beam, the parapet is not included in the section cut. When defining the section cut, the centroid of the section should be specified in order to get the right longitudinal moment. The load is then moved transversely -

Figure 6.3: Critical load position for the 30 ft. span in beam line analysis
element by element - in order to get the maximum demand at the critical locations for both the interior beam and exterior beam.

6.3.4 Live load distribution factor determination

The beam demands determined from the bridge FE model and the beam line analysis are listed in Appendix B. Multiple presence factors 1.2, 1, and 0.85 are applied to the one-truck, two-truck, and three-truck load cases separately. Finally the load distribution factors are determined. For the one-truck load case, the summation of the load factors does not give 1.0, which is due to the contribution of parapet and shear key, and the precision of the load position and section cut position in the FE model. These factors are then scaled up so that the summation is 1.0 to neglect the contribution of the parapet in the design as per the SCDOT Bridge Design manual 14.1.1.2 (SCDOT, 2006). Load distribution factors in the two-truck, and three-truck load cases are processed the same way. The final load distribution factors for the NEXT-8 and the NEXT-6 are listed in Table 6.1 and Table 6.2. The values highlighted are obtained based on the lever rule method, which assumes that the deck is simply supported on the interior girders and continuous over the exterior girder. In the calculation, a whole beam is assumed as a support.

The axle load is applied 2 ft. from the face of the parapet. The reaction in the exterior beam is then multiplied by the multiple presence factor of 1.2. Comparing with the LDFs based on AASHTO’s formula for cross section I (sufficiently connected to act as a unit - AASHTO Table 4.6.2.2.2), similar results are observed for the interior beams in all the cases. For the exterior beam, AASHTO’s method gives similar shear LDFs to those from the FE model, but is notably conservative in predicting the moment LDFs. Except for a few cases, the AASHTO’s method generally gives
Table 6.1: Load distribution factors for NEXT-8

<table>
<thead>
<tr>
<th>Span length (ft)</th>
<th>Girder location</th>
<th>Number of lanes loaded</th>
<th>Positive moment 3D FEM</th>
<th>Positive moment AASHTO</th>
<th>Shear 3D FEM</th>
<th>Shear AASHTO</th>
</tr>
</thead>
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<td>40</td>
<td>Exterior</td>
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<td>0.368</td>
<td>0.813</td>
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<td></td>
<td></td>
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<td>0.590</td>
<td>0.744</td>
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appropriate LDFs. Some of the FE model results do exceed the AASHTO values by as much as 35%, however this is not true for the critical load cases. Rather, for the critical values only a few cases may exceed between 2 and 16%. This only occurs in a very few cases is hence deemed to be an acceptable range. As such the AASHTO’s approximate method is recommended for the NEXT-D beam design.

6.4 Beam Design

The beam is designed using the bridge design software CONSPAN (Bentley Systems, 2012) which includes prestressing strand and vertical reinforcement design. For a specific design, CONSPAN checks the stress of prestressing strands and the precast beam at various stages and different limit states. It does this for the bending
<table>
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and shear strengths at strength limit states. In addition it also provides a summary of deflection and camber, and information for longitudinal bonded reinforcement design in the deck and vertical reinforcement design in the beam anchorage zone. Take the NEXT-8 40 ft. span bridge for example, the following paragraphs will cover detailed design inputs including bridge modeling, material properties, and loads.

### 6.4.1 Geometry

The overall bridge width is 48 ft. The shear key is not modeled; rather its width is included in the Double-Tee beam (Figure 6.4). Therefore each beam is 8 ft. wide. The properties of the beam cross section used are those automatically calculated by CONSPAN. The left and right curb width is 19 in., resulting in a distance of 11
in. from the interior face of the parapet to the centerline of the exterior stem of the exterior beam. This information is used for calculating the live load distribution factor based on the lever rule and determining design lanes, which is three in this case. The release span length is set to the same as the total span length, which is 39 ft and 10.5 in., and the bearing to bearing distance is 39 ft. At this stage the beam cross section is type I without post-tension, but the beams are not considered sufficiently connected to act as a unit, which results in live load distribution factors that are not desired. In order to get the appropriate distribution factors, post-tension is added by checking the box ‘Post-Tensioned’ in the geometry tab. It should be noted that for NEXT-6 bridge, due to the geometry difference between the interior beam and the exterior beam, the modeling is not as straight-forward as the NEXT-8’s. The asymmetric geometry of the exterior beam cannot be directly modeled. This aspect is taken care of by utilizing the same geometry as that of the interior beam but the cross section properties are obtained from the real cross section to make sure that the beam self-weight and stress calculations are right (Figure 6.5). The effective flange width of the exterior beam is assigned 7 ft., which ensures the correct flexural resistance.

Figure 6.4: Model geometry of NEXT 8 in CONSPAN
6.4.2 Material properties

The compressive strength of the precast beam is 5200 psi at release, and 6500 psi at service. The unit weight of the precast is assigned a value of 150 pcf. It is assumed that the shear key has the same unit weight as that of the precast. For calculating the beam self-weight, considering the presence of reinforcement, this density is appropriate; while when calculating the Young’s modulus using LRFD equation 5.4.2.4-1, this density may be a little bit larger. Since high strength concrete is used and the precast has high quality control, the Young’s modulus is directly calculated based on the density of 150 pcf. A smaller Young’s modulus can be directly input in the tab however. Bonded low relaxation strands with diameter of 0.5 in., tensile strength of 270 ksi, and a Young’s modulus of 28500 ksi are used to produce a straight strand pattern. At the top of each stem (7.5 in. to the strand center from the top of the deck), two strands are provided to support the deck reinforcement. For the plain longitudinal reinforcement, the Young’s modulus is 29000 ksi, and the yield stress is 60 ksi. For vertical reinforcement, #4 reinforcing steel is used in the NEXT-D beams. Transformed area of the prestressing strand is not applied according to SCDOT Bridge Design Manual Section 15.5.6.3. SCDOT (2006).
6.4.3 Loads

The dead load of the parapet is uniformly distributed to the exterior beam and the adjacent interior beam. In CONSPAN, this load is input as a line load on the precast over the design span length. The dead load of the bituminous wearing surface, which has an average depth of 4 in., covers an area in between the inner faces of the two parapets. In CONSPAN, this load is split uniformly to each beam as line load on the precast over the design span length. The beam self-weight is automatically calculated by the software. The vehicular live load applied is HL−93 as specified in LRFD Article 3.6.1.2. All of the factors are based on the AASHTO LRFD method. Although the bridge has an ADTT less than 5000, this advantage is not taken to reduce the impact factor, and the bridge is designed for an ADTT of 5000.

6.4.4 Comments concerning beam design

When calculating the prestress loss, the approximate method is used. When checking the tensile stress in concrete before prestress losses against the limits provided in LRFD Table 5.9.4.1.2-1, it was found out that bonded reinforcement needs to be provided above the top transverse reinforcement. The area of the bonded reinforcement is calculated based on LRFD Figure C5.9.4.1.2-1. In the anchorage zone, within a distance of $\frac{h}{4}$ from the end of the beam, where $h$ is the total height of the beam, vertical reinforcement is provided as specified in LRFD Article 5.10.10.1. The area of this reinforcement is proportional to the total area of prestressing strands used. Therefore it is important to design the beam to be structurally efficient. Compared with other interior beams, the first interior beam, due to the additional parapet load exerted, has to take more demand and therefore needs more strands in some cases. Since the difference is not significant, all the other interior beams are designed the
same as the first one. The exterior beam, in some cases, due to the smaller live load distribution factors than those of the interior beam, attracts less demand and requires fewer strands. However, LRFD Article 2.5.2.7 requires that in general the exterior beam should not have less resistance than that of the interior beam. Taking this suggestion, the exterior beam is designed the same as the interior beam in these cases. The load factors calculated by lever rule in CONSPAN v12 were manually updated to the accurate values.

A hand-calculation example for the first interior beam design of NEXT−8 40 ft. span is provided in Appendix C. Refer to Appendix E for detailed drawings of all designed cases.

6.5 Deck Analysis and Design

6.5.1 Refined analysis using 3D finite element model

LRFD Article 3.6.1.3.3 specifies that when the refined methods are used to analyze decks, if the slab spans primarily in the transverse direction, only the axles of the design truck of Article 3.6.1.2.2 or design tandem of Article 3.6.1.2.3 shall be applied to the deck slab. According to the previous chapter, the most critical normalized live load demand results from the design tandem. As such a single design tandem is applied to the bridge FE model, moving across the bridge model element by element both transversely and longitudinally. The tandem is positioned as specified in LRFD Article. 3.6.1.3 such that the center of any wheel load is not closer than 2 ft. from the face of the parapet for the deck design. Article 3.6.1.3.4 of the AASHTO specifications also suggests checking the overhang design for a vertical load positioned 1 ft from the face of the parapet (AASHTO, 2012). However, this design case is not
applicable to the proposed NEXT-D section because the overhang is only 30 in. wide. This would then place the vertical load directly over the beam stem and not on the overhang. The dead loads, as described above, include beam self weight, parapet self weight, and bituminous wearing surface self weight. Section cuts are created at critical cross sections: sections beside the stem, sections in the middle of two stems of a beam, and sections besides the shear key (Figure 6.6). The live load and dead load demands at the critical sections from the FE model are then normalized by the distribution widths of 10 ft. and the bridge total length respectively to get the demands for a 1 ft. wide strip. Typical normalized dead load demand distributions and the envelope of normalized live load demand distributions (NEXT-8 40 ft. span) are displayed in Figure 6.7. The normalized live load demands are then factored by the multiple presence factor 1.2 (LRFD Table 3.6.1.1-1) and the impact factor 1.33 (LRFD Table 3.6.2.1-1). These are combined with the normalized dead load demands using load factors as specified in LRFD Table 3.4.1-1 for strength I and service I limit states (Figure 6.8).

Notice that there are two different alternating spans for the deck, one includes the shear key (span w/ key) and the other does not (span w/o key). For the span without a shear key, the positive moment demand is higher than that in the span with a shear key. This is different when compared with the AASHTO strip method discussed later, where the larger spans result in a larger positive moment. This is
because in the three-dimensional (3D) FE model, the stiffness of the shear key is not as large as that within the stem of a beam and thus attracts less positive moment demand. However, in the AASHTO strip method, the stiffness of all the components are the same and the demand distribution is controlled by the support spacing.

When determining the transverse demands, the elastic-cracked stiffness is applied to the shear key. To be consistent, the post-cracked stiffness is applied to the
deck by assigning a value of 0.35 to both bending m11 and bending m12 modifiers for the shell element, where m11 is the modifier for the transverse bending stiffness, and m12 is the modifier that represents the influence of transverse bending stiffness on torsional stiffness in local 2 direction (Figure 6.9). The transverse bending stiffness will be weakened by the existence of longitudinal cracks, which will also lead to the reduction of the torsional stiffness. Therefore a reduction factor of 0.35 is assigned for both modifiers.

Figure 6.9: Local axis for a shell element in the bridge FE model

The demands determined this way in the FE model can be used directly for the bridges studied. However, considering the future alteration of the bridge geometry, running the 3D FE model to determine deck demands is complex and time-consuming. A general formula for the deck demands will facilitate the design of NEXT-D bridges with different spans and different beam spacing. Since the AASHTO strip method is popularly used for deck demand determination, it is expected that a relationship between the 3D FE model demands and those from the AASHTO strip method can be established.
6.5.2 AASHTO method

6.5.2.1 Modeling

The AASHTO strip method assumes that the deck is continuous and the supports are all rigid. This method results in larger negative demands in the middle of the longitudinal span because it does not consider the deformation of the supports in real cases. In the One-dimensional (1D) AASHTO FE model, the deck (including the shear key) is modeled using frame elements and the stems are modeled as rigid supports (Figure 6.10). This can be performed in any basic analysis package. The real stem spacing and overhang length are used in the FE model. For the NEXT-8, the rigid supports representing the stems are spaced at alternating distances: 3 ft. and 5 ft. For the NEXT-6, the supports are spaced at 3 ft. In both the NEXT-6 and NEXT-8, the distance from the outer rigid support to the edge of the overhang is 2.5 ft. The deck is meshed into smaller frame elements with the same size, which in this case is 3 in. As far as the live load is concerned, LRFD 3.6.1.3.3 specifies that where the slab spans primarily in the transverse direction, only the axles of the design truck or design tandem shall be applied to the deck slab. It also states that the centrifugal and braking forces need not be considered in the deck design. Two axles of the design tandem result in larger critical demands than a single 32 kip axle of the design truck does. However, since the design tandem has a wider distribution width, the normalized demand on a 1 ft. cross section will be reduced. Therefore in the FE model, a single 32 kip axle load of the design truck is applied. The two 16 kip point loads spaced 6 ft. apart are applied as frame joint loads and positioned no closer than 2 ft. from the face of the parapet. In the case studied, the furthest loading position should be 43 in. (1 in. + 18 in. parapet width + 2 ft.) from the edge of the overhang. In the finite element model, this distance is 42
in., which is equivalent to the total length of 14 frame elements. Multiple trucks can be considered but with many permutations. It is therefore decided to use only one design truck with the result multiplied by the multiple presence factor of 1.2. Within the distance specified, the design truck point loads are moving together element by element, creating 141 load cases for the NEXT-8 and 149 load cases for the NEXT-6.

The dead loads on a 1 ft. strip are applied on the AASHTO FE model as line loads. The distribution of the dead loads are as that in the 3D FE model, i.e, each beam takes its own weight, which is uniformly distributed on the beam; parapet self-weight is uniformly distributed to the exterior beam and the first interior beam; and the self-weight of the wearing surface is uniformly distributed in-between the innerfaces of the two parapets.

![Figure 6.10: AASHTO model of NEXT-8 using SAP2000](image)

### 6.5.2.2 Data acquisition

For the frame element, compared with the global x, y, z axis, local axis in the case studied are as follows:

- Local axis 1 is the longitudinal axis of the element directed from end I to end J (positive x) (see Figure 6.11);
- The local 2 axis is taken as the upward z;
- The local 3 axis lies in x-y plane, and is the downward y.

Therefore the frame joint moment in local 2 direction (m2) is desired. The positive m2 is defined from the frame element end i, which, in the case studied,
results in a negative moment (tension on top of the bridge deck). Therefore when processing results, this sign is reversed so that the positive value of \( m_2 \) represents positive moment. This moment \( m_2 \) is monitored at each frame joint for each load case. The whole process—adding load patterns, applying frame joint loads, running analysis, and acquiring data—was completed by running a MATLAB code, which gives a final matrix containing \( m_2 \) at each frame element joint for each load case. However, in actual use, influence lines may be used to identify critical loading locations. According to LRFD Article 4.6.2.1, when the strip method is used, the critical positive moment and negative moment within the deck should be used for the deck design. As such, the critical positive moment and negative moment caused by the live load are recorded for later use. For the NEXT-8, the total critical positive moment is 156.73 kip-in, and the critical negative moment is -95.11 kip-in. For the NEXT-6, the total critical positive moment is 123.09 kip-in, and the critical negative moment is -51.8 kip-in. The total live load demands are then normalized using the strip widths specified in LRFD Table 4.6.2.1.3-1 for the cast-in-place concrete deck without stay-in-place concrete formwork. For the NEXT-8, the strip width is 52.4 in. for positive moment demand, and 60 in. for negative moment demand using the average stem spacing of 4 ft. For the NEXT-6, the strip width is 45.8 in. for positive moment demand, and 57 in. for negative moment demand using the stem spacing of 3 ft. The normalized dead load demand distributions and envelope of normalized live load demand distributions are displayed in Figure 6.12. For the NEXT-8, the critical normalized positive moment demand is 2.99 kip-ft/ft, and the critical normalized negative moment demand is -1.59 kip-ft/ft. For the NEXT-6, the critical normalized positive moment demand is 2.69 kip-ft/ft, and the critical normalized negative moment demand is -0.91 kip-ft/ft.
Figure 6.11: Output convention for frame element in SAP2000

Figure 6.12: Transverse demand distribution for NEXT-8 in the AASHTO model

6.5.3 Formula development

6.5.3.1 Live load effect

AASHTO makes some general assumptions of effective strip width for positive moment and negative moment based on regular layouts. For the NEXT-D beam bridge, the usual case is that the stem spacing varies. For instance in the NEXT-8, the stem spacing alternates between 3 ft. and 5 ft. Also the AASHTO strip method does not consider the influence of span length. Simulation results however show that for both the NEXT-6 and NEXT-8, as the span length increases, the normalized live
load moment demands also increase; and for the same span length, the normalized live load moment demands in NEXT-8 are always more critical than those in the NEXT-6. Based on this phenomenon, the normalized moment demands caused by the live load are considered highly correlated with design span length and stem spacing for the NEXT-D Beam Bridge. This is particularly the case for the positive moment demands.

In the 3D FE model, five span lengths are studied for the NEXT-8 (48 ft. wide) and the NEXT-6 (50 ft. wide): 22 ft., 26 ft., 30 ft., 35 ft., and 40 ft. with design span lengths of 21 ft., 25 ft., 29 ft., 34 ft., and 39 ft. respectively. The envelope distributions of the normalized live load transverse moment demand for all the five spans for both the NEXT-6 and NEXT-8 are displayed in Figure 6.13. The critical positive moment demands in the two deck spans are recorded. Meanwhile, the critical negative moment demands for both interior beam and exterior beam are also recorded. In order to establish a relationship of demands between the 3D FE model and the AASHTO strip-method, the critical positive moment demands in the 3D FE model are divided by the maximum positive moment from the AASHTO method, and the critical negative moment demands in the 3D FE model are divided by the minimum negative moment from the AASHTO method. As stated before, these critical demand ratios increase their magnitudes as the span length and average beam spacing increase (Figure 6.14). For simplicity, the ratios are modified so that they are linearly dependent upon the span lengths. It is observed that for the NEXT-6, the negative moment obtained from AASHTO is much more conservative due to the rigid support assumption. In the formula, the demand ratio is assumed to be linearly dependent on the exponential form of the design span length multiplied by
the exponential form of the average stem spacing as follows:

\[ R = ax + b \] (6.1)

\[ x = \frac{l_{\text{design}}}{m} S_{\text{design}}^n \] (6.2)

where, \( R \) is the demand ratio, \( l_{\text{design}} \) is the design span length (ft), which is the distance between supports; and \( S_{\text{design}} \) is the average stem spacing (ft). For the NEXT-6, the average stem spacing is 3 ft. For the NEXT-8 with an alternating stem spacing of 3 ft and 5 ft, the average stem spacing is 4 ft. Other parameters including \( a \), \( b \), \( m \), and \( n \) are constant coefficients, of which \( m \) and \( n \) are determined first so that the demand ratios of the NEXT-6 and NEXT-8 are approximately on a linear line (Figure 6.15). After that, the equation of the line is obtained and \( a \) and \( b \) are determined. Following this procedure, the formulas for the live load demands are given below. The negative moment in the interior beam is directly taken as the AASHTO FE model value, which is proper for NEXT-8 and fairly conservative for NEXT-6 (Figure 6.14c) due to the rigid support assumption. For bridges of 22 ft. span and 30 ft. span, the critical negative moment demand in the interior beams of NEXT-6 from AASHTO FE model is so conservative that it even exceeds the critical demand in the exterior beam, which, according to the 3D simulation, is not correct. Therefore when applying the formula for negative moment demand in the interior beam, this value shall not exceed that in the exterior beam.

**Positive moment demand in the span without shear key:**

\[ M_{\text{positive}} = M_{p,\text{AASHTO}}[0.77 + 0.0027(l_{\text{design}}^{1.3} S_{\text{design}}^{1.4} - 244)] \] (6.3)
Positive moment demand in the span with shear key:

\[ M_{positive} = M_{p,AASHTO}[0.58 + 0.0196(l_{design}^{0.8} - 51)] \] (6.4)

Negative moment demand in the exterior beam:

\[ M_{negative} = M_{n,AASHTO}[0.4 + 6.28(l_{design}^{0.2} - 1.69)] \] (6.5)

Negative moment demand in the interior beam:

\[ M_{negative} = M_{n,AASHTO} \] (6.6)

where \( M_{p,AASHTO} \) and \( M_{n,AASHTO} \) represent the critical positive and negative moment demands separately in the 1-D analysis model caused by a single 32-kip axle of the design truck, and \( M_{positive} \) and \( M_{negative} \) represent the critical positive and negative moment demands separately caused by the live load to be used for design. It
should be stressed that when dealing with the negative moment demand in the interior beam, the value shall not exceed that in the exterior beam. The above formulas apply to both the NEXT-6 and the NEXT-8 and any width in between. The range of applicable spans, without further verification, is between 22 ft. to 40 ft. The formulas above are based on a constant stem depth of 12 in. and do not consider the change of the shear key stiffness. Increasing the stem depth or decreasing the shear key stiffness will decrease the transverse demands in
the deck as talked about later. Thus, the developed formulas provide a conservative and acceptable demand estimate for stem depths of 12 in. or greater.

6.5.3.2 Dead load effect

Dead load effects - including the demands from beam self-weight, parapet self-weight, and wearing surface self-weight - are also determined based on the 1D FE model. The dead load demands at each frame element joint are then combined together for Strength I and Service I limit states using the factors as specified in
LRFD Table 3.4.1-1 and Table 3.4.1-2. The moment distributions due to dead load effect for each limit state are given in Figure 6.16 and Figure 6.17. The factored critical positive and negative moment demands due to the dead loads for each limit state are listed in Table 6.3 for later final design demand determination.

![Figure 6.16: Factored dead load effect for NEXT-8](image1)

(a) Strength I limit state  
(b) Service I limit state

![Figure 6.17: Factored dead load effect for NEXT-6](image2)

(a) Strength I limit state  
(b) Service I limit state
Table 6.3: Factored moment demand caused by dead load

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<td>Service I</td>
</tr>
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<tr>
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<td>-0.5732</td>
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</tbody>
</table>

6.5.3.3 Final design demand determination

The final design positive moment demand ($M_{p\_total}$) is determined by summing up the factored live load effect and four times the factored dead load effect (Equation 6.7). The final design negative moment demand ($M_{n\_total}$) is determined by summing up the factored live load effect and the factored dead load effect (Equation 6.8).

\[
M_{p\_total} = M_{positive} \times IM \times m \times LF + 4M_{p\_DL} \tag{6.7}
\]

\[
M_{n\_total} = M_{negative} \times IM \times m \times LF + M_{n\_DL} \tag{6.8}
\]

where,

$M_{positive}$ = unfactored positive moment demand from live load based on Equation 6.3 and Equation 6.4, (kip-ft/ft)

$M_{negative}$ = unfactored negative moment demand from live load based on Equation 6.5 and Equation 6.6, (kip-ft/ft)

$M_{p\_DL}$ = factored positive moment demand from dead load, (kip-ft/ft)

$M_{n\_DL}$ = factored negative moment demand from dead load, (kip-ft/ft)

$IM$ = dynamic load allowance percent, $IM = 1.33$ (LRFD Table 3.6.2.1-1)

$m$ = multiple presence factor, $m = 1.2$ (LRFD Table 3.6.1.1.2-1)
$LF =$ load combination factor, $LF = 1.75$ for Strength I limit state, and 1 for Service I limit state (LRFD Table 3.4.1-1)

The multiplier 4 for the design positive moment demand is to account for the large increase of positive moment demand due to dead loads as the span length increases (see Figure 6.18 and Figure 6.19). There is not much change of negative moment demand with span length change in each design limit state, therefore a multiplier of 1 is used.

Figure 6.18: Comparison between dead load effect of 3D FE model and AASHTO 1D FE model in Strength I limit state
Figure 6.19: Comparison between dead load effect of 3D FE model and AASHTO 1D FE model in Service I limit state

The design demands obtained this way are compared with the 3D FE model results (in bracket) in Table 6.4 to Table 6.7. Most of the demands from the proposed formula are conservative. The unconservative values are highlighted. Taking the 3D FE model’s results as the “correct” values, the maximum percentage error of the unconservative demands is less than six percent.

Table 6.4: NEXT-8 — Design demands provided by the formula vs demands from the bridge FE model (shown in parentheses) for the strength I limit state

<table>
<thead>
<tr>
<th>Critical section location</th>
<th>22 ft.</th>
<th>30 ft.</th>
<th>40 ft.</th>
</tr>
</thead>
<tbody>
<tr>
<td>M+ (kip-ft/ft)</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Span w/o key</td>
<td>11.02 (8.87)</td>
<td>15.30 (14.25)</td>
<td>21.18 (20.65)</td>
</tr>
<tr>
<td>Span w/ key</td>
<td>8.78 (7.00)</td>
<td>12.75 (11.31)</td>
<td>17.72 (17.12)</td>
</tr>
<tr>
<td>M- (kip-ft/ft)</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Exterior beam</td>
<td>-5.25 (-5.56)</td>
<td>-6.88 (-6.70)</td>
<td>-8.42 (-8.45)</td>
</tr>
<tr>
<td>Interior beam</td>
<td>-5.15 (-4.52)</td>
<td>-5.15 (-4.19)</td>
<td>-5.15 (-4.34)</td>
</tr>
</tbody>
</table>
Table 6.5: NEXT-8 — Design demands provided by the formula vs demands from the bridge FE model (shown in parentheses) for the service I limit state

<table>
<thead>
<tr>
<th>Critical section location</th>
<th>22 ft.</th>
<th>30 ft.</th>
<th>40 ft.</th>
</tr>
</thead>
<tbody>
<tr>
<td>M+ (kip-ft/ft) Span w/o key</td>
<td>6.67(5.10)</td>
<td>9.12(8.29)</td>
<td>12.47(12.08)</td>
</tr>
<tr>
<td>M+ (kip-ft/ft) Span w/ key</td>
<td>5.39(4.06)</td>
<td>7.66(6.62)</td>
<td>10.50(10.04)</td>
</tr>
<tr>
<td>M- (kip-ft/ft) Exterior beam</td>
<td>-3.16(-3.25)</td>
<td>-4.09(-3.94)</td>
<td>-4.97(-4.98)</td>
</tr>
<tr>
<td>M- (kip-ft/ft) Interior beam</td>
<td>-3.10(-2.61)</td>
<td>-3.10(-2.34)</td>
<td>-3.10(-2.15)</td>
</tr>
</tbody>
</table>

Table 6.6: NEXT-6 — Design demands provided by the formula vs demands from the bridge FE model (shown in parentheses) for the strength I limit state

<table>
<thead>
<tr>
<th>Critical section location</th>
<th>22 ft.</th>
<th>30 ft.</th>
<th>40 ft.</th>
</tr>
</thead>
<tbody>
<tr>
<td>M+ (kip-ft/ft) Span w/o key</td>
<td>6.46(6.15)</td>
<td>9.04(8.85)</td>
<td>12.57(12.70)</td>
</tr>
<tr>
<td>M+ (kip-ft/ft) Span w/ key</td>
<td>4.98(4.35)</td>
<td>7.81(7.37)</td>
<td>11.36(11.84)</td>
</tr>
<tr>
<td>M- (kip-ft/ft) All beams</td>
<td>-1.77(-0.98)</td>
<td>-2.65(-1.55)</td>
<td>-3.49(-2.61)</td>
</tr>
</tbody>
</table>

Table 6.7: NEXT-6 — Design demands provided by the formula vs demands from the bridge FE model (shown in parentheses) for the service I limit state

<table>
<thead>
<tr>
<th>Critical section location</th>
<th>22 ft.</th>
<th>30 ft.</th>
<th>40 ft.</th>
</tr>
</thead>
<tbody>
<tr>
<td>M+ (kip-ft/ft) Span w/o key</td>
<td>3.84(3.59)</td>
<td>5.31(5.19)</td>
<td>7.33(7.47)</td>
</tr>
<tr>
<td>M+ (kip-ft/ft) Span w/ key</td>
<td>2.99(2.51)</td>
<td>4.61(4.28)</td>
<td>6.64(6.94)</td>
</tr>
<tr>
<td>M- (kip-ft/ft) All beams</td>
<td>-1.18(-0.54)</td>
<td>-1.69(-0.82)</td>
<td>-2.16(-1.43)</td>
</tr>
</tbody>
</table>

### 6.6 Sensitivity Study

#### 6.6.1 Sensitivity of demand distribution to shear key stiffness

There are many sources that can lead to a change of shear key stiffness. In the case studied, the bond is stronger than the concrete, and the interface crack actually happens in the adjacent concrete. Therefore the material variability of the concrete is one source. Second, the elastic-crack stiffness of the shear key actually covers a range of values depending upon the seriousness of the crack. Third, the change of rebar
configuration can also change the shear key stiffness. Some of these sources can also change the stiffness of the precast beam. It is important to keep in mind that the transverse demand distribution is dependent upon the ratio of the stiffness among the relevant components rather than the change of stiffness of a single component. This sensitivity study of transverse demand distribution to shear key stiffness answers the question: How does the demand change from a condition when the shear key interface is intact to the condition where serious shear key interface cracks exist. In the 3D FE model, shear key stiffness is changed by either increasing or decreasing the modifier of moment of inertia of the shear key frame element. With the modifier of 1 as the reference value, other modifiers include 0.1, 5, 10, and 100. The design tandem load is applied, running element by element both transversely and longitudinally with no wheels closer than 2 ft. from the face of the parapet. The transverse live load demand distribution envelope corresponding to each modifier is displayed in Figure 6.20.

For the NEXT-8, the following trends are observed. It is seen that for the positive moment demand, as the stiffness modifier changes from 1 to 0.1, there is a big drop in the demand in both the shear key and the precast beam. The shear key acts similar to an internal hinge when the modifier is 0.1, meaning severe cracks happened. As the modifier changes from 5 to 100, there is only a very small change of the demand distribution and magnitudes, implying the shear key interface is close being fully intact. For the negative moment, the increase of shear key stiffness attracts more demands to the shear key itself, but subtracts demands in the precast beam. There is little change of demands as the modifier increases from 1 to 100. The trend seen in the negative moment is also seen for the shear. For the NEXT-6, the change of positive moment demand distribution is similar to that of the NEXT-8, with the only difference being for the negative moment demand. As the shear key stiffness increases, both the negative moment demands in the key and in the precast increase.
There is not much change of negative moment demand as the modifier changes from 1 to 100. As in the NEXT-8, the change of shear is similar to the change of negative moment demand.

6.6.2 Sensitivity of demand distribution to stem depth

As the bridge span length increases, the stem depth may also need to be increased. The increase in stem depth will result in a larger stem stiffness, causing demand redistribution. To better understand the influence of stem stiffness on trans-
verse demand distribution, for each span length, a sensitivity study is carried out using different stem depths: 12 in. (current), 16 in., and 20 in., which correspond to an overall section depth of 20 in., 24 in., and 28 in. (NEXT 20D, NEXT 24D, NEXT 28D) respectively. The live load demand distribution envelope is displayed in Figure 6.21. Similar trends are found for all of the cases: as the stem depth increases, the positive moment demands both in the shear key and the precast beam decrease. There is very little change of negative moment demands and shear demands. The increase in stem stiffness attracts more demands to the stem itself, and therefore the demands in the deck decrease.

6.6.3 Sensitivity of demand distribution to Young’s modulus of concrete

The material properties of concrete used in the 3D FE model are obtained based on the cylinder test results during the specimen test day. The Young’s modulus of concrete is directly related to the component’s stiffness, and therefore the demand distribution. The current value of Young’s modulus used in the 3D FE model is 5600 ksi corresponding to a compressive strength of 9.6 ksi. Considering the material variability that is inevitably involved or the change of the concrete design strength in the future, Young’s modulus of 4415 ksi and 5100 ksi corresponding to compressive strengths of 6 ksi and 8 ksi are also investigated for each span length. Within this range of Young’s modulus (from 4415 ksi to 5600 ksi) there is little change of live load demand distribution envelope for all the cases (Figure 6.22).
6.7 Deck Design

The deck is designed based on the demands obtained from the formulas proposed. In general there are four steps to determine the design demands in Strength I and Service I limit states.

**Step 1, Determine $M_{p,AASHTO}$ and $M_{p,AASHTO}$:** Establish the 1D AASHTO FE model using the true geometry of the overhang length and stem spacing. The deck is modeled as a continuous beam and stems are modeled as rigid supports.
A single 32 kip axle of the design truck is modeled as two 16 kip point loads. The load is positioned no closer than 2 ft. from the face of the parapet as specified in LRFD Article. 3.6.1.3. Determine the critical positive and negative moment demands, which should be normalized based on the strip width specified in LRFD Table 4.6.2.1.3-1 for cast-in-place concrete deck without stay-in-place concrete formwork. In this way $M_{p,AASHTO}$ and $M_{n,AASHTO}$ are obtained. Influence lines may assist in this step.
Step 2, **Determine** $M_{\text{positive}}$ and $M_{\text{negative}}$: Use Equation 6.3 and Equation 6.4 to determine the unfactored positive moment demand due to live load for the deck span without shear key and the deck span with shear key respectively. Use Equation 6.5 and Equation 6.6 to determine the unfactored negative moment demand due to live load for the exterior beam and interior beam respectively. The negative moment demand in the interior beam shall not be larger than that in the exterior beam.

Step 3, **Determine** $M_{p_{DL}}$ and $M_{n_{DL}}$: Use the 1D beam FE model to determine the moment demands on a 1 ft. strip from dead loads (line loads). The dead loads include beam self-weight, parapet self-weight, and wearing surface self-weight. For beam self weight, each beam takes its own weight, which is uniformly distributed to the whole width of the beam. The parapet self-weight is uniformly distributed to the exterior beam and the first interior beam. The self-weight of the wearing surface is distributed uniformly in-between the inner faces of the two parapets. The dead load effects are combined based on the factors provided in LRFD Table 3.4.1-1 and Table 3.4.1-2 for Strength I and Service I limit states. The critical positive and negative moment demands are then obtained ($M_{p_{DL}}$ and $M_{n_{DL}}$) for each design limit state.

Step 4, **Determine** $M_{p_{\text{total}}}$ and $M_{n_{\text{total}}}$: The final design positive moment demand $M_{p_{\text{total}}}$ for each limit state is determined by multiplying $M_{\text{positive}}$ by IM (dynamic load allowance percent), m (multiple presence factor), and LF (load factor), the result of which is added to four times $M_{p_{DL}}$ (see Equation 6.7). The final design negative moment demand $M_{n_{\text{total}}}$ for each limit state is determined by multiplying $M_{\text{negative}}$ by IM, m, and LF, the result of which is added to $M_{n_{DL}}$ (see Equation 6.8).
The deck is then designed for capacity in Strength I limit state and cracking control in Service I limit state. For the positive moment design in both limit states, the larger positive moment demand in the span without shear key is taken care of by considering the overlapping of the development length of the reinforcement. Demand-capacity charts in Figure 6.23 provide a direct comparison between current demands for strength I and service I limit states and capacities from various rebar configurations (Table 6.8). Take the NEXT-8 with a 22 ft. span for instance, the cracking control determines the final rebar configuration, which is #4@7in.; while for the 40 ft. span, the capacity determines the final rebar configuration, which is #4@5in. The final reinforcement designs for the NEXT-8 and the NEXT-6 with various spans are listed in Table 6.9 and Table 6.10. Other requirements that are checked include minimum transverse reinforcement requirement (LRFD Article 5.7.3.3), distribution reinforcement requirement (LRFD Article 9.7.3.2), temperature and shrinkage reinforcement requirement (LRFD Article 5.10.8), and bonded reinforcement requirement (LRFD Article 5.9.4.1). Refer to Appendix D and E for detailed deck design drawings and calculations.

![Demand vs capacity provided by various rebar configurations](image)

(a) Strength I limit state

(b) Service I limit state

Figure 6.23: Demand vs capacity provided by various rebar configurations
Table 6.8: Reinforcing bar configuration capacities

<table>
<thead>
<tr>
<th>Rebar config.</th>
<th>$A_s (in^2/ft)$</th>
<th>Strength I limit state</th>
<th>Service I limit state</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>M+ (kip-ft/ft)</td>
<td>M- (kip-ft/ft)</td>
</tr>
<tr>
<td>#4@5&quot;</td>
<td>0.47</td>
<td>18.29</td>
<td>-11.47</td>
</tr>
<tr>
<td>#4@6&quot;</td>
<td>0.39</td>
<td>15.43</td>
<td>-9.88</td>
</tr>
<tr>
<td>#4@7&quot;</td>
<td>0.34</td>
<td>13.59</td>
<td>-8.90</td>
</tr>
<tr>
<td>#4@8&quot;</td>
<td>0.29</td>
<td>11.71</td>
<td>-7.80</td>
</tr>
<tr>
<td>#4@9&quot;</td>
<td>0.26</td>
<td>10.56</td>
<td>-7.05</td>
</tr>
<tr>
<td>#4@10&quot;</td>
<td>0.24</td>
<td>9.79</td>
<td>-6.55</td>
</tr>
</tbody>
</table>

Table 6.9: NEXT-8 — Final design capacity vs demand

<table>
<thead>
<tr>
<th>Span (ft)</th>
<th>Rebar config.</th>
<th>Strength I</th>
<th>Service I</th>
</tr>
</thead>
<tbody>
<tr>
<td>22</td>
<td>#4@7&quot;</td>
<td>8.78</td>
<td>13.59</td>
</tr>
<tr>
<td>30</td>
<td>#4@7&quot;</td>
<td>12.75</td>
<td>13.59</td>
</tr>
<tr>
<td>40</td>
<td>#4@5&quot;</td>
<td>17.72</td>
<td>18.29</td>
</tr>
</tbody>
</table>

Table 6.10: NEXT-6 — Final design capacity vs demand

<table>
<thead>
<tr>
<th>Span (ft)</th>
<th>Rebar config.</th>
<th>Strength I</th>
<th>Service I</th>
</tr>
</thead>
<tbody>
<tr>
<td>22</td>
<td>#4@10&quot;</td>
<td>4.98</td>
<td>9.79</td>
</tr>
<tr>
<td>30</td>
<td>#4@10&quot;</td>
<td>7.81</td>
<td>9.79</td>
</tr>
<tr>
<td>40</td>
<td>#4@7&quot;</td>
<td>11.36</td>
<td>13.59</td>
</tr>
</tbody>
</table>

6.8 Closure

This Chapter addresses the design of the parapet, deck overhang, deck and beam. The typical parapet design that has been used in the SCDOT is adopted for the NEXT-D Beam Bridge. The reinforcing steel configuration is designed to the end zone impact along the entire length of the parapet. This produces a very conservative design for the middle zone of the parapet. The effect of this over design is also felt
within the deck overhang. The overhang must be stronger than the parapet to keep
the damage within the parapet. As a result, the design of the deck overhang must
be more robust than would be otherwise required. The advantage to this approach,
however, is that the same rebar configuration (#5@12in) can be used throughout the
parapet, which is a more construction friendly detail.

The overhang is designed considering not only the negative moment caused
by the collision load, but also the axial tensile force due to the collision effect. The
precast beam is designed using the live load distribution factors from AASHTO for
cross section type I as specified in AASHTO LRFD Table 4.6.2.2.1-1, which is con-
firmed through finite element analysis. For the deck design, the live load effects are
determined using Equation 6.3 to Equation 6.6, in which, the live load effect is a
function of design span length and average stem spacing. Other parameters in the
function are obtained based on line-fit. The proposed formulas are developed based
on a constant stem depth of 12 in. and do not consider the change of shear key
stiffness. Increasing the stem depth or decreasing shear key stiffness will decrease the
transverse demands in the deck. The change of concrete Young's modulus from 4415
ksi to 5600 ksi exerts little influence on the live load demand distribution. The dead
load effect is also obtained based on the 1D FE model. The final design demands in
Strength I and Service I limit states are obtained by combining live load effects and
dead load effects together using Equation 6.7 and Equation 6.8.

The conclusions chapter of this dissertation will provide a basic and succinct
outline of the design guideline recommendations to be used for the modified NEXT-
D beam to be used in South Carolina. The recommendations are restricted to the
design space prescribed by the SCDOT which is 22 ft. 40 ft. spans and beam widths
between 6 ft. and 8 ft. The details of these designs are found in Appendix E.
Chapter 7

Summary and Conclusions

7.1 Summary

Many state departments of transportation are in need of the development of a precast concrete solution for the construction of short span bridges that meet the objectives of accelerated bridge construction but do not have restrictions on the level of traffic service and are durable. In the State of South Carolina, an alternative bridge system is needed to replace the current CIP bridges and does not have the joint durability issue that is commonly experienced by the hollow core beam bridge. With ultra-high performance concrete as shear key material, the longitudinal joint performance of the modified NEXT-D beam bridge was experimentally confirmed.

The demand evaluation required both crude and refined finite element analyses. It also relied on feedback from the experimental program to calibrate the computer models. From both the experimental activities and the computer modeling a bridge design was carried out and proposed for adoption within the State of South Carolina.
7.2 Conclusions

Major conclusions of this research are:

- For specimens that failed at the joint interface (UHPC with PVA under high shear test and grout with PVA under high moment test), fatigue test reduced both specimen strength and joint ductility. Fatigue tests had no influence on the capacities of the specimens that failed within the precast concrete slab.

- For specimens that failed at the joint interface, rebar strain indicated that both layers of rebar close to this cross section yielded during the strength tests. This implies that the anchorage length of the U-bar is sufficient and full-capacity joint can be achieved with the current rebar configuration.

- A durable bond is usually accompanied by a high stiffness of the joint material, which can increase the stiffness of the surrounding components, and reduce the overall ductility of the specimen, as shown in the case of the UHPC with steel combination. Also there is interdependency between the stiffness of the shear key and the demand that it attracts under loading. High joint stiffness will attract more demands to not only the joint, but also the surrounding components. Concerning durability, ductility, stiffness and demands, the UHPC with PVA fiber is a plausible material for use in the shear key between the NEXT-D beam. For this material combination, the joint age from 6 day to 14 day is a critical period for bond strength development. A selective open to traffic should be considered during this period in the real bridge.
• For the SCDOT NEXT-D beam bridge design, the AASHTO’s live load distribution factors for beam cross section I are suggested to be used for beam design; and for deck design, AASHTO’s strip width method is suggested to be used together with the proposed four-step demand determination procedure.

7.2.1 Design guidelines for modified NEXT-D

The researchers recommended that the South Carolina Department of Transportation adopt a precast bridge design based on using NEXT-D beam elements with a modified cross section that is more amenable to the shorter spans of interest to the Department. While this study provides critical details for the design of a modified NEXT-D bridge and the recommendation for the shear key material, the following list provides the general design guidelines that should be followed by bridge engineers:

1. For the parapet design, use the current steel reinforcement configuration as currently recommended for precast concrete hollow core bridges. One change required is the length of the dowel bar anchored into the deck needs to be changed to a 90-degree standard hook.

2. For the overhang, the design needs to consider the axial tensile force caused by the collision loads.

3. For the deck design, use the strip method. Follow this four-step procedure to determine the design demands on a 1 ft. strip for Strength I and Service I limit states:

   Step 1: **Determine** $M_{p,AASHTO}$ and $M_{n,AASHTO}$: Establish the 1D FE model beam model using the true geometry of the overhang length and stem
spacing. The deck (including the shear key) is modeled as a continuous beam and stems are modeled as rigid supports. A single 32-kip axle of the design truck is modeled as two 16-kip point loads. The load is positioned no closer than 2 ft. from the face of the parapet as specified in LRFD Article 3.6.1.3.1. However, some conservativeness is all right to accommodate the mesh in the AASHTO FE model. Determine the critical positive and negative moment demands, which should be normalized based on the strip width specified in LRFD Table 4.6.2.1.3-1 for cast-in-place concrete deck without stay-in-place concrete formwork. In this way $M_{p,AASHTO}$ and $M_{n,AASHTO}$ are obtained. Note that influence lines most certainly could be used for this step.

Step 2: **Determine $M_{positive}$ and $M_{negative}$:** Follow the formulas proposed for modifying the demand values $M_{p,AASHTO}$ and $M_{n,AASHTO}$ for the unfactored live load effects $M_{positive}$ and $M_{negative}$. These formulas are:

**Positive moment demand in the span without shear key:**

$$M_{positive} = M_{p,AASHTO}[0.77 + 0.0027(l_{design}^{1.3}S_{design}^{1.4} - 244)] \quad (7.1)$$

**Positive moment demand in the span with shear key:**

$$M_{positive} = M_{p,AASHTO}[0.58 + 0.0196(l_{design}^{0.8}S_{design}^{0.8} - 51)] \quad (7.2)$$

**Negative moment demand in the exterior beam:**

$$M_{negative} = M_{n,AASHTO}[0.4 + 6.28(l_{design}^{0.1}S_{design}^{0.2} - 1.69)] \quad (7.3)$$
Negative moment demand in the interior beam:

\[ M_{\text{negative}} = M_{n_{\text{AASHTO}}} \]  \hspace{1cm} (7.4)

where

\[ l_{\text{design}} = \text{the design span length (ft)} \]

\[ S_{\text{design}} = \text{the average stem spacing (ft)}. \]

The negative moment demand in the interior beam shall not exceed that in the exterior beam.

Step 3: **Determine** \( M_{p_{\text{DL}}} \) and \( M_{n_{\text{DL}}} \): Use the 1D FE model to determine the moment demands on a 1 ft. strip from dead loads (line loads). The dead loads include beam self-weight, parapet self-weight, and wearing surface self-weight. For beam self-weight, each beam takes its own weight, which is uniformly distributed to the whole width of the beam. The parapet self-weight is uniformly distributed to the exterior beam and the first interior beam. The self-weight of the wearing surface is distributed uniformly in-between the inner faces of the two parapets. The dead load effects are combined based on the factors provided in LRFD Table 3.4.1-1 and Table 3.4.1-2 for Strength I and Service I limit states. The critical positive and negative moment demands are then obtained (\( M_{p_{\text{DL}}} \) and \( M_{n_{\text{DL}}} \)) for each design limit state.

Step 4: **Determine** \( M_{p_{\text{total}}} \) and \( M_{n_{\text{total}}} \): The final design positive moment demand \( M_{p_{\text{total}}} \) and negative moment demand \( M_{n_{\text{total}}} \) for each limit state is determined using the following formulas.


\[ M_{p\text{,total}} = M_{\text{positive}} \times IM \times m \times LF + 4M_{p\text{,DL}} \quad (7.5) \]

\[ M_{n\text{,total}} = M_{\text{negative}} \times IM \times m \times LF + M_{n\text{,DL}} \quad (7.6) \]

where,

\( IM = \) dynamic load allowance percent, \( IM = 1.33 \) (LRFD Table 3.6.2.1-1)

\( m = \) multiple presence factor, \( m = 1.2 \) (LRFD Table 3.6.1.1.2-1)

\( LF = \) load combination factor, \( LF = 1.75 \) for Strength I limit state, and \( 1 \) for Service I limit state (LRFD Table 3.4.1-1)

The multiplier four for the positive moment demand due to dead load is to account for the big influence of span length change on critical positive moment demand. This influence, however, does not impact negative moment demand as much. Therefore a multiplier of one is used for the negative moment demand due to dead load.

4. For the beam design, use the live load distribution factors provided by AASHTO for cross section I as specified in AASHTO LRFD Table 4.6.2.2.1-1. The dead loads due to beam self-weight, parapet self-weight, and wearing surface self-weight are distributed as mentioned in Step 3.

This design guideline was developed based on a beam width range from 6 ft to 8 ft, a span length range from 22 ft to 40 ft, the minimum stem depth of 12 in, and a constant elastic-cracked rotational stiffness of the joint. Sensitivity study showed that either increasing the stem depth or decreasing the joint rotational stiffness decrease the transverse demands on the deck and the joint.
### 7.2.2 Further research

It should be noted that the recommendations provided in this study are based on limited experimental testing by both the researchers of this study and investigators at other universities, industry research and development labs at state and federal transportation research labs. The results of this collected research, field experience with existing cast-in-place and precast bridges and sound engineering judgment allow the researchers to believe with a reasonable degree of confidence that the construction of a bridge using the provided recommendations will lead to durable bridges that are cost effective and can be constructed using an accelerated schedule.

After the modified NEXT-D beam bridge is implemented in the state, the bridge could be monitored in the following aspects:

- **Check load distribution factors during the bridge service life using controlled loading.** Strain gauges can be placed longitudinally at the bottom of the stem at the critical cross section as identified in beam line analysis. The load distribution factors will then be compared with the simulation LDFs and the AASHTO LDFs.

- **Check curvature change of the joint using LVDTs under controlled loading.** In the 3d bridge FE model, it was assumed that all the joints had the same stiffness. This should be checked against the real data to see how much difference there is, and to determine how much influence this variation of stiffness would change the demand distribution.

- **Check rebar strain at the joint interface for fatigue performance under controlled loading.**
• Compare the performances of NEXT-D beam bridge with hollow core beam bridge under the same traffic volume. Remote monitoring can be applied for data acquisition during the bridge’s service life.
Appendices
Appendix A

Shear Key Modeling
### Targeted Stiffness Matrix (6) for a single shear key element in SAP2000

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**Units:** ksi, inches, radians.
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Appendix C

Sample Bridge Calculations for NEXT-8 40 ft
NEXT-8 Beam2 Design Check Outline:

1. Overview
2. Basic properties
3. Gross cross-section properties for a beam section including shear key:
4. Shear forces and bending moments:
   4.1 Shear forces and bending moments due to dead loads:
      4.1.1 Dead loads
      4.1.2 Unfactored shear forces and bending moments:
   4.2 Shear forces and bending moments due to live loads:
      4.2.1 Live loads
      4.2.2 Live load distribution factors for a typical interior beam
      4.2.3 Dynamic allowance
      4.2.4 Unfactored shear forces and bending moments
      4.2.5 Load combinations
5. Prestress loss
   5.1 Strand pattern:
   5.2 Gross cross section properties at the middle span:
   5.3 Prestress losses
      5.3.1 Elastic shortening
      5.3.2 Time-dependent losses using approximate method
      5.3.3 Total losses at transfer
      5.3.4 Total losses at service loads
6. Stress at limit states:
   6.1 Concrete stresses at transfer:
      6.1.1 Stress limits for concrete
      6.1.2 Stresses at transfer length section of bonded strands
   6.2 Concrete stresses at service loads after losses
      6.2.1 Stress limits for concrete
      6.2.2 Stresses at midspan or transfer length cross section
   6.3 Fatigue stress limit
7. Strength limit state
   7.1 Strain in strands
   7.2 Stress in strands
   7.3 Total force in each row of strands
   7.4 Moment contributed by each row of strand
   7.5 Moment capacity

8. Limits of reinforcement
   8.1 Maximum reinforcement
   8.2 Minimum reinforcement

9. Anchorage zone reinforcement
   10.1 Anchorage zone reinforcement
   10.2 Confinement reinforcement
1. Overview

In addition to the design of NEXT-D bridges using CONSPAN, a hand-calculation is provided here for the first interior pretensioned prestressed concrete beam of NEXT-8 beam bridge. In the calculation, the design span length is 39ft from center to center of bearings, and the total span length is 40ft. The overall width of the bridge is 48ft, and the roadway width between the interior faces of parapets is 44.83ft. An average 4-in bituminous overlay will be used for wearing surface. This example includes the load effect calculation from prestress load, dead loads, and HL-93 live loads, stress check of concrete and prestress tendon at different stages and different limit states, moment capacity check, reinforcement requirement check, and pretensioned anchorage zone check. Four design limit states are investigated, including Service I, Service III, Fatigue, and Strength I limit states. The format follows the example design of NEXTD 36 D provided by PCI bridge design manual. This hand calculation is provided as a comparison with the corresponding results provided by CONSPAN, the bridge model in which has a total span length of 39ft 10.5in.

![Figure I.1: NEXT-8 Bridge Cross Section](image)

2. Basic properties:

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<td>Specified concrete compressive strength for use in design:</td>
<td>( f'_{c} = 6.5\text{ksi} )</td>
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<td>Precast beam unit weight: ( w_c )</td>
<td>( 0.15 \text{kip/ft}^3 )</td>
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Note: the shear key weight is included in the precast beam weight, and its density is assumed to be the same as the precast beam.

Modulus of elasticity, ksi = \( 33000 \cdot K_1 \cdot w_c^{1.5} / \sqrt{f'_c} \)  
where  
Correction factor for source of aggregate: \( K_1 = 1 \)  
Unit weight of concrete: \( w_c = 150 \text{pcf} \)
For Young's modulus calculation, this unit weight is higher than what is given in LRFD Table 3.5.1-1. It is to be used for design check unless more precise information is provided.

\[ f'_{c} = \text{specified compressive strength of concrete, ksi} \]

Therefore, the modulus of elasticity for:

- Precast beam at transfer:
  \[ E_{ct} = 33000 \times K_{1} \left( \frac{w_{c}}{1000 \text{pcf}} \right)^{1.5} \sqrt[1.5]{f'_{ct}} \text{ksi} = 4.372 \times 10^{3} \text{ksi} \]

- Precast beam at service loads:
  \[ E_{c} = 33000 \times K_{1} \left( \frac{w_{c}}{1000 \text{pcf}} \right)^{1.5} \sqrt[1.5]{f'_{c}} \text{ksi} = 4.888 \times 10^{3} \text{ksi} \]

Future bituminous wearing surface (average 4in): \( w_{fws} = 140 \text{pcf} \) LRFD Table 3.5.1-1

New Jersey-type barrier on each side: \( w_{p} = 443 \text{plf} \)

Prestressing strands: 0.5-in.-dia., seven-wire, low-relaxation

Area of one strand = 0.153in²

Specified tensile strength: \( f'_{pu} = 270 \text{ksi} \)

Yield strength: \( f'_{py} = 0.9 \cdot f'_{pu} = 243 \cdot \text{ksi} \) LRFD Table 5.4.4.1-1

Stress limits for prestressing strands (pretensioning): LRFD Table 5.9.3-1

- Before transfer: \( f_{pt} \leq 0.75 f'_{pu} = 202.5 \cdot \text{ksi} \)
- At service limit state (after all losses): \( f_{pe} \leq 0.8 f'_{py} = 194.4 \cdot \text{ksi} \)

Modulus of elasticity, \( E_{p} = 28500 \text{ksi} \) LRFD Art. 5.4.4.2

Reinforcing bars:

Yield strength, \( f'_{y} = 60 \text{ksi} \)

Modulus of elasticity, \( E_{s} = 29000 \text{ksi} \)

3. Gross cross-section properties for a beam section including shear key:
Area of cross section of precast beam: \( A_g = 1147.4 \text{in}^2 \)

Overall depth of precast beam: \( h = 21 \text{in} \)

Moment of inertia about the centroid of the noncomposite precast beam: \( I_g = 37120 \text{in}^4 \)

Distance from centroid to the extreme bottom fiber of the beam: \( y_b = 13.54 \text{in} \)

Distance from centroid to the extreme top fiber of the beam: \( y_t = h - y_b = 7.46 \text{in} \)

Section modulus for extreme bottom fiber of the beam: \( S_b = \frac{I_g}{y_b} = 2.742 \times 10^3 \text{in}^3 \)

Section modulus for extreme top fiber of the beam: \( S_t = \frac{I_g}{y_t} = 4.976 \times 10^3 \text{in}^3 \)

4. Shear forces and bending moments:

4.1 Shear forces and bending moments due to dead loads:

4.1.1 Dead loads

DC = dead load of structural components and nonstructural attachments

LRFD Art. 3.3.2

Beam self weight (including shear key weight): \( w_g := w_c \cdot A_g = 1.195 \frac{\text{kip}}{\text{ft}} \)

LRFD Article 4.6.2.2.1 states that permanent loads of and on the deck (barrier and wearing surface loads) may be distributed uniformly among all the beams if the following conditions are met:

a. Width of deck is constant \( \text{OK} \)

b. Number of beams \( (N_b) \) not less than four \( N_b = 6 \)
c. Beams are parallel and approximately of the same stiffness  \textbf{OK}

d. The roadway part of the overhang, \( d_e \leq 3 \text{ft.} \)

\[
d_e := 2.5 \text{ft} - 1 \text{in} - 18 \text{in} = 0.917 \cdot \text{ft}  \quad \textbf{OK}
\]
e. Curvature is less than specified in LRFD Specifications, (curvature = 0°)  \textbf{OK}

f. Cross section of the bridge is consistent with one of the cross sections given in LRFD Table 4.6.2.2.1-1. The bridge is "sufficiently connected to act as unit" and the bridge type is (i).  \textbf{OK}

Since these criteria are satisfied, the wearing surface loads are distributed equally among the six beams.

\[
DW = \text{dead load of wearing surface per each beam:} \quad \text{LRFD Art. 3.3.2}
\]

\[
DW := \frac{(4 \text{in} \cdot w_{fws}) \cdot 44.83 \text{ft}}{6} = 0.349 \cdot \text{kip/ft}
\]

where,  \( w_{fws} = 140 \cdot \text{pcf} \)

For the barriers, their weights are decided to be distributed to only the exterior beam and the adjacent interior beam.

Barrier weight per each beam :  \( w_b := \frac{w_p}{2} = 0.221 \cdot \text{kip/ft} \)

4.1.2 Unfactored shear forces and bending moments:

Values of shear forces and bending moments for a typical interior beam under the self weight of beam (including shear key weight), barriers, and wearing surface are calculated using finite element software SAP2000, which agree with CONSPAN results. For these calculations, the span length (L) is the design span, 39ft. However, for calculations of stresses and deformations at the time prestress is transferred, the overall length of the precast member, 40ft, is used.

4.2 Shear forces and bending moments due to live loads:

4.2.1 Live loads

Design live load is HL-93, which consists of a combination of: \text{LRFD 3.6.1.2.1}

1. Design truck or design tandem with dynamic allowance

\text{LRFD 3.6.1.2.2 and 3.6.1.2.3}

2. Design lane load of 0.64klf without dynamic allowance  \text{LRFD 3.6.1.2.4}

4.2.2 Live load distribution factors for a typical interior beam

The live load bending moments and shear forces are determined by using the simplified distribution factor formulas (LRFD Art. 4.6.2.2), provided that the following conditions are met: \text{LRFD Art.4.6.2.2.1}

a. Width of deck is constant  \textbf{OK}
b. Number of beams ($N_b$) not less than four $N_b := 6$ \text{OK}

c. Beams are parallel and approximately of the same stiffness \text{OK}

d. The roadway part of the overhang, $d_e \leq 3\text{ft.}$

$$d_e := 2.5\text{ft} - 1\text{in} - 18\text{in} = 0.917\cdot\text{ft} < 3\text{ft} \text{ OK}$$

e. Curvature is less than specified in LRFD Specifications, (curvature = 0$^0$) \text{OK}

f. For a precast concrete double-tee section with shear keys without transverse post-tensioning, the bridge type is (i) \text{LRFD Table 4.6.2.2.1-1}

Number of design lanes = the integer part of the ratio $(w/12)$, where $w$ is the clear roadway width, in ft, between the curbs \text{LRFD Art. 3.6.1.1.1}

$$w := 44.83\text{ft} \quad \frac{w}{12\text{ft}} = 3.736$$

Number of design lanes: $N_L := 3$

4.2.2.1 Distribution factor for bending moments for cross section type I

a. For all limit states except fatigue limit state:

Regardless of number of loaded lanes:

$$DFM = 0.075 + \left(\frac{S}{9.5}\right)^{0.5}\left(S/L\right)^{0.2}\left(K_g/12/L/t_s^3\right)^{0.1} \text{LRFD Table 4.6.2.2b-1}$$

Provided that:

$$3.5 \leq S \leq 16 \quad S := 8\text{ft} \quad \text{OK}$$

$$4.5 \leq t_s \leq 12 \quad t_s := 8\text{in} \quad \text{OK}$$

$$20 \leq L \leq 240 \quad L := 39\text{ft} \quad \text{OK}$$

$$N_b \geq 4 \quad N_b = 6 \quad \text{OK}$$

$$10000 \leq K_g \leq 700000 \quad \text{OK} \text{ (see below)}$$

where

$DFM =$ distribution factor for moment for interior beam

$S =$ beam spacing, ft

$t_s =$ structural depth of concrete deck, in

$L =$ beam span, ft

$K_g =$ longitudinal stiffness parameter, in$^4$ = $n(l_{bs} + A_{bs}e_g^2)$

where

$n =$ modular ratio between beam and deck slab concrete
n := \frac{E_c}{E_c} = 1

I_{bs} = \text{moment of inertia of the beam} \ \text{in}^4

A_{bs} = \text{cross-sectional area of the beam, in}^2

e_g = \text{distance between the centers of gravity of the basic beam and deck, in}

LRFD Artical 4.6.2.2 is unclear on how to calculate \( K_g \) for bridges without a composite deck. In this design, both the stems and the flange are considered together as a beam.

Therefore, \[ K_g := I_g + A_g y_t^2 = 1.01 \times 10^5 \text{in}^4 \]

\[ \text{DFM} := 0.075 + \left( \frac{S}{9.5\text{ft}} \right)^{0.6} \left( \frac{S}{L} \right)^{0.2} \left( \frac{\frac{K_g}{\text{in}^4}}{12 \frac{L}{\text{ft}} \frac{t_s}{\text{in}^3}} \right)^{0.1} = 0.678 \text{lanes/beam} \]

Consapan DFM = 0.677 \ OK

For one design lane loaded:

\[ \text{DFM} := 0.06 + \left( \frac{S}{14\text{ft}} \right)^{0.4} \left( \frac{S}{L} \right)^{0.3} \left( \frac{\frac{K_g}{\text{in}^4}}{12 \frac{L}{\text{ft}} \frac{t_s}{\text{in}^3}} \right)^{0.1} = 0.516 \text{lanes/beam} \]

Consapan DFM = 0.514 \ OK

Thus, the case of two or more lanes loaded controls and \( \text{DFM} := 0.677 \text{lanes/beam} \)

b. For fatigue limit state:

The distribution factor for fatigue load should be calculated based on a single design truck without the multiple presence factor 1.2 specified in LRFD Article 3.6.1.1.2. Therefore the factor for fatigue limit state is:

\[ \text{DFM} := \frac{0.514}{1.2} = 0.428 \text{lanes/beam} \]

4.2.2.2 Distribution factor for shear force

For two or more lanes loaded:

\[ \text{DFV} = 0.2 + \left( \frac{S}{12} \right) - \left( \frac{S}{35} \right)^2 \]
Provided that:

- $3.5 \leq S \leq 16$  
  $S := 8$ ft  
  OK
- $4.5 \leq t_s \leq 12$  
  $t_s := 8$ in  
  OK
- $20 \leq L \leq 240$  
  $L := 39$ ft  
  OK
- $N_b \geq 4$  
  $N_b = 6$  
  OK

where

- $DFV =$ distribution factor for shear for interior beam
- $S =$ beam spacing, ft

Therefore, the distribution factor for shear force is:

$$DFV = 0.2 + \left( \frac{S}{12} \right) - \left( \frac{S}{35} \right)^2 = 0.814 \text{ lanes/beam}$$

Consplan DFM = 0.825  OK

For one design lane loaded:

$$DFV = 0.36 + \left( \frac{S}{25} \right) = 0.68 \text{ lanes/beam}$$

Consplan DFM = 0.681  OK

Thus, the case of two or more lanes loaded controls and $DFV = 0.825 \text{ lanes/beam}$

4.2.3 Dynamic allowance  
LRFD Tabel 3.6.2.1-1

- $IM := 15\%$ for fatigue and fracture limit states
- $IM := 33\%$ for all other limit states

where $IM =$ dynamic load allowance, applied to design truck and design tandem

4.2.4 Unfactored shear forces and bending moments

4.2.4.1 Due to truck load; $V_{LT}$ and $M_{LT}$

a. For all limit states except for fatigue limit state:

Shear force and bending moment envelops on a per-lane basis are calculated using SAP2000. The results agree with CONSPAN results and are not given here. Truck load shear forces and bending moments per beam are:

$$V_{LT} = (\text{shear force per lane})(DFV)(1+IM)$$
$$= (\text{shear force per lane})(0.825)(1+0.33)$$
$$= (\text{shear force per lane})(1.097)\text{kips}$$
MLT = (bending moment per lane)(DFV)(1+IM)
      = (bending moment per lane)(0.677)(1+0.33)
      = (bending moment per lane)(0.9)kips

b. For fatigue limit state:

Article 3.6.1.4.1 in the LRFD Specifications states that fatigue load is a single design truck which has the same axle weight used in all other limit states but with a constant spacing of 30ft between the 32-kip axles. Bending moment envelope on a per-lane basis is calculated using SAP2000.

Therefore, bending moment of fatigue truck load is:

\[ M_r = (\text{bending moment per lane})(DFM)(1+IM) \]
\[ = (\text{bending moment per lane})(0.428)(1+0.15) \]
\[ = (\text{bending moment per lane})(0.492) \text{ ft-kips} \]

4.2.4.2 Due to Design lane load; \( V_{LL} \) and \( M_{LL} \)

To obtain the maximum shear force at a section located at a distance \((x)\) from the left support under a uniformly distributed load of 0.64kips/ft, load the member to the right of the section under consideration. Therefore, the maximum shear force per lane is:

\[ V_x = \frac{0.32(L-x)^2}{L} \quad \text{for } x \leq 0.5L \]

where \( V_x \) is in Kips/lane and \( L \) and \( x \) are in ft.

The maximum bending moment at any section is also calculated using SAP2000. Lane load shear force and bending moment per typical interior beam are as follows:

\[ V_{LL} = (\text{lane load shear force})(DFV) \]
\[ = (\text{lane load shear force})(0.825) \text{ kips} \]

For all limit states except for fatigue limit state:

\[ M_{LL} = (\text{lane load bending moment})(DFM) \]
\[ = (\text{lane load bending moment})(0.677) \text{ ft-kips} \]

Note that dynamic allowance is not applied to the design lane loading.

4.2.5 Load combinations

Total factored load is taken as:

\[ Q = \sum \eta_i \gamma_i Q_i \]

where \( \eta_i \) = a load modifier relating to ductility, redundancy, and operational importance.

\( \gamma_i \) = load factors

\( Q_i \) = force effects from specified loads

LRFD Art. 1.3.2.1

LRFD Art. 3.4

LRFD Table 3.4.1-1

LRFD Art. 3.4.1-1
Investigating different limit states given in LRFD Article 3.4.1, the following limit states are applicable:

Service I: check compressive stresses in prestressed concrete components:

\[ Q = 1.00 \cdot (DC + DW) + 1.00 \cdot (LL+IM) \]

Service III: check tensile stresses in prestressed concrete components:

\[ Q = 1.00 \cdot (DC + DW) + 0.80 \cdot (LL+IM) \]  LRFD Table 3.4.1-1

Strength I: check ultimate strength: LRFD Tables 3.4.1-1 and 3.4.1-2

Maximum \( Q = 1.25 \cdot (DC)+1.50 \cdot (DW)+1.75 \cdot (LL+IM) \)

Minimum \( Q = 0.90 \cdot (DC)+0.65 \cdot (DW)+1.75 \cdot (LL+IM) \)

This load combination is the general load combination for strength limit state design. Since only critical positive moment needs to be investigated, only the first combination will be applied.

Fatigue I: check stress range in strands:

\[ Q = 1.50 \cdot (LL+IM) \]  LRFD Table 3.4.1-1

This load combination is a special load combination to check the tensile stress range in the strands due to live load and dynamic allowance.

5. Prestress loss

5.1 Strand pattern:
Strand type: 1/2-270K-LL, low relaxation strands
Quantity : 26
Pattern : straight
End pattern:

4 @ 13.5in
2 @ 6.5in
10 @ 4.5in
10 @ 2.5in

![Figure I.3: End Strand Pattern](image)

Parameters:
Strand diameter: 0.5in
Strand area: 0.153in²
Tensile strength: \( f_{pu} := 270 \text{ksi} \)
5.2 Gross cross section properties at the middle span:

The distance between center of gravity of strands and the bottom concrete fiber of the beam is:

\[
y_{bs} := \frac{10 \cdot (2.5\text{in} + 4.5\text{in}) + 2 \cdot 6.5\text{in} + 4 \cdot 13.5\text{in}}{26} = 5.269\text{in}
\]

Gross cross section at transfer:

- \(A_{ti} = \) area of gross cross section at transfer: \(A_{ti} := A_g = 1.147 \times 10^3\text{in}^2\)
- \(I_{ti} = \) moment of inertia of the gross cross section at transfer: \(I_{ti} := I_g = 3.712 \times 10^4\text{in}^4\)
- \(y_{bti} = \) distance from the centroid of the gross cross section to the extreme bottom fiber of the beam at transfer: \(y_{bti} := y_b = 13.54\text{in}\)
- \(e_{ti} = \) eccentricity of strands with respect to gross cross section at transfer:
  \[e_{ti} := y_{bti} - y_{bs} = 8.271\text{in}\]
- \(S_{bti} = \) section modulus for the extreme bottom fiber of the gross cross section at transfer:
  \[S_{bti} := \frac{I_{ti}}{y_{bti}} = 2.742 \times 10^3\text{in}^3\]
- \(S_{tti} = \) section modulus for the extreme top fiber of the gross cross section at transfer:
  \[S_{tti} := \frac{I_{ti}}{h - y_{bti}} = 4.976 \times 10^3\text{in}^3\]

Gross cross section at final time:

- \(A_{tf} = \) area of gross cross section at final time: \(A_{tf} := A_g = 1.147 \times 10^3\text{in}^2\)
- \(I_{tf} = \) moment of inertia of the gross cross section at final time: \(I_{tf} := I_g = 3.712 \times 10^4\text{in}^4\)
- \(y_{btf} = \) distance from the centroid of the gross cross section to the extreme bottom fiber of the beam at final time: \(y_{btf} := y_b = 13.54\text{in}\)
- \(e_{tf} = \) eccentricity of strands with respect to gross cross section at final time:
  \[e_{tf} := y_{btf} - y_{bs} = 8.271\text{in}\]
- \(S_{btf} = \) section modulus for the extreme bottom fiber of the gross cross section at final time:
  \[S_{btf} := \frac{I_{tf}}{y_{btf}} = 2.742 \times 10^3\text{in}^3\]
S_{ttf} = section modulus for the extreme top fiber of the gross cross section at final
time

\[ s_{ttf} = \frac{l_{tf}}{h - y_{btf}} = 4.976 \times 10^3 \text{in}^3 \]

5.3 Prestress losses

Total prestress loss:

\[ \Delta f_{pT} = \Delta f_{pES} + \Delta f_{pLT} \]

where

\( \Delta f_{pT} \) = total loss in prestressing steel stress

\( \Delta f_{pES} \) = sum of all losses or gains due to elastic shortening or extension at the time
of application of prestress and/or external loads

\( \Delta f_{pLT} \) = long-term losses due to shrinkage and creep of concrete, and relaxation of
steel after transfer. In this design, the approximate estimates of time-dependent
losses are used.

5.3.1 Elastic shortening

\[ \Delta f_{pES} = \frac{E_p}{E_{ct}} f_{cgp} \]

LRFD Eq. 5.9.5.2.3a-1

where

\( E_p \) = modulus of elasticity of prestressing strands \( E_p = 2.85 \times 10^4 \text{ksi} \)

\( E_{ct} \) = modulus of elasticity of beam concrete at transfer \( E_{ct} = 4.372 \times 10^3 \text{ksi} \)

\( f_{cgp} \) = sum of concrete stresses at the center of gravity of prestressing strands
due to prestressing force at transfer and the self weight of the member at
sections of maximum moment.

When gross section properties are used to calculate concrete stress, the
effects of losses and gains due to elastic deformations \( \Delta f_{pES} \) should be
included in calculating \( f_{cgp} \)

\[ f_{cgp} = \frac{P_{pi} - \Delta f_{pES}\left(26 \cdot 0.153\text{in}^2\right)}{A_{ti}} + \frac{P_{pi} - \Delta f_{pES}\left(26 \cdot 0.153\text{in}^2\right)}{I_{ti}} e_{ti}^2 - \frac{M_{g} e_{ti}}{I_{ti}} \]

where

\( P_{pi} \) = total prestressing force before transfer:

\[ P_{pi} := 26 \cdot 0.153\text{in}^2 \cdot 0.75 \cdot f_{pu} = 805.545 \text{kip} \]

\( e_{ti} \) = eccentricity of strands at midspan at transfer

\[ e_{ti} = 8.271 \text{in} \]
M_g should be calculated based on the overall beam length of 40 ft. Here, the M_g using the design span length of 39 is applied, which will give a larger stress in concrete.

\[ M_g = 227.24 \text{kip-ft} \]

Therefore,

\[
f_{cgp} = \frac{P_{pi}}{E_{ct}} f_{cgp} \left(26 \cdot 0.153 \text{in}^2\right) + \left[ \frac{P_{pi}}{E_{ct}} f_{cgp} \left(26 \cdot 0.153 \text{in}^2\right) \right] \frac{e_{ti}^2}{I_{ti}} - \frac{M_g \cdot e_{ti}}{I_{ti}}
\]

\[
f_{cgp} := \frac{500 \cdot A_{ti} \cdot E_{ct} \cdot P_{pi} \cdot e_{ti}^2 - 500 \cdot A_{ti} \cdot E_{ct} \cdot M_g \cdot e_{ti} + 500 \cdot E_{ct} \cdot I_{ti} \cdot P_{pi}}{1989.0 \cdot A_{ti} \cdot E_{ct} \cdot e_{ti}^2 \cdot \text{in}^2 + 1989.0 \cdot E_{ct} \cdot I_{ti} \cdot \text{in}^2 + 500 \cdot A_{ti} \cdot E_{ct} \cdot I_{ti}} = 1.475 \times 10^3 \text{ psi}
\]

CONSPAN : \( f_{cgp} = 1.474 \text{ksi} \)

\[
\Delta f_{pES} := \frac{E_p}{E_{ct}} f_{cgp} = 9.617 \cdot \text{ksi} \quad \text{CONSPAN : } f_{pES} = 9.61 \text{ksi}
\]

### 5.3.2 Time-dependent losses using approximate method

The long-term prestress loss, \( \Delta f_{pLT} \), due to creep of concrete, shrinkage of concrete, and relaxation of steel shall be estimated using the following formula:

\[
\Delta f_{pLT} = 10 \cdot \frac{f_{pi} \cdot A_{ps}}{A_g} \cdot \gamma_h \cdot \gamma_{st} + 12 \cdot \gamma_h \cdot \gamma_{st} + \Delta f_{pR}
\]

LRFD Eq. 5.9.5.3-1

with the first term corresponds to creep losses, the second term to shrinkage losses, and the third to relaxation losses.

where

- \( f_{pi} \) = prestressing steel stress immediately prior to transfer (ksi)

\[ f_{pi} := 0.75 \cdot f_{pu} = 202.5 \text{ ksi} \]

- \( A_{ps} \) = area of prestressing steel (in\(^2\))

\[ A_{ps} := 26 \cdot 0.153 \text{in}^2 = 3.978 \text{in}^2 \]

- \( \gamma_h \) = correction factor for relative humidity of the ambient air

\[ \gamma_h = 1.7 - 0.01 \cdot H \]

in which,

- \( H \) = the average annual ambient relative humidity (%) \( H := 75 \)

Therefore \( \gamma_h := 1.7 - 0.01 \cdot H = 0.95 \)
\[ \gamma_{st} = \text{correction factor for specified concrete strength at time of prestress transfer to the concrete member} \]

\[ \gamma_{st} := \frac{5}{1 + \frac{f'_{ct}}{\text{ksi}}} = 0.806 \]

\[ \Delta f_{pR} = \text{an estimate of relaxation loss taken as 2.4ksi for low relaxation strand.} \quad \Delta f_{pR} := 2.4\text{ksi} \]

Therefore

\[ \Delta f_{pLT} := 10.0 \frac{f_{pi} A_{ps}}{A_g} \cdot \gamma_h \cdot \gamma_{st} + 12.0\text{ksi} \cdot \gamma_h \cdot \gamma_{st} + \Delta f_{pR} = 16.972\text{-ksi} \]

CONSPAN : \( f_{pLT} = 16.97\text{ksi} \)

5.3.3 Total losses at transfer

AASHTO LRFD C5.9.5.2.3a and C5.9.5.3 indicate that the losses or gains due to elastic deformation must be included in determining the total prestress losses and the effective stress in prestressing strands.

\[ \Delta f_{pL} := \Delta f_{pES} = 9.617\text{-ksi} \]

Effective stress in tendons immediately after transfer, \( f_{pL} := f_{pL} - \Delta f_{pL} = 192.883\text{-ksi} \)

Total prestressing force after transfer, \( P_{pt} := f_{pt} A_{ps} = 767.29\text{-kip} \)

Initial loss, % = (Total losses at transfer) / (\( f_{pL} \)) = \( \frac{\Delta f_{pL}}{f_{pL}} \cdot 100 = 4.749 \)

When determining the concrete stress using gross cross section properties, the strand force is that at transfer:

The total prestressing force at transfer, \( P_{pt} = 767.29\text{-kip} \)

5.3.4 Total losses at service loads

Total loss due to elastic shortening at transfer and long-term losses (Service III) is:

\[ \Delta f_{pT} := \Delta f_{pES} + \Delta f_{pLT} = 26.589\text{-ksi} \]

The elastic gain due to superimposed dead load, and live load is:

\[ \left( M_p + M_{ws} \right) + 0.8 \left( M_{LL} + M_{LT} \right) \right) \frac{E_p}{E_c} \]

\[ = (716.7\text{kip-ft} - 227.24\text{kip-ft}) \left( \frac{E_{tf}}{E_c} \right) = 7.631\text{-ksi} \]

The effective stress in strands after all losses and gains:
\[ f_{pe} := f_{pi} - \Delta f_{pT} + 7.631 \text{ksi} = 183.542 \text{ksi} \quad \text{CONSPAN: } f_{pe} = 183 \text{ksi} \]

Check prestressing stress limit at service limit state: \( \text{LRFD Table 5.9.3-1} \)
\[
0.8f_{py} = 194.4 \text{ksi} > f_{pe} = 183.542 \text{ksi} \quad \text{OK}
\]

Force per strand after all losses and gains = \(f_{pe} \cdot 0.153 \text{in}^2 = 28.082 \text{kip}\)

Therefore, the total prestressing force after all losses = \(28.016 \text{kip} \times 26 = 728.416 \text{kip}\)

Final loss percentage = \[
\frac{f_{pi} - f_{pe}}{f_{pi}} \cdot 100 = 9.362
\]

CONSPAN considers an adjustment of prestress loss due to superimposed dead load, and live load. \(\text{CONSPAN : } =9.63\%\)

Force per strand with elastic loss and total time-dependent losses = \(\left(f_{pi} - \Delta f_{pT}\right) \cdot 0.153 \text{in}^2 = 26.914 \text{kip}\)

Total prestressing force, \(P_{pe} := 26.914 \text{kip} \times 26 = 699.764 \text{kip}\)

CONSPAN considers the effect due to superimposed dead load, and live load. \(\text{CONSPAN : } P_{pe} := 183 \text{ksi} \cdot 0.153 \text{in}^2 \cdot 26 = 727.974 \text{kip}\)

6. Stress at limit states:

6.1 Concrete stresses at transfer:

Because the gross cross section is used, the total prestressing force at transfer, \(P_{pt} = 767.29 \text{kip}\)

6.1.1 Stress limits for concrete \(\text{LRFD Art. 5.9.4}\)

Compression:
\[0.6f'_{ct} = 3.12 \text{ksi}\]

where \(f'_{ct} = \text{concrete strength at transfer} = 5.2 \text{ksi}\)

Tension:

In areas other than the precompressed tensile zone and without bonded auxiliary reinforcement

\[-0.0948 \cdot \sqrt[\text{ksi}]{f'_{ct}} = -0.216 \text{ksi} \leq -0.2 \text{ksi}\]

Therefore, \(-0.2 \text{ksi} \quad \text{(controls)}\)
with bonded auxiliary reinforcement sufficient to resist the tensile force in the concrete computed assuming an uncracked section, where reinforcement is proportioned using a stress of 0.5f_y, not to exceed 30ksi.

\[-0.24 \sqrt{\frac{f_{ct}}{ksi}} = -0.547 \text{ksi}\]

6.1.2 Stresses at transfer length section of bonded strands

Stresses at this location are checked since this cross section is most critical as reflected from CONSPAN result summary.

Transfer length = 60 (strand diameter) = 60 \cdot 0.5\text{in} = 2.5\text{ft}

\[M_g := \frac{w_g \cdot 40\text{ft}^2 \cdot 2.5\text{ft} - \frac{w_g \cdot (2.5\text{ft})^2}{2}}{2} = 56.025\text{-kip-ft} \quad \text{where, } w_g = 1.195\text{-klf}\]

Compute concrete stress in the top of beam:

\[f_t := \frac{P_{pt}}{A_{ti}} - \frac{P_{pt} \cdot c_{ti}}{s_{tti}} + \frac{M_g}{s_{tti}} = -0.472\text{-ksi} \quad \text{CONSPAN } f_t = -0.472\text{ksi}\]

Tensile stress limit for concrete with bonded reinforcement: -0.547ksi  \text{ OK}

Bonded auxiliary reinforcement must be provided in the top of the beam.

In order to determine the required bonded reinforcement area, the tensile zone needs to be determined first by using the stresses at extreme fibers.

Compute concrete stress in bottom of beam:

\[f_b := \frac{P_{pt}}{A_{ti}} + \frac{P_{pt} \cdot c_{ti}}{s_{tti}} - \frac{M_g}{s_{tti}} = 2.738\text{-ksi} \quad \text{CONSPAN } f_b = 2.739\text{ksi}\]

Compressive stress limit for concrete: 3.12ksi  \text{ OK}

The depth of the tensile zone x is:  \text{ LRFD C5.9.4.1.2}

\[- \frac{f_t}{x} = \frac{f_b}{h - x}\]

\[x := \frac{f_t \cdot h}{f_b - f_t} = 3.085\text{-in} \leq 8\text{in}\]

The tensile force in the concrete T is:

\[T = \frac{f_t}{2} \cdot b \cdot x\]

where  \(b\) is the width of the beam at top  \(b := 96\text{in}\)

Therefore  \(T := \frac{-f_t}{2} \cdot b \cdot x = 69.825\text{-kip}\)
The required area of bonded reinforcement is: 
\[ A_{\text{req}} = \frac{T}{f_s} \]

where \( f_s := 0.5 \cdot f_y = 30 \text{ ksi} \)

Therefore 
\[ A_{\text{req}} := \frac{T}{f_s \cdot b} = 0.291 \frac{\text{in}^2}{\text{ft}} \]

Use #4 @ 7in within the tensile zone \( A_s := 0.34 \frac{\text{in}^2}{\text{ft}} > A_{\text{req}} \quad \text{OK} \)

6.2 Concrete stresses at service loads after losses LRFD Art. 5.9.4.2

With elastic loss and total time-dependent losses, \( P_{pe} = 727.974 \text{kip} \)

6.2.1 Stress limits for concrete

Compression:
Due to the sum of effective prestress, permanent loads, and transient loads (i.e. all dead loads and live loads), for load combination Service I (final 1 in CONSPAN)

for precast beams: \( 0.6 \phi_w f'_c \)

where \( \phi_w \) is a reduction factor, it shall be taken to be equal to 1.0 when the web and flange slenderness ratios, calculated according to Article 5.7.4.7.1 are not greater than 15. This condition is satisfied for the NEXT-8 beam. Therefore \( \phi_w := 1 \quad 0.6 \phi_w f'_c = 3.9 \text{ksi} \)

Due to sum of effective prestress and permanent loads (i.e. beam self weight including shear key, weight of future wearing surface, and weight of barriers), for load combination Service I (final II in CONSPAN):

for precast beams: \( 0.45 f'_c = 2.925 \text{ksi} \)

Tension: LRFD Table 5.9.4.2.2-1

For components with bonded prestressing tendons or reinforcement that are subjected to not worse than moderate corrosion conditions

for load combination Service III:

for precast beams \( -0.19 \frac{f_c}{\sqrt{\text{ksi}}} = -0.484 \text{ksi} \)

6.2.2 Stresses at midspan or transfer length cross section

6.2.2.1 Concrete compressive stress at top fiber of the beam at middle span

To check top compressive stresses, middle span is the critical cross section. Two cases are considered:
1. Under permanent loads and prestress, load combination service I
\[ f_{tg} := \frac{P_{pe}}{A_{tf}} - \frac{P_{pe} e_{tf}}{s_{tf}} + \frac{227.24 \text{kip} \cdot \text{ft} + 42.02 \text{kip} \cdot \text{ft} + 66.35 \text{kip} \cdot \text{ft}}{s_{tf}} = 0.234 \text{ ksi} \]

Compressive stress limit: 2.925 ksi OK CONSPAN \( f_{tg} = 0.234 \text{ ksi} \)

2. Under prestress, permanent and transient loads, load combination Service I:
\[ f_{tg} := \frac{P_{pe}}{A_{tf}} - \frac{P_{pe} e_{tf}}{s_{tf}} + \frac{812 \text{kip} \cdot \text{ft}}{s_{tf}} = 1.383 \text{ ksi} \]

Compressive stress limit: 3.9 ksi OK

6.2.2.2 Concrete compressive stress at bottom fiber of the beam at transfer cross section
Only the case under permanent loads and prestress need to be considered. And the transfer cross section is critical.

Load combination Service I
\[ f_{tg} := \frac{P_{pe}}{A_{tf}} + \frac{P_{pe} e_{tf}}{s_{btf}} - \frac{42.43 \text{kip} \cdot \text{ft} + 7.846 \text{kip} \cdot \text{ft} + 12.388 \text{kip} \cdot \text{ft}}{s_{btf}} = 2.556 \text{ ksi} \]

Compressive stress limit: 2.925 ksi OK CONSPAN \( f_{tg} = 2.536 \text{ ksi} \)

6.2.2.3 Concrete tensile stress in bottom of beam at middle span, load combination Service III
\[ f_{b} := \frac{P_{pe}}{A_{tf}} + \frac{P_{pe} e_{tf}}{s_{btf}} - \frac{716.7 \text{kip} \cdot \text{ft}}{s_{btf}} = -0.306 \text{ ksi} \]

Tensile stress limit: 0.484 ksi OK

6.3 Fatigue stress limit

LRFD Article 5.5.3.1 states that in fully prestressed components other than segmentally constructed bridges, the compressive stress due to Fatigue I load combination and one half the sum of effective prestress and permanent loads shall not exceed 0.4\( f'_c \) after losses.

At middle span, the unfactored fatigue bending moment is 247kip*ft. Therefore, stress at the top fiber of the beam is:
\[ \frac{247 \text{kip} \cdot \text{ft}}{s_{ttf}} + \frac{1}{2} \left( \frac{P_{pe}}{A_{tf}} - \frac{P_{pe} e_{tf}}{s_{ttf}} + \frac{227.24 \text{kip} \cdot \text{ft} + 42.02 \text{kip} \cdot \text{ft} + 66.35 \text{kip} \cdot \text{ft}}{s_{ttf}} \right) = 0.713 \text{ ksi} \]

CONSPAN 0.713 ksi
At the transfer length cross section, the unfactored fatigue bending moment is 51.2 kip*ft. Therefore, stress at the bottom fiber of the beam is:

\[ \frac{-51.2 \text{kip-ft}}{s_{btf}} + \frac{1}{2} \left( \frac{P_{pe}}{A_{tf}} + \frac{P_{pe} \cdot c_{tf}}{s_{btf}} - \frac{42.43 \text{kip-ft} + 7.846 \text{kip-ft} + 12.388 \text{kip-ft}}{s_{btf}} \right) = 1.054 \text{ksi} \]

\[ < 0.4f'_c = 2.6 \text{ksi} \quad \text{OK} \]

7. Strength limit state

Total ultimate bending moment for strength I is:

\[ M_u = 1.25(DC) + 1.5(DW) + 1.75(LL + IM) \]

From CONSPAN or SAP2000, this ultimate bending moment is

\[ M_u := 1269.8 \text{kip-ft} \]

Strain compatibility method is used without consideration of the contribution of any rebar, which is what CONSPAN does. The procedure here is the same as that used in CONSPAN. The whole procedure is iterative. A loop can be used to locate the neutral axis c. By trial and error, c is as below.

\[ c := 2.76 \text{in} \]

\[ \beta_1 = \text{stress factor of compression block} \]
\[ = 0.85 \text{ for } f'c \leq 4 \text{ksi} \]
\[ = 0.85 - 0.05 (f'c - 4) \geq 0.65 \text{ for } f'c > 4 \text{ksi} \]

\[ \beta_1 := 0.85 - 0.05 \left( \frac{f_c}{\text{ksi}} - 4 \right) = 0.725 \]

where, \( f'_c = \text{specified compressive strength of concrete} \quad f'_c = 6.5 \text{ksi} \)

\[ a = \text{depth of the equivalent stress block, in.} \]
\[ a := \beta_1 \cdot c = 2.001 \text{in} \quad \text{CONSPAN: } a = 2 \text{in} \]

7.1 Strain in strands

7.1.1 After all the prestress losses, assume each strands has the same stress, when the beam is under the action of prestressing force alone. At this time, the strain in each strand is:

\[ \varepsilon_{psln} := \frac{f_{pe}}{E_p} \]

where

\[ f_{pe} = \text{stress in strands after elastic and long-term loss due to creep, shrinkage, and steel relaxation.} \quad f_{pe} := f_{pi} - \Delta f_{pt} = 175.911 \text{ksi} \]
\[ \varepsilon_{ps1n} = \frac{P_{pe}}{E_p} = 6.172 \times 10^{-3} \]

7.1.2 As the beam concrete is totally decompressed (strain is zero), the increment in strand strain is equal to the strain in surrounding concrete with \( P_e \) acting alone. The strain in strand now is:

\[ \varepsilon_{ps2n} = \frac{P_{pe}}{E_c \cdot A_{tf}} + \frac{P_{pe \cdot tf}^2}{E_c \cdot l_{tf}} \]

where

\( P_e \) = the total prestressing force in the beam after long-term loss

\( P_{pe} = 7.28 \times 10^5 \text{ lbf} \)

Therefore,

\[ \varepsilon_{ps2n} = \frac{P_{pe}}{E_c \cdot A_{tf}} + \frac{P_{pe \cdot tf}^2}{E_c \cdot l_{tf}} = 4.043 \times 10^{-4} \]

7.1.3 The third component of strains in each row of strand is:

Row 1: \[ \varepsilon_{ps31} := 0.003 \cdot \frac{(h - 2.5\text{in} - c)}{c} = 0.017 \]

Row 2: \[ \varepsilon_{ps32} := 0.003 \cdot \frac{(h - 4.5\text{in} - c)}{c} = 0.015 \]

Row 3: \[ \varepsilon_{ps33} := 0.003 \cdot \frac{(h - 6.5\text{in} - c)}{c} = 0.013 \]

Row 4: \[ \varepsilon_{ps34} := 0.003 \cdot \frac{(h - 13.5\text{in} - c)}{c} = 5.152 \times 10^{-3} \]

7.1.4 The total strain in each row of strand is:

Row 1: \[ \varepsilon_{ps1} = \varepsilon_{ps1n} + \varepsilon_{ps2n} + \varepsilon_{ps31} = 0.024 \]

Row 2: \[ \varepsilon_{ps2} = \varepsilon_{ps1n} + \varepsilon_{ps2n} + \varepsilon_{ps32} = 0.022 \]

Row 3: \[ \varepsilon_{ps3} = \varepsilon_{ps1n} + \varepsilon_{ps2n} + \varepsilon_{ps33} = 0.019 \]

Row 4: \[ \varepsilon_{ps4} = \varepsilon_{ps1n} + \varepsilon_{ps2n} + \varepsilon_{ps34} = 0.012 \]

7.2 The stress in each row of strand is:
Row 1: \( f_{ps1} := f_{pu} - \frac{0.04}{\varepsilon_{ps1} - 0.007} \text{ksi} = 267.603 \text{ksi} \)

Row 2: \( f_{ps2} := f_{pu} - \frac{0.04}{\varepsilon_{ps2} - 0.007} \text{ksi} = 267.244 \text{ksi} \)

Row 3: \( f_{ps3} := f_{pu} - \frac{0.04}{\varepsilon_{ps3} - 0.007} \text{ksi} = 266.758 \text{ksi} \)

Row 4: \( f_{ps4} := f_{pu} - \frac{0.04}{\varepsilon_{ps4} - 0.007} \text{ksi} = 261.541 \text{ksi} \)

7.3 Total force in each row of strand is:

Row 1: \( F_{p1} := f_{ps1} \cdot 10 \cdot 0.153 \text{in}^2 = 409.432 \text{kip} \)
Row 2: \( F_{p2} := f_{ps2} \cdot 10 \cdot 0.153 \text{in}^2 = 408.883 \text{kip} \)
Row 3: \( F_{p3} := f_{ps3} \cdot 2 \cdot 0.153 \text{in}^2 = 81.628 \text{kip} \)
Row 4: \( F_{p4} := f_{ps4} \cdot 4 \cdot 0.153 \text{in}^2 = 160.063 \text{kip} \)

Check force balance

\( F_p := F_{p1} + F_{p2} + F_{p3} + F_{p4} = 1.06 \times 10^3 \text{kip} \)

\( F_c := 0.85 \cdot f'_c \cdot a \cdot b \)

where

\( f'_c = \text{specified compressive strength of concrete} \quad f'_c = 6.5 \text{ksi} \)
\( b = \text{width of compression flange} \quad b := 96 \text{in} \)

Therefore

\( F_c := 0.85 \cdot f'_c \cdot a \cdot b = 1.061 \times 10^3 \text{kip} \) close to \( F_p \quad \text{OK} \)

7.4 Moment contributed by each row of strand is:

Row 1: \( M_{p1} := F_{p1} \left( h - 2.5\text{in} - \frac{a}{2} \right) = 597.071 \text{kip} \cdot \text{ft} \)
Row 2: \( M_{p2} := F_{p2} \left( h - 4.5\text{in} - \frac{a}{2} \right) = 528.123 \text{kip} \cdot \text{ft} \)
Row 3: \( M_{p3} := F_{p3} \left( h - 6.5\text{in} - \frac{a}{2} \right) = 91.828 \text{kip} \cdot \text{ft} \)
Row 4: \[ M_{p4} := F_{p4} \left( h - 13.5\text{in} - \frac{a}{2} \right) = 86.694\text{-kip-ft} \]

7.5 Moment capacity

\[ M_n := M_{p1} + M_{p2} + M_{p3} + M_{p4} = 1.304 \times 10^3\text{-kip-ft} \]

**CONSPAN:** \( M_n = 1304.2 \text{kip-ft} \)

Factored flexural resistance:

\[ M_r = \phi \cdot M_n \quad \text{LRFD Eq. 5.7.3.2.1-1} \]

where

\( \phi = \) resistance factor \quad \text{LRFD Art. 5.5.4.2.1} \\
\( \phi_1 = 1 \), tension controlled prestressed concrete sections

\[ M_r := \phi \cdot M_n = 1.304 \times 10^3\text{-kip-ft} \quad \Rightarrow \quad M_u = 1.27 \times 10^3\text{-kip-ft} \quad \text{OK} \]

8. Limits of reinforcement

8.1 Maximum reinforcement

The check of maximum reinforcement limits was removed from the LRFD Specifications in 2005. \quad \text{LRFD Art. 5.7.3.3.1}

8.2 Minimum reinforcement

At any section of a noncompression-controlled flexural component, the amount of prestressed and nonprestressed tensile reinforcement shall be adequate to develop a factored flexural resistance, \( M_r \), at least equal to the lesser of:

1. 1.33 times the factored moment required by the applicable strength load combination specified in Table 3.4.1-1 (Strength I); and

2. \[ M_{cr} = \gamma_3 \left( \gamma_1 f_r + \gamma_2 f_{cpe} \right) s_{bf} \quad \text{LRFD Eq. 5.7.3.3.2-1} \]

The above equation is a simplified form of LRFD Equation 5.7.3.3.2-1 because no composite section exists, therefore the composite and noncomposite section modulus are the same.

where,

\( f_r = \) modulus of rupture of concrete \quad \text{LRFD Art 5.4.2.6} \\
\[ f_r := 0.37 \sqrt{\frac{f_c}{\text{ksi}}} = 0.943\text{-ksi} \]

\( f_{cpe} = \) compressive stress in concrete due to effective prestress forces only (after allowance for all prestress losses) at extreme fiber of section where tensile stress is caused by externally applied loads (ksi) (bottom fiber)
\[
\text{fcpe} := \frac{P_{pe}}{A_{tf}} + \frac{P_{pe}^\text{etf}}{s_{btf}} = 2.831 \text{ ksi}
\]

\[s_{btf} = \text{section modulus for the extreme fiber of the section where tensile stress is caused by externally applied loads (in}^3\text{)} \quad s_{btf} = 2.742 \times 10^3 \text{ in}^3\]

\[\gamma_1 = \text{flexural cracking variability factor} \]
\[\gamma_1 := 1.2 \quad \text{for precast segmental structures}\]
\[\gamma_2 = \text{prestress variability factor} \]
\[\gamma_2 := 1.1 \quad \text{for bonded tendons}\]
\[\gamma_3 = \text{ratio of specified minimum yield strength to ultimate tensile strength of the reinforcement} \]
\[\gamma_3 := 1 \quad \text{for prestressed concrete structures}\]

\[M_{cr} := \gamma_3 (\gamma_1 f_r + \gamma_2 f_{cpe}) s_{btf} = 969.969 \text{ kip} \cdot \text{ft}\]

The above \(M_{cr}\) applies to any cross section in between the two transfer length cross sections. At other cross sections, \(M_{cr}\) is smaller due to the smaller effective prestress \(P_{pe}\).

The factored moment required by strength I load combination at the middle span is:

\[M_u = 1.27 \times 10^3 \text{ kip} \cdot \text{ft}\]

Thus, \(1.33 \cdot M_u = 1.689 \times 10^3 \text{ kip} \cdot \text{ft} > M_{cr} = 969.969 \text{ kip} \cdot \text{ft}\)

Therefore \(M_{cr}\) requirement controls.

\[M_f = 1.304 \times 10^3 \text{ kip} \cdot \text{ft} > M_{cr} = 969.969 \text{ kip} \cdot \text{ft} \quad \text{OK}\]

Note: the LRFD specifications requires that this criterion be met at every section. At cross sections within the development length of prestress strands, the moment capacity is reduced. But those cross sections are not checked here.

9. Anchorage zone reinforcement \(\text{LRFD Art. 5.10.10}\)

Design of the anchorage zone reinforcement is computed using the force in the strands just prior to transfer.

\[P_{pi} = 805.545 \text{ kip}\]

The splitting resistance of pretensioned anchorage zones provided by reinforcement in the ends of pretensioned beams shall be taken as:
Pr := \frac{f_s A_s}{0.04 \cdot P_{pi}} \geq 32.222 \text{kip}

where

A_s = \text{total area of vertical reinforcement located within a distance of } h/4 \text{ from the end of the beam, in}^2

f_s = \text{stress in steel, but not taken greater than } 20\text{ksi}

A_s := \frac{0.04 \cdot P_{pi}}{20\text{ksi}} = 1.611 \text{in}^2

At least 1.611 \text{in}^2 \text{ of vertical transverse reinforcement should be provided within a distance of } (h/4 = 21\text{in}/4 = 5.25\text{in}) \text{ from the end of the beam.}

Use 3 No.4 four-leg bars at 1.5\text{in} \text{ spacing starting at } 2.25\text{in} \text{ from the end of the beam}

The provided \[A_s := 3 \cdot 4 \cdot 0.2\text{in}^2 = 2.4\text{in}^2 > 1.611\text{in}^2 \text{ OK}\]
Appendix D

Deck Design for NEXT-D Beam
NEXT-6---Deck Design Outline:

1. Deck properties:

2. 3d finite element method----Loads and load effects:
   2.1 Dead load:
   2.2 Live load
   2.3 Moment distribution
   2.4 Distribution strip width:
   2.5 Load combination
   2.6 Design demands for a 1ft strip:

3. Demands based on 1d AASHTO FEM:

4. 40ft bridge design---- #4 @ 7in
   4.1 Development length
   4.2. Limits of reinforcement
      4.2.1 Maximum reinforcement
      4.2.2 Minimum reinforcement
   4.3. Distribution reinforcement
   4.4. Shrinkage and temperature reinforcement
   4.5. Control of cracking
      4.5.1 Check of the positive moment reinforcement
      4.5.2 Check of the negative moment reinforcement

5. 30ft bridge design using #4@10in
   5.1 Development length
   5.2. Minimum reinforcement
      5.2.1 Positive moment
      5.2.2 Negative moment
   5.3. Distribution reinforcement
   5.4. Shrinkage and temperature reinforcement
   5.5. Control of cracking
      5.5.1 Check of the positive moment reinforcement
      5.5.2 Check of the negative moment reinforcement

6. 22 ft bridge design using #4@10in
   6.1 Development length
   6.2. Distribution reinforcement
   6.3. Shrinkage and temperature reinforcement
7. Design summary
   7.1 Reinforcement configuration
   7.2 Criteria check
       7.2.1 Demand VS Capacity
       7.2.2 Minimum reinforcement check
       7.2.3 Cracking control check
1. Deck properties:

Cross section: NEXT-6

Effective width of an interior beam (including shear key): \( b_{6\text{int}} := 6\text{ft} \)

Area of an interior beam: \( A_{6\text{int}} := 955.4\text{in}^2 \)

Effective width of an exterior beam (including shear key): \( b_{6\text{ext}} := 7\text{ft} \)

Area of an exterior beam: \( A_{6\text{ext}} := 1051.4\text{in}^2 \)

Structural deck depth of the deck: \( h := 8\text{in} \)

Moment of inertia considered: \( I := \frac{1}{12}\cdot1\text{ft}^3\cdot h^3 = 512\cdot\text{in}^4 \)

Section modulus: \( S_r := \frac{I}{h} \cdot 2 = 128\cdot\text{in}^3 \)

Deck top cover: \( \text{Cover}_t := 2.5\text{in} \) AASHTO LRFD Table 5.12.3-1

Deck bottom cover: \( \text{Cover}_b := 1.0\text{in} \) AASHTO LRFD Table 5.12.3-1

Reinforced concrete density: \( w_c := 150\text{pcf} \)

Concrete compressive strength (final): \( f'_c := 6.5\text{ksi} \)

Rebar Young's modulus: \( E_s := 29000\text{ksi} \)

Reinforcement strength: \( f_y := 60\text{ksi} \)

Bituminous wearing surface: \( w_{\text{fws}} := 140\text{pcf} \) AASHTO LRFD Table 3.5.1-1

2. 3d finite element method—Loads and load effects:

2.1 Dead load:

DC:

parapet self weight: \( w_p := 443\text{plf} \)

Parapet self weight is uniformly distributed to the outer two beams. Therefore for each beam mentioned above,

\[
\frac{w_p}{b_{6\text{int}} + b_{6\text{ext}}} = 0.034\text{kip/ft}^2 \quad \text{where} \quad b_{6\text{int}} = 6\text{-ft} \quad b_{6\text{ext}} = 7\text{-ft}
\]
Beam self weight:

Exterior beam:

$$w_{g\_ext} = \frac{w_c \cdot A_{6\_ext}}{b_{6\_ext}} = 0.156 \text{kip/ft}^2 \text{ where, } A_{6\_ext} = 1.051 \times 10^3 \text{in}^2 \text{, } w_c = 150 \text{pcf}$$

Interior beam:

$$w_{g\_int} = \frac{w_c \cdot A_{6\_int}}{b_{6\_int}} = 0.166 \text{kip/ft}^2 \text{ where, } A_{6\_int} = 955.4 \text{in}^2$$

DW:

Future wearing surface (4in on average):

$$DW = (4\text{in} \cdot w_{f\_w}) = 0.047 \text{kip/ft}^2 \text{ where, } w_{f\_w} = 140 \text{pcf}$$

The load from future wearing surface is distributed from face to face of the two parapets.

In the finite element model, the above area loads are distributed to the shell element of the slab. For the frame element of the shear key, according to the calculation in NEXT-8, the load exerted on the shear key has little influence on the demands, therefore load is not applied on the shear key for NEXT-6.

2.2 Live load

LRFD Art 3.6.1.3.3 specifies that when the refined methods are used to analyze decks, if the slab spans primarily in the transverse direction, only the axles of the design truck of Article 3.6.1.2.2 or design tandem of Article 3.6.1.2.3 shall be applied to the deck slab.

In order to obtain the most critical demand, a single design tandem specified in LRFD Art 3.6.1.2.3 is applied to the finite element model, moving across the bridge model transversely and longitudinally. The position of the tandem follows that specified in LRFD Art. 3.6.1.3: the design tandem shall be positioned transversely such that the center of any wheel load is not closer than: For the design of components other than deck overhang—2 ft from the edge of the design lane. In this case, the axle load is positioned no closer than 2ft from the face of the parapet.

2.3 Moment distribution

The total unfactored transverse moment demands on the deck for the 40ft span bridge resulting from the dead loads and live loads are displayed as follows, in which the black lines represent the location of the shear key:
2.4 Distribution strip width:

The distribution width for the design tandem is 10ft
2.5 Load combination LRFD Table 3.4.1-1

strength I limit state:
Maximum $Q = 1.25(DC)+1.50(DW)+1.75(LL+IM)$
Minimum $Q = 0.90(DC)+0.65(DW)+1.75(LL+IM)$

service I limit state:
$Q = 1(DC)+1(DW)+1(LL+IM)$

where

$LL = \text{live load effect including multiple presence factor 1.2}$

$IM = \text{dynamic load allowance percentage}$

$IM := 1.33 \text{ LRFD Table 3.6.2.1-1}$

2.6 Design demands for a 1ft strip:

The dead load effect is divided by the total span length to get the normalized demand on a 1ft strip. And the live load effect is divided by the strip width 10ft to get the normalized demand on the 1ft strip. The resulting normalized transverse moment demands combined together for Strength I and Service I limit states for the 40ft span are displayed below. Positive moment means the bottom deck fiber is in tension.

![Figure F.17: Demand distribution in Strength I limit state](image-url)
A summary of the demands on a 1ft strip for the 22ft, 30ft, and 40ft spans are listed as follows:

<table>
<thead>
<tr>
<th>Location</th>
<th>22 ft</th>
<th>30 ft</th>
<th>40 ft</th>
</tr>
</thead>
<tbody>
<tr>
<td>M+ (kip*ft/ft)</td>
<td>6.15</td>
<td>8.85</td>
<td>12.70</td>
</tr>
<tr>
<td>M+ (kip*ft/ft)</td>
<td>4.35</td>
<td>7.37</td>
<td>11.84</td>
</tr>
<tr>
<td>M- (kip*ft/ft)</td>
<td>-0.89</td>
<td>-1.55</td>
<td>-2.61</td>
</tr>
<tr>
<td>M- (kip*ft/ft)</td>
<td>-0.98</td>
<td>-1.29</td>
<td>-1.41</td>
</tr>
</tbody>
</table>
Table 2

<table>
<thead>
<tr>
<th>Location</th>
<th>Service I limit state</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>22 ft</td>
</tr>
<tr>
<td>M+ (kip*ft/ft) Span w/o key</td>
<td>3.59</td>
</tr>
<tr>
<td>M+ (kip*ft/ft) Span w/ key</td>
<td>2.51</td>
</tr>
<tr>
<td>M- (kip*ft/ft) Exterior beam</td>
<td>-0.45</td>
</tr>
<tr>
<td>M- (kip*ft/ft) Interior beam</td>
<td>-0.54</td>
</tr>
</tbody>
</table>

Notice that in the deck span that has no shear key, the reinforcement is doubled by the development length, which will be considered in checking the moment capacity and cracking control.

3. Demands based on 1d AASHTO FEM:

The final design demands are calculated based on the results from 1d AASHTO FEM using a finite element software SAP2000. In the FEM, the deck (including shear key) is modeled as a continuous beam using frame elements and the stems are modeled as rigid supports. Refer to the deck design guideline in Chapter 9.2.1 for details.

From the 1d AASHTO FEM, the maximum positive moment and critical negative moment (unfactored) resulting from a single 32kip axle of the design truck as specified in LRFD Article 3.6.1.2.2 are:

\[ M_{p\_total} := 123.09 \text{kip}\cdot\text{in} \]
\[ M_{n\_total} := -51.80 \text{kip}\cdot\text{in} \]

The strip widths for positive and negative moment for cast-in-place deck without stay-in-place concrete formwork are: LRFD Table 4.6.2.1.3-1

\[ M_{p\_width} = 26 + 6.6\cdot s_{\text{design}} \]
\[ M_{n\_width} = 48 + 3\cdot s_{\text{design}} \]

where,

\[ s_{\text{design}} = \text{average stem spacing (ft)} \]

\[ s_{\text{design}} := 3 \]

Therefore,

\[ M_{p\_width} := (26 + 6.6\cdot s_{\text{design}})\text{in} = 45.8\text{in} \]
\[ M_{n\_width} := (48 + 3\cdot s_{\text{design}})\text{in} = 57\text{in} \]

The normalized demands for a 1ft load strip are:

\[ M_{n\_AASHTO} := \frac{M_{n\_total}}{M_{n\_width}} = -0.909 \frac{\text{kip}\cdot\text{ft}}{\text{ft}} \]
Based on the formula proposed, the design demands from the live load are:

Positive design moment:

For span w/o key: \[ M_{\text{positive}} = M_{\text{p,AASHTO}} \left[ 0.77 + 0.0027 \left( l_{\text{design}}^{1.3} s_{\text{design}}^{1.4} - 244 \right) \right] \]

For span w/ key: \[ M_{\text{positive}} = M_{\text{p,AASHTO}} \left[ 0.58 + 0.0196 \left( l_{\text{design}}^{1} s_{\text{design}}^{0.8} - 51 \right) \right] \]

Negative design moment:

For exterior beams: \[ M_{\text{negative}} = M_{\text{n,AASHTO}} \left[ 0.4 + 6.28 \left( l_{\text{design}}^{0.1} s_{\text{design}}^{0.2} - 1.69 \right) \right] \]

For interior beams: \[ M_{\text{negative}} = M_{\text{n,AASHTO}} \]

The negative moment demand in the interior beam shall not exceed that in the exterior beam.

where,

\[ l_{\text{design}} = \text{design span length, (ft)} \quad l_{\text{design}} = 39 \text{ft for the 40ft span bridge} \]

\[ M_{\text{positive}} = \text{unfactored positive moment demand from live load (kip*ft/ft)} \]

\[ M_{\text{negative}} = \text{unfactored negative moment demand from live load (kip*ft/ft)} \]

Based on the formula, the normalized live load demands (kip*ft/ft) are:

<table>
<thead>
<tr>
<th>Table 3</th>
<th>Location</th>
<th>22 ft</th>
<th>30 ft</th>
<th>40 ft</th>
</tr>
</thead>
<tbody>
<tr>
<td>[ M_{\text{positive}} ] (kip*ft/ft)</td>
<td>Span w/o key</td>
<td>2.07</td>
<td>2.99</td>
<td>4.25</td>
</tr>
<tr>
<td>[ M_{\text{positive}} ] (kip*ft/ft)</td>
<td>Span w/ key</td>
<td>1.54</td>
<td>2.55</td>
<td>3.82</td>
</tr>
<tr>
<td>[ M_{\text{negative}} ] (kip*ft/ft)</td>
<td>Exterior beam</td>
<td>−0.36</td>
<td>−0.67</td>
<td>−0.97</td>
</tr>
<tr>
<td>[ M_{\text{negative}} ] (kip*ft/ft)</td>
<td>Interior beam</td>
<td>−0.91</td>
<td>−0.91</td>
<td>−0.91</td>
</tr>
</tbody>
</table>

Since the first two negative moment demands in the interior beam exceed those in the exterior beam, these two values will be equal to those in the exterior beam. There is not much difference between the negative moment values for the 40ft span, therefore, it is decided to use the negative moment demands in the exterior beam for all the beam design for NEXT-6.

Therefore the table above is updated to be:
Table 4

<table>
<thead>
<tr>
<th>Location</th>
<th>22 ft</th>
<th>30 ft</th>
<th>40 ft</th>
</tr>
</thead>
<tbody>
<tr>
<td>$M_{\text{positive}}$ (kip*ft/ft) Span w/o key</td>
<td>2.07</td>
<td>2.99</td>
<td>4.25</td>
</tr>
<tr>
<td>$M_{\text{positive}}$ (kip*ft/ft) Span w/ key</td>
<td>1.54</td>
<td>2.55</td>
<td>3.82</td>
</tr>
<tr>
<td>$M_{\text{negative}}$ (kip*ft/ft) All beams</td>
<td>$-0.36$</td>
<td>$-0.67$</td>
<td>$-0.97$</td>
</tr>
</tbody>
</table>

The dead load effect is obtained based on the 1d AASHTO FEM. The moment demands from beam self weight, barrier self weight, and wearing surface are combined for the Strength I limit state and Service limit state. For each limit state, both the maximum and minimum combined moment demands are obtained and listed in the following table.

Table 5

<table>
<thead>
<tr>
<th></th>
<th>Strength I</th>
<th>Service I</th>
</tr>
</thead>
<tbody>
<tr>
<td>$M_{\text{p,DL}}$ (kip*ft/ft)</td>
<td>0.1724</td>
<td>0.1351</td>
</tr>
<tr>
<td>$M_{\text{n,DL}}$ (kip*ft/ft)</td>
<td>$-0.7662$</td>
<td>$-0.6097$</td>
</tr>
</tbody>
</table>

The total load effect is calculated as:

$$M_{\text{p,\text{total}}} = M_{\text{positive}} \cdot IM \cdot m \cdot LF \cdot M_{\text{p,DL}}$$

$$M_{\text{n,\text{total}}} = M_{\text{negative}} \cdot IM \cdot m \cdot LF \cdot M_{\text{n,DL}}$$

where,

- $M_{\text{p,\text{total}}}$ = final design positive moment demand for either Strength I or Service I limit state (kip*ft/ft)
- $M_{\text{n,\text{total}}}$ = final design negative moment demand for either Strength I or Service I limit state (kip*ft/ft)
- $M_{\text{positive}}$ = unfactored positive moment demand from live load (kip*ft/ft)
- $M_{\text{negative}}$ = unfactored negative moment demand from live load (kip*ft/ft)
- $M_{\text{p,DL}}$ = factored positive moment demand from dead load (kip*ft/ft)
- $M_{\text{n,DL}}$ = factored negative moment demand from dead load (kip*ft/ft)
- $IM$ = dynamic load allowance percent, $IM = 1.33$ LRFD Table 3.6.2.1-1
- $m$ = multiple presence factor, $MP = 1.2$ LRFD Table 3.6.1.1.2-1
- $LF$ = live load factor, $LL = 1.75$ for Strength I limit state, and $1$ for Service I limit state LRFD Table 3.4.1-1

The final design demands for Strength I and Service I limit states are listed in Table 6 and 7. The values given in brackets are the demands based on the 3d FEM results. The percentage errors for the unconservative demands are also given by taking the 3d FEM results as the 'correct' values. The unconservative values are all within five percentage of the FEM results.
Table 6

<table>
<thead>
<tr>
<th>Location</th>
<th>22 ft</th>
<th>30 ft</th>
<th>40 ft</th>
</tr>
</thead>
<tbody>
<tr>
<td>$M_p_{\text{total}}$ (kip*ft/ft) Span w/o key</td>
<td>6.46 (6.15)</td>
<td>9.04 (8.85)</td>
<td>12.57 (12.70)</td>
</tr>
<tr>
<td>$M_p_{\text{total}}$ (kip*ft/ft) Span w/ key</td>
<td>4.98 (4.35)</td>
<td>7.81 (7.37)</td>
<td>11.36 (11.84)</td>
</tr>
<tr>
<td>$M_n_{\text{total}}$ (kip*ft/ft) All beams</td>
<td>-1.77 (-0.98)</td>
<td>-2.65 (-1.55)</td>
<td>-3.49 (-2.61)</td>
</tr>
</tbody>
</table>

Table 7

<table>
<thead>
<tr>
<th>Location</th>
<th>22 ft</th>
<th>30 ft</th>
<th>40 ft</th>
</tr>
</thead>
<tbody>
<tr>
<td>$M_p_{\text{total}}$ (kip*ft/ft) Span w/o key</td>
<td>3.84 (3.59)</td>
<td>5.31 (5.19)</td>
<td>7.33 (7.47)</td>
</tr>
<tr>
<td>$M_p_{\text{total}}$ (kip*ft/ft) Span w/ key</td>
<td>2.99 (2.51)</td>
<td>4.61 (4.28)</td>
<td>6.64 (6.94)</td>
</tr>
<tr>
<td>$M_n_{\text{total}}$ (kip*ft/ft) All beams</td>
<td>-1.18 (-0.54)</td>
<td>-1.69 (-0.82)</td>
<td>-2.16 (-1.43)</td>
</tr>
</tbody>
</table>

Comparing the demands with capacities provided by various rebar configurations:

Table 8

<table>
<thead>
<tr>
<th>Location</th>
<th>Demand</th>
<th>Capacity provided</th>
</tr>
</thead>
<tbody>
<tr>
<td>22 ft</td>
<td>30 ft</td>
<td>40 ft</td>
</tr>
<tr>
<td>$M_p_{\text{total}}$ (kip*ft/ft) Span w/o key</td>
<td>6.46</td>
<td>9.04</td>
</tr>
<tr>
<td>$M_p_{\text{total}}$ (kip*ft/ft) Span w/ key</td>
<td>4.98</td>
<td>7.81</td>
</tr>
<tr>
<td>$M_n_{\text{total}}$ (kip*ft/ft) All beams</td>
<td>-1.77</td>
<td>-2.65</td>
</tr>
</tbody>
</table>

For the deck span without shear key—-which is the deck in-between the two stems of a beam—due to the development length of the rebar, the actual capacity is larger than that in the span with shear key. This also applies to the table in section 4, 5, 6, and 7.2.

4. 40ft bridge design—#4 @ 7in

A summary of the normalized demands in Strength I limit state for the 40ft span, according to Table 6, is as follows:

Table 9

<table>
<thead>
<tr>
<th>Location</th>
<th>Demand</th>
<th>Capacity provided by #4@7in Ubar</th>
</tr>
</thead>
<tbody>
<tr>
<td>$M_p_{\text{total}}$ (kip*ft/ft) Span w/o key</td>
<td>12.57</td>
<td>N/A</td>
</tr>
<tr>
<td>$M_p_{\text{total}}$ (kip*ft/ft) Span w/ key</td>
<td>11.36</td>
<td>13.59</td>
</tr>
<tr>
<td>$M_n_{\text{total}}$ (kip*ft/ft) All beams</td>
<td>-3.49</td>
<td>-8.90</td>
</tr>
</tbody>
</table>
4.1 Development length

Rebar area: \( A_4 := 0.2 \text{in}^2 \)

Rebar diameter: \( d_4 := 0.5 \text{in} \)

**Straight development:**

LRFD Art. 5.11.2.1

\[
\begin{align*}
    l_{hb} := \max \left( \frac{1.25 \cdot A_4 \cdot f_y \cdot \text{in}}{ \frac{f'_c}{\text{ksi}} \cdot \frac{kip}{\text{ksi}}}, 0.4 \cdot \frac{d_4 \cdot f_y}{\text{ksi}} \right) &= 12 \text{in} \quad \text{where,} \quad f'_c = 6.5 \text{ksi} \quad f_y = 60 \text{ksi}
\end{align*}
\]

\( f_{\text{modS}} := 1 \)

\[
l_{dh} := \max(f_{\text{modS}} \cdot l_{hb}, 12 \text{in}) = 12 \text{in}
\]

According to LRFD Art. 5.11.1.2.1, except at supports of simple spans and at the free ends of cantilevers, reinforcement shall be extended beyond the point at which it is no longer required to resist flexure for a distance calculated above.

**Interior beam:**

For the both positive and negative reinforcement, this point is taken at the centerline of the beam.

4.2. Limits of reinforcement

4.2.1 Maximum reinforcement

The check of maximum reinforcement limits was removed from the LRFD Specifications in 2005. LRFD Art. 5.7.3.3.1

4.2.2 Minimum reinforcement

At any section of a noncompression-controlled flexural component, the amount of prestressed and nonprestressed tensile reinforcement shall be adequate to develop a factored flexural resistance, \( M_r \), at least equal to the lesser of:

1. 1.33 times the factored moment required by the applicable strength load combination specified in Table 3.4.1-1 (Strength I); and

2. \( M_{cr} = \gamma_3 (\gamma_1 \cdot f_y) \cdot S_t \quad \text{LRFD Eq. 5.7.3.3.2-1} \)

The above equation is a simplified form of LRFD Equation 5.7.3.3.2-1 because no composite section exists, therefore the composite and noncomposite section modulus are the same. Also since there is no transverse post-tensioning, the cracking moment capacity won't be increased.
where,

\[ f_r = \text{modulus of rupture of concrete} \quad \text{LRFD Art 5.4.2.6} \]

\[ f'_r := 0.37 \frac{f_c}{\text{ksi}} = 0.943 \text{ ksi} \]

\[ S_r = \text{section modulus for the extreme fiber of the section where tensile stress is caused by externally applied loads (in}^3) \quad S_r = 128 \text{ in}^3 \]

\[ \gamma_1 = \text{flexural cracking variability factor} \]

\[ \gamma_1 := 1.2 \quad \text{for precast segmental structures} \]

\[ \gamma_3 = \text{ratio of specified minimum yield strength to ultimate tensile strength of the reinforcement} \]

\[ \gamma_3 := 0.75 \quad \text{for A 706, Grade 60 reinforcement} \]

\[ M_{cr} := \gamma_3 (\gamma_1 \cdot f'_r) \cdot S_r = 9.056 \text{ kip} \cdot \text{ft} \]

The above \( M_{cr} \) applies to any cross section in the deck

4.2.2.1 Positive moment

The factored normalized positive moment demand in Strength I limit state in the deck span with shear key, according to Table 9, is:

\[ M_u := 11.36 \text{ kip} \cdot \text{ft} \]

Thus, \( 1.33 \cdot M_u = 15.109 \text{ kip} \cdot \text{ft} > M_{cr} = 9.056 \text{ kip} \cdot \text{ft} \)

Therefore \( M_{cr} \) requirement controls.

The factored capacity provided by \#4@7in is:

\[ M_r := 13.59 \text{ kip} \cdot \text{ft} > M_{cr} = 9.056 \text{ kip} \cdot \text{ft} \quad \text{OK} \]

This criteria can be met at every deck section.

4.2.2.2 Negative moment

The factored normalized negative moment demand in Strength I limit state for all the beams, according to Table 9, is:

\[ M_u := -3.49 \text{ kip} \cdot \text{ft} \]

Thus, \( 1.33 \cdot |M_u| = 4.642 \text{ kip} \cdot \text{ft} < M_{cr} = 9.056 \text{ kip} \cdot \text{ft} \)

Therefore \( 1.33 \cdot |M_u| \) requirement controls.
The least negative moment capacity comes from the rebar configuration of #4 @7in:

\[
|M_h| = 8.9\text{kip-ft} > |1.33\cdot M_u| = 4.642\text{-kip-ft} \quad \text{OK}
\]

Therefore this criteria can be met at every deck section.

4.3. Distribution reinforcement

The required area of secondary reinforcement at the bottom of the deck is a percentage of the primary positive moment reinforcement. For primary reinforcement perpendicular to traffic, LRFD Art. 9.7.3.2 specifies that the percentage should be:

\[
\text{Percentage} = \frac{220}{\sqrt{S}} \leq 67\%
\]

where

\[ S = \text{the effective span length (ft)} \quad \text{LRFD Art. 9.7.2.3} \]

\[ S := 3\text{ft} - 15\text{in} = 21\cdot\text{in} \]

\[ \text{Percentage} := \frac{220}{\sqrt{\frac{S}{\text{ft}}}} = 166.30\% \quad \text{use 67\%} \]

For both the Interior beam and exterior beam:

Positive moment reinforcement in the transverse direction (#4@7in): \( A_s := 0.34 \text{in}^2/\text{ft} \)

\[ A_s\cdot67\% = 0.228\text{in}^2/\text{ft} \]

For longitudinal bottom bar, Use #4@10in, which gives \( 0.24\text{in}^2/\text{ft} \)

4.4. Shrinkage and temperature reinforcement

The minimum reinforcement area per foot, on each face and in each direction, shall satisfy:

\[
A_{s\_TS} \geq \frac{1.3\cdot b\cdot h}{2\cdot (b + h)\cdot f_y} \quad \text{LRFD Eq. 5.10.8-1}
\]

where

\[ A_{s\_TS} = \text{area of reinforcement in each direction and each face (in}^2/\text{ft}) \]

\[ b = \text{least width of component section (in)} \quad b := 1\text{ft} \]

\[ h = \text{least thickness of component section (in)} \quad h = 8\text{-in} \]

\[ f_y = \text{specified yield strength of reinforcing bars} \quad f_y = 60\text{ksi} \leq 75\text{ksi} \]

Therefore
\[ A_{s\_TS} \geq \frac{1.3 \cdot 12 \cdot 8}{2 \cdot (12 + 8) \cdot 60} \text{ in}^2 = 0.052 \text{ in}^2 / \text{ft} \]

Use \[ A_{s\_TS} = 0.11 \text{ in}^2 / \text{ft} \] LRFD Eq. 5.10.8-2

This requirement can be satisfied in each direction and each face. Since the deck depth is more than 6in, the shrinkage and temperature rebars need to be provided equally on both layers. The maximum spacing of the rebar shall not exceed either 3 times the deck depth or 18in. The top longitudinal bars are provided by the bonded reinforcement, which is \#4 @7in for the exterior beam, and \#4@10in for the interior beam as calculated below.

From CONSPAN, at release, the tension stress in the top fiber of the exterior beam at the transfer cross section is:

\[ f_t = -0.476 \text{ksi} \]

The compressive stress in the bottom fiber of the beam at the same cross section is:

\[ f_b = 2.78 \text{ksi} \]

The depth of the tensile zone \( x \) is: LRFD C5.9.4.1.2

\[
\frac{-f_t}{x} = \frac{f_b}{21 \text{in} - x}
\]

\[
x := \frac{21 \cdot f_b - f_t}{f_b - f_t} = 3.07 \text{in} < 8 \text{in}
\]

The tensile force \( T \) in the concrete is:

\[ T := \frac{f_t}{2} \cdot b_{6\_ext} \cdot x \]

where, \( b_{6\_ext} \) is the width of the beam at top \( b_{6\_ext} = 84 \text{in} \)

Therefore \[ T := \frac{-f_t}{2} \cdot b_{6\_ext} \cdot x = 61.376 \text{-kip} \]

The required area of bonded reinforcement is:

\[ A_{req} = \frac{T}{f_s} \]
where \( f_s := 0.5 \cdot f_y = 30 \text{ ksi} \) \[ \text{LRFD C5.9.4.1.2} \]

Therefore \( A_{\text{req}} := \frac{T}{f_s \cdot b_{6\text{ext}}} = 0.292 \text{ in}^2 \text{ ft} \)

Use #4 @ 7in within the tensile zone \( A_s := 0.34 \text{ in}^2 \text{ ft} > A_{\text{req}} \text{ OK} \)

From CONSPAN, at release, the tension stress in the top fiber of the interior beam at the transfer cross section is:

\( f_t := -0.398 \text{ ksi} \)

The compressive stress in the bottom fiber of the beam at the same cross section is:

\( f_b := 2.392 \text{ ksi} \)

The depth of the tensile zone \( x \) is: \[ \text{LRFD C5.9.4.1.2} \]

\[ \frac{-f_t}{x} = \frac{f_b}{21 \text{in} - x} \]

\[ x := \frac{21 \cdot f_t \cdot \text{in}}{f_b - f_t} = 2.996 \cdot \text{in} < 8 \text{ in} \]

The tensile force \( T \) in the concrete is: \[ T := \frac{f_t}{2} \cdot b_{6\text{int}} \cdot x \]

where, \( b_{6\text{int}} \) is the width of the beam at top \( b_{6\text{int}} = 72 \text{ in} \)

Therefore \[ T := \frac{-f_t}{2} \cdot b_{6\text{int}} \cdot x = 42.922 \text{ kip} \]

The required area of bonded reinforcement is: \[ A_{\text{req}} = \frac{T}{f_s} \]

where \( f_s := 0.5 \cdot f_y = 30 \text{ ksi} \) \[ \text{LRFD C5.9.4.1.2} \]

Therefore \[ A_{\text{req}} := \frac{T}{f_s \cdot b_{6\text{int}}} = 0.238 \text{ in}^2 \text{ ft} \]

Use #4 @ 10in within the tensile zone \( A_s := 0.24 \text{ in}^2 \text{ ft} > A_{\text{req}} \text{ OK} \)

4.5. Control of cracking \[ \text{LRFD Art. A.5.7.3.4} \]

In the longitudinal direction, due to the existence of prestress strands, cracking is assumed to not happen. Therefore only in the transverse direction, cracking is considered.
The spacing of mild steel reinforcement in the layer closest to the tension face shall satisfy the following:

\[ s \leq \frac{700 \cdot \gamma_e}{\beta_s \cdot f_{ss}} - 2 \cdot d_c \]

where

\[ \gamma_e = \text{exposure factor} \]
- \( 1.00 \) for Class 1 exposure condition
- \( 0.75 \) for Class 2 exposure condition

\[ \gamma_e = 0.75 \]

SCDOT Bridge Design Manual 15.1.7

\[ d_c = \text{thickness of concrete cover measured from extreme tension fiber to center of the flexural reinforcement located closest thereto (in)} \]

\[ f_{ss} = \text{tensile stress in steel reinforcement at the service limit state (ksi)} \]

\[ h = \text{overall thickness or depth of the component (in)} \]

\[ h = 8 \text{ in} \]

LRFD Art. 3.4.1 specifies that Service I limit state should be investigated for crack control in reinforced concrete structures. According to the previous calculation in Table 7, the final design demands for the 40ft span in the Service I limit state are summarized below:

<table>
<thead>
<tr>
<th>Location</th>
<th>Demand</th>
</tr>
</thead>
<tbody>
<tr>
<td>( M_{p \text{total}} ) (kip*ft/ft)</td>
<td>Span w/o key</td>
</tr>
<tr>
<td>( M_{p \text{total}} ) (kip*ft/ft)</td>
<td>Span w/ key</td>
</tr>
<tr>
<td>( M_{n \text{total}} ) (kip*ft/ft)</td>
<td>All beams</td>
</tr>
</tbody>
</table>

The section is transformed elastic, cracked cross section. LRFD Art. 5.7.1

modulus of elasticity, ksi \[ = \frac{33000 \cdot K_1 \cdot w_c^{1.5} \cdot f_c^{1.5}}{150 \cdot f_c} \]

LRFD Eq 5.4.2.4-1

where

correction factor for source of aggregate: \( K_1 := 1 \)

unit weight of concrete: \( w_c = 150 \text{pcf} \)

This unit weight is higher than what is given in LRFD Table 3.5.1-1. It is to be used for deck design unless more precise information is provided.

\[ f_c = \text{specified compressive strength of concrete, ksi} \]

\[ f_c = 6.5 \text{ ksi} \]

Therefore, the modulus of elasticity:
\[ E_c := 33000 \cdot K_1 \left( \frac{w_c}{1000 \text{pcf}} \right)^{1.5} \sqrt{\frac{f_c'}{\text{ksi}}} = 4.888 \times 10^3 \cdot \text{ksi} \]

Modulus ratio:
\[ n_c := \frac{E_s}{E_c} = 5.933 \]

where, \[ E_s = 2.9 \times 10^4 \cdot \text{ksi} \]

4.5.1 Check of the positive moment reinforcement

Check the maximum positive moment against rebar configuration of \#4@7in

According to Table 10, \[ M_{\text{pos}} := 6.64 \frac{\text{kip-ft}}{\text{ft}} \]

4.5.1.1 Cracked moment of inertia

For a 1ft cross section, \( b := 1\text{ft} \)

The distance from the top fiber of the deck to the bottom layer of reinforcement:
\[ d := h - \text{Cover}_b - 0.5 \cdot d_4 = 6.75\text{-in} \]

The distance from the bottom fiber of the deck to the top layer of reinforcement:
\[ d' := h - \text{Cover}_t - 0.5 \cdot d_4 = 5.25\text{-in} \]

Bottom layer of rebar:
\[ A_s := 0.34 \frac{\text{in}^2}{\text{ft}} \]

Top layer of rebar:
\[ A'_s := 0.34 \frac{\text{in}^2}{\text{ft}} \]

The location of neutral axis \( x \) measured from the top fiber of the beam is determined as below.

Sum of statical moments about the neutral axis gives:
\[
\frac{1}{2} x^2 = n_c \cdot A_s \cdot (d - x) + n_c \cdot A'_s \cdot (h - d' - x)
\]

\[ x := \sqrt{n_c \left( A_s^2 \cdot n_c + A'_s^2 \cdot n_c + 2 \cdot A_s \cdot d - 2 \cdot A'_s \cdot d' + 2 \cdot A'_s \cdot h + 2 \cdot A_s \cdot A'_s \cdot n_c \right)} - A_s \cdot n_c - A'_s \cdot n_c = 1.482\text{-in} \]

The cracked moment of inertia therefore is:
\[ I_{\text{cr}} := \frac{b \cdot x^3}{3} + n_c \cdot A_s \cdot b \cdot (d - x)^2 + n_c \cdot A'_s \cdot b \cdot (h - d' - x)^2 = 72.247\text{-in}^4 \]

4.5.1.2 Tensile stress in the bottom steel

\[ f_{ss} := \frac{n_c \cdot M_{\text{pos}} \cdot b}{I_{\text{cr}}} \cdot (d - x) = 34.47 \cdot \text{ksi} \]

where, \[ n_c = 5.933 \]
4.5.1.3 Rebar spacing

The thickness of concrete cover measured from extreme tension fiber to center of the flexural reinforcement located closest thereto (in) is:

\[ d_c := h - d = 1.25 \text{ in} \quad \text{where,} \quad h = 8 \text{ in} \]

Therefore:

\[ \beta_s := 1 + \frac{d_c}{0.7(h - d_c)} = 1.265 \]

where, \( h = 8 \text{ in} \)

So that the maximum rebar spacing is:

\[ s_{\text{max}} := \frac{700 \cdot \gamma_c}{f_{\text{ss}}} \text{ in} - 2d_c = 9.544 \text{ in} \quad > 7\text{ in} \quad \text{OK} \quad \text{where,} \quad \gamma_c = 0.75 \]

4.5.2 Check of the negative moment reinforcement

Check the maximum negative moment demand against the #4@7in

According to Table 10, \( M_{\text{neg}} := 2.06 \frac{\text{kip-ft}}{\text{ft}} \)

4.5.2.1 Cracked moment of inertia

For a 1ft cross section, \( b := 1\text{ft} \)

Bottom layer of rebar: \( A_s := 0.34 \frac{\text{in}^2}{\text{ft}} \)

Top layer of rebar: \( A'_s := 0.34 \frac{\text{in}^2}{\text{ft}} \)

The location of neutral axis \( x \) measured from the bottom fiber of the beam is determined as below.

Sum of statical moments about the neutral axis gives:

\[ \frac{1}{2}x^2 = n_c \cdot A_s \cdot (h - d - x) + n_c \cdot A'_s \cdot (d' - x) \]

where, \( n_c = 5.933 \quad d = 6.75 \text{ in} \quad d' = 5.25 \text{ in} \)

\[ x := \sqrt{n_c \left( A_s^2 \cdot n_c + A'_s^2 \cdot n_c - 2 \cdot A_s \cdot d + 2 \cdot A'_s \cdot d' + 2 \cdot A_s \cdot h + 2 \cdot A'_s \cdot A'_s \cdot n_c \right) - A_s \cdot n_c - A'_s \cdot n_c} = 1.18 \text{ in} \]

The cracked moment of inertia therefore is:

\[ I_{\text{cr}} := \frac{b \cdot x^3}{3} + n_c \cdot A_s \cdot b \cdot (h - d - x)^2 + n_c \cdot A'_s \cdot b \cdot (d' - x)^2 = 39.998 \text{ in}^4 \]

4.5.2.2 Tensile stress in the top steel

\[ f_{\text{ss}} := \frac{n_c \cdot M_{\text{neg}} \cdot b}{I_{\text{cr}}} \cdot (d' - x) = 14.925 \text{ ksi} \]
4.5.2.3 Rebar spacing

The thickness of concrete cover measured from extreme tension fiber to center of the flexural reinforcement located closest thereto (in) is:

\[ d_c := h - d' = 2.75 \text{ in} \quad \text{where,} \quad h = 8 \text{ in} \quad d' = 5.25 \text{ in} \]

Therefore:

\[ \beta_s := 1 + \frac{d_c}{0.7(h - d_c)} = 1.748 \]

So that the maximum rebar spacing is:

\[ s_{\text{max}} := \frac{700 \cdot \gamma_{\text{c}}}{\beta_s} - 2 \cdot d_c = 14.62 \text{ in} > 7 \text{ in} \quad \text{OK} \]

5. 30 ft bridge design using #4@10in

A summary of the demands in Strength I limit state on a 1ft strip for the 30ft span, according to Table 6, is as follows:

<table>
<thead>
<tr>
<th>Table 11</th>
<th>Location</th>
<th>Demand</th>
<th>Capacity provided by #4@10in Ubar</th>
</tr>
</thead>
<tbody>
<tr>
<td>( M_{\text{p total}} ) (kip*ft/ft)</td>
<td>Span w/o key</td>
<td>9.04</td>
<td>N/A</td>
</tr>
<tr>
<td>( M_{\text{p total}} ) (kip*ft/ft)</td>
<td>Span w/ key</td>
<td>7.81</td>
<td>9.79</td>
</tr>
<tr>
<td>( M_{\text{n total}} ) (kip*ft/ft)</td>
<td>All beams</td>
<td>-2.65</td>
<td>-6.55</td>
</tr>
</tbody>
</table>

5.1 Development length

The development length for #4 bar is just as determined before: \( l_{dh} = 12 \text{ in} \)

Interior beam:

For both the positive and negative moment reinforcement, this point where the development length begins is taken at the centerline of the beam.

5.2. Minimum reinforcement LRFD Art. 5.7.3.3.1

At any section of a noncompression-controlled flexural component, the amount of prestressed and nonprestressed tensile reinforcement shall be adequate to develop a factored flexural resistance, \( M_r \), at least equal to the lesser of:

1. 1.33 times the factored moment required by the applicable strength load combination specified in Table 3.4.1-1 (Strength I); and

2. \( M_{cr} = 9.056 \cdot \text{kip} \cdot \text{ft} \)

The above \( M_{cr} \) applies to any cross section in the deck

5.2.1 Positive moment

The factored positive moment required by strength I load combination in between beams, according to Table 11, is:
$M_u = 7.81 \text{kip-ft}$

Thus, $1.33 \times M_u = 10.387 \text{kip-ft} > M_{cr} = 9.056 \text{kip-ft}$

Therefore $M_{cr}$ requirement controls.

The factored capacity provided by #4@10in is:

$M_r = 9.79 \text{kip-ft} > M_{cr} = 9.056 \text{kip-ft}$ OK

5.2.2 Negative moment

The critical factored negative moment required by strength I load combination, according to Table 11, is:

$M_u = -2.65 \text{kip-ft}$

Thus, $1.33 \times M_u = 3.524 \text{kip-ft} < M_{cr} = 9.056 \text{kip-ft}$

Therefore $1.33 \times M_u$ requirement controls.

The least negative moment capacity comes from the rebar configuration of #4@10in

$M_r = 6.55 \text{kip-ft} > 1.33 \times M_u = 3.524 \text{kip-ft}$ OK

Therefore this criteria can be met at every deck section.

5.3. Distribution reinforcement

As determined before, the minimum percentage of bottom longitudinal reinforcement should be:

Percentage := 67%

For both the Interior beam and exterior beam

Positive moment reinforcement in the transverse direction (#4@10in): $A_s := 0.24 \frac{\text{in}^2}{\text{ft}}$

$A_s{67\%} = 0.161 \frac{\text{in}^2}{\text{ft}}$

For longitudinal bottom bar, Use #4@12in, which gives $0.2 \frac{\text{in}^2}{\text{ft}}$

5.4. Shrinkage and temperature reinforcement

As determined before, the minimum reinforcement area per foot, on each face and in each direction, shall be:

$A_{s\_TS} := 0.11 \frac{\text{in}^2}{\text{ft}}$

This criteria can be met on each face and in each direction. But this reinforcement need to be provided equally on both layers. The maximum spacing of the rebar shall not exceed either 3 times the deck depth or 18in. The top longitudinal bars are provided by the bonded reinforcement, which is #4@16in for both the interior and exterior beams.
From CONSPAN, at release, the tension stress in the top fiber of the **exterior beam** at the transfer cross section is: 

\[ f_t := -0.26 \text{ksi} \]

The compressive stress in the bottom fiber of the beam at the same cross section is:

\[ f_b := 1.678 \text{ksi} \]

Refer to Figure F.19, the depth of the tensile zone \(x\) is: 

\[ \frac{-f_t}{f_b} = \frac{21 \cdot f_t \text{in}}{f_b - f_t} = 2.817 \text{in} < 8 \text{in} \]

The tensile force \(T\) in the concrete is: 

\[ T := \frac{f_t}{2} \cdot b_{6\text{ext}} x \]

where, \(b_{6\text{ext}}\) is the width of the beam at top \(b_{6\text{ext}} = 84 \text{in}\)

Therefore

\[ T := \frac{-f_t}{2} \cdot b_{6\text{ext}} x = 30.765 \text{kip} \]

The required area of bonded reinforcement is: 

\[ A_{\text{req}} := \frac{T}{f_s} \]

where \(f_s := 0.5 \cdot f_y = 30 \text{ksi}\)

LRFD C5.9.4.1.2

Therefore

\[ A_{\text{req}} := \frac{T}{f_s b_{6\text{ext}}} = 0.147 \frac{\text{in}^2}{\text{ft}} \]

Use #4 @16in within the tensile zone

\[ A_S := 0.15 \frac{\text{in}^2}{\text{ft}} > A_{\text{req}} \quad \text{OK} \]

From CONSPAN, at release, the tension stress in the top fiber of the **interior beam** at the transfer cross section is: 

\[ f_t := -0.26 \text{ksi} \]

The compressive stress in the bottom fiber of the beam at the same cross section is:

\[ f_b := 1.706 \text{ksi} \]

The depth of the tensile zone \(x\) is: 

\[ \frac{-f_t}{f_b} = \frac{21 \cdot f_t \text{in}}{f_b - f_t} = 2.777 \text{in} < 8 \text{in} \]

The tensile force \(T\) in the concrete is: 

\[ T := \frac{f_t}{2} \cdot b_{6\text{int}} x \]

where, \(b_{6\text{int}}\) is the width of the beam at top \(b_{6\text{int}} = 72 \text{in}\)
Therefore \[ T := \frac{-f_t}{2} \cdot b_{\text{int}} \cdot x = 25.995 \text{kip} \]

The required area of bonded reinforcement is: \[ A_{\text{req}} := \frac{T}{f_s} \]

where \[ f_s := 0.5 \cdot f_y = 30 \text{ ksi} \] LRFD C5.9.4.1.2

Therefore \[ A_{\text{req}} := \frac{T}{f_s \cdot b_{\text{int}}} = 0.144 \text{ in}^2 / \text{ft} \]

Use #4 @16in within the tensile zone \[ A_s := 0.15 \text{ in}^2 / \text{ft} > A_{\text{req}} \text{ OK} \]

5.5. Control of cracking LRFD Art. A.5.7.3.4

A summary of the demands on a 1ft strip in Service I limit state for the 30ft span, according to Table 7, is as follows:

Table 12 Location Demand
-----------------------------------------------------------------------
\[ M_{p_{\text{total}}} \text{ (kip}\cdot\text{ft/ft)} \quad \text{Span w/o key} \quad 5.31 \]
\[ M_{p_{\text{total}}} \text{ (kip}\cdot\text{ft/ft)} \quad \text{Span w/ key} \quad 4.61 \]
\[ M_{n_{\text{total}}} \text{ (kip}\cdot\text{ft/ft)} \quad \text{All beams} \quad -2.06 \]
-----------------------------------------------------------------------

The section is transformed elastic, cracked cross section. LRFD Art. 5.7.1

5.5.1 Check of the positive moment reinforcement

Check the positive moment against rebar configuration of #4@10in

According to Table 12, \[ M_{\text{pos}} := 4.61 \text{ kip}\cdot\text{ft} / \text{ft} \]

5.5.1.1 Cracked moment of inertia

For a 1ft cross section, \[ b := 1\text{ft} \]

The distance from the top fiber of the deck to the bottom layer of reinforcement:
\[ d := h - \text{Cover}_b - 0.5 \cdot d_4 = 6.75 \text{ in} \]

The distance from the bottom fiber of the deck to the top layer of reinforcement:
\[ d' := h - \text{Cover}_t - 0.5 \cdot d_4 = 5.25 \text{ in} \]

Bottom layer of rebar: \[ A_s := 0.24 \text{ in}^2 / \text{ft} \]

Top layer of rebar: \[ A'_s := 0.24 \text{ in}^2 / \text{ft} \]

The location of neutral axis \( x \) measured from the top fiber of the beam is determined as below.

Sum of statical moments about the neutral axis gives:
\[
\frac{1}{2}x^2 = n_c A_s (d - x) + n_c A'_s (h - d' - x)
\text{ where, } n_c = 5.933
\]

\[
x := \sqrt{n_c \left( A_s^2 - n_c^2 A'_s + 2 A_s d - 2 A'_s d' + 2 A'_s h + 2 A_s A'_s n_c \right)} - A_s n_c - A'_s n_c = 1.283 \text{-in}
\]

\[
x < h - d' = 2.75 \text{-in}
\]

The cracked moment of inertia therefore is:

\[
I_{cr} := \frac{b \cdot x^3}{3} + n_c A_s b \cdot (d - x)^2 + n_c A'_s b \cdot (h - d' - x)^2 = 54.072 \text{-in}^4
\]

where, \( n_c = 5.933 \int 8 \) \text{-in}

5.5.1.2 Tensile stress in the bottom steel

\[
f_{ss} := n_c \frac{M_{pos} b}{I_{cr}} (d - x) = 33.186 \text{-ksi}
\]

5.5.1.3 Rebar spacing

The thickness of concrete cover measured from extreme tension fiber to center of the flexural reinforcement located closest thereto (in) is:

\[
d_c := h - d = 1.25 \text{-in}
\]

where, \( h = 8 \) \text{-in}

Therefore:

\[
\beta_s := 1 + \frac{d_c}{0.7 \left(h - d_c\right)} = 1.265
\]

So that the maximum rebar spacing is:

\[
s_{\text{max}} := \frac{700 \gamma_{c_e}}{f_{ss}} \text{in} - 2 d_c = 10.01 \text{-in} \quad \text{OK}
\]

where, \( \gamma_{c_e} = 0.75 \)

5.5.2 Check of the negative moment reinforcement

Check the maximum negative moment demand against the \#4@10in rebar configuration

According to Table 12, \( M_{\text{neg}} := 1.69 \frac{\text{kip} \cdot \text{ft}}{\text{ft}} \)

5.5.2.1 Cracked moment of inertia

For a 1ft cross section, \( b := 1 \text{-ft} \)

Bottom layer of rebar: \( A_s = 0.24 \frac{\text{in}^2}{\text{ft}} \)

Top layer of rebar: \( A'_s = 0.24 \frac{\text{in}^2}{\text{ft}} \)
The location of neutral axis $x$ measured from the bottom fiber of the beam is determined as below.

Sum of statical moments about the neutral axis gives:

$$\frac{1}{2}x^2 = n_c A_s (h - d - x) + n_c A'_s (d' - x)$$

where, $n_c = 5.933$, $d = 6.75$-in, $d' = 5.25$-in

$$x = \sqrt{n_c \left( A_s^2 n_c + A'_s^2 n_c - 2A_s d + 2A'_s d' + 2A_s h + 2A'_s A_s n_c \right) - A_s n_c A'_s n_c - 1.027 \text{-in}}$$

The cracked moment of inertia therefore is:

$$I_{cr} = \frac{b x^3}{3} + n_c A_s b (h - d - x)^2 + n_c A'_s b (d' - x)^2 = 29.798 \text{-in}^4$$

5.5.2.2 Tensile stress in the top steel

$$f_{ss} := n_c \frac{M_{neg-b}}{I_{cr}} - (d' - x) = 17.052 \text{-ksi}$$

5.5.2.3 Rebar spacing

The thickness of concrete cover measured from extreme tension fiber to center of the flexural reinforcement located closest thereto (in) is:

$$d_c := h - d' = 2.75 \text{-in}$$

where, $h = 8$-in, $d' = 5.25$-in

Therefore:

$$\beta_s := 1 + \frac{d_c}{0.7(h - d_c)} = 1.748$$

So that the maximum rebar spacing is:

$$s_{max} := \frac{700 \cdot \gamma_c}{\beta_s f_{ss}} \text{-in} - 2d_c = 12.111 \text{-in} > 10 \text{-in} \quad \text{OK}$$

6. 22 ft bridge design using #4@10in

A summary of the demands for Strength I limit state on a 1ft strip for the 22ft span, according to Table 6, is as follows:

<table>
<thead>
<tr>
<th>Table 13</th>
<th>Location</th>
<th>Demand</th>
<th>Capacity provided by #4@10in Ubar</th>
</tr>
</thead>
<tbody>
<tr>
<td>$M_p_{total}$ (kip*ft/ft)</td>
<td>Span w/o key</td>
<td>6.46</td>
<td>N/A</td>
</tr>
<tr>
<td>$M_p_{total}$ (kip*ft/ft)</td>
<td>Span w/ key</td>
<td>4.98</td>
<td>9.79</td>
</tr>
<tr>
<td>$M_n_{total}$ (kip*ft/ft)</td>
<td>All beams</td>
<td>–1.77</td>
<td>–6.55</td>
</tr>
</tbody>
</table>
Compared with the 30ft bridge design, with the same reinforcement configuration and smaller demands (Table 6 and 7), the requirements of minimum reinforcement and cracking control reinforcement are definitely satisfied.

6.1 Development length

The development length for #4 bar is as determined before: $l_{dh} = 12\text{ in}$

For both the positive and negative moment reinforcement, this point where the development length begins is taken at the centerline of the beam.

6.2 Distribution reinforcement

For longitudinal bottom bar, Use #4@12 in (refer the 30ft bridge design)

6.3. Shrinkage and temperature reinforcement

As determined before, the minimum reinforcement area per foot, on each face and in each direction, shall be:

$$A_{s\_TS} = 0.11 \text{ in}^2/\text{ft}$$

This criteria can be met on each face and in each direction. The maximum spacing of the rebar shall not exceed either 3 times the deck depth or 18 in. For the top longitudinal bar, use #4@18 in. There is no bonding reinforcement needed as shown below:

From CONSPAN, at release, the maximum tension stress in the top fiber of the beam at the transfer cross section is $0.148\text{ksi}$, which is smaller than the limiting tensile stress of concrete $0.2\text{ksi}$

7. Design summary

7.1 Reinforcement configuration

<table>
<thead>
<tr>
<th></th>
<th>Development length (in)</th>
<th>Cut-off point (Interior beam)</th>
<th>Distribution bottom rebar (longitudinal)</th>
<th>Top rebar (longitudinal)</th>
</tr>
</thead>
<tbody>
<tr>
<td>40ft</td>
<td>#4@7 in</td>
<td>centerline of the beam</td>
<td>#4@10 in</td>
<td>#4@7 in (exterior beam)</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>#4@10 in (interior beam)</td>
</tr>
<tr>
<td>30ft</td>
<td>#4@10 in</td>
<td>centerline of the beam</td>
<td>#4@12 in</td>
<td>#4@16 in</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>Use #4@12</td>
</tr>
<tr>
<td>22ft</td>
<td>#4@10 in</td>
<td>centerline of the beam</td>
<td>#4@12 in</td>
<td>#4@18 in</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>Use #4@12</td>
</tr>
</tbody>
</table>

Note: bonded reinforcement needs to be provided within the tension zone of concrete at the transfer cross section at the time of release.
7.2 Criteria check

7.2.1 Demand VS Capacity

<table>
<thead>
<tr>
<th>Location</th>
<th>Demand</th>
<th>Capacity provided by #4@7\text{in} Ubar</th>
</tr>
</thead>
<tbody>
<tr>
<td>Mp\text{total} (kip*ft/ft)</td>
<td>40 ft span</td>
<td>40 ft span</td>
</tr>
<tr>
<td>Span w/o key</td>
<td>12.57</td>
<td>N/A</td>
</tr>
<tr>
<td>Span w/ key</td>
<td>11.36</td>
<td>13.59</td>
</tr>
<tr>
<td>M\text{n_total} (kip*ft/ft)</td>
<td>All beams</td>
<td>-3.49</td>
</tr>
<tr>
<td>---</td>
<td>---</td>
<td>---</td>
</tr>
<tr>
<td>---</td>
<td>---</td>
<td>---</td>
</tr>
<tr>
<td>30 ft span</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Mp\text{total} (kip*ft/ft)</td>
<td>Span w/o key</td>
<td>9.04</td>
</tr>
<tr>
<td>Span w/ key</td>
<td>7.81</td>
<td>9.79</td>
</tr>
<tr>
<td>M\text{n_total} (kip*ft/ft)</td>
<td>All beams</td>
<td>-2.65</td>
</tr>
<tr>
<td>---</td>
<td>---</td>
<td>---</td>
</tr>
<tr>
<td>---</td>
<td>---</td>
<td>---</td>
</tr>
<tr>
<td>22 ft span</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Mp\text{total} (kip*ft/ft)</td>
<td>Span w/o key</td>
<td>6.46</td>
</tr>
<tr>
<td>Span w/ key</td>
<td>4.98</td>
<td>9.79</td>
</tr>
<tr>
<td>M\text{n_total} (kip*ft/ft)</td>
<td>All beams</td>
<td>-1.77</td>
</tr>
</tbody>
</table>

Note: the positive moment capacity in the deck span without shear key, due to the development length of the reinforcing steel, is larger than that in the deck span with shear key.

7.2.2 Minimum reinforcement check

<table>
<thead>
<tr>
<th>M+ rebar</th>
<th>40 ft span</th>
</tr>
</thead>
<tbody>
<tr>
<td>#4@7\text{in}</td>
<td>( M = 13.59 \text{kip}\cdot\text{ft} &gt; 9.056 \text{kip}\cdot\text{ft} ) <strong>OK</strong></td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>M- rebar</th>
<th>40 ft span</th>
</tr>
</thead>
<tbody>
<tr>
<td>#4@7\text{in}</td>
<td>(</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>M+ rebar</th>
<th>30 ft span</th>
</tr>
</thead>
<tbody>
<tr>
<td>#4@10\text{in}</td>
<td>( M = 9.79 \text{kip}\cdot\text{ft} &gt; 9.056 \text{kip}\cdot\text{ft} ) <strong>OK</strong></td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>M- rebar</th>
<th>30 ft span</th>
</tr>
</thead>
<tbody>
<tr>
<td>#4@10\text{in}</td>
<td>(</td>
</tr>
</tbody>
</table>
22 ft span

With the same rebar configuration with 30ft span and less demands, this requirement needs not to be checked.

The minimum reinforcement requirement can be satisfied at any cross section.

7.2.3 Cracking control check

40 ft span

\[ s_{\text{max}} = \frac{700 \cdot \gamma_e}{f_{\text{SS}}} - 2 \cdot d_c = 9.544 \text{ in} > 7 \text{ in} \quad \text{OK} \]

30 ft span

\[ s_{\text{max}} = \frac{700 \cdot \gamma_e}{f_{\text{SS}}} - 2 \cdot d_c = 10.01 \text{ in} > 10 \text{ in} \quad \text{OK} \]

22 ft span

With the same rebar configuration with 30ft span and less demands, this requirement needs not to be checked.
NEXT-8---Deck Design Outline:

1. Deck properties
2. 3d finite element method--- Loads and load effects
   2.1 Dead load:
   2.2 Live load
   2.3 Moment distribution
   2.4 Distribution strip width
   2.5 Load combination
   2.6 Design demands for a 1ft strip
3. Demands based on 1d AASHTO FEM
4. Moment capacity
   4.1 The moment capacity of the current U-bar configuration: #4@8in c2c
      4.1.1 Negative moment capacity
      4.1.2 Positive moment capacity
   4.2 Capacities provided by various rebar configurations:
5. 40 ft bridge design--- #4@5
   5.1 Development length
   5.2. Limits of reinforcement
      5.2.1 Maximum reinforcement
      5.2.2 Minimum reinforcement
   5.3 Distribution reinforcement
   5.4 Shrinkage and temperature reinforcement
   5.5 Control of cracking
      5.5.1 Check of the positive moment reinforcement
      5.5.2 Check of the negative moment reinforcement
6. 30ft bridge design ---- #4 @ 7in
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   6.2 Minimum reinforcement
      6.2.1 Positive moment
      6.2.2 Negative moment
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   6.4. Shrinkage and temperature reinforcement
   6.5 Control of cracking
6.5.1 Check of the positive moment reinforcement
6.5.2 Check of the negative moment reinforcement

7. 22ft bridge design using #4@7in
   7.1 Development length
   7.2 Distribution reinforcement
   7.3 Shrinkage and temperature reinforcement

8. Design summary
   8.1 Reinforcement configuration
   8.2 Criteria check
      8.2.1 Demand VS Capacity
      8.2.2 Minimum reinforcement check
      8.2.3 Cracking control check
1. Deck properties:

Cross section: NEXT-8

Area of a single beam cross section: \( A_8 := 1147.4 \text{in}^2 \)

Effective width of a single cross section (including shear key): \( b_8 := 8 \text{ft} \)

Structural deck depth: \( h := 8 \text{in} \)

Moment of inertia considered: \( I := \frac{1}{12} \cdot 1 \text{ft} \cdot h^3 = 512 \cdot \text{in}^4 \)

Section modulus: \( S_r := \frac{I}{h} = 128 \cdot \text{in}^3 \)

Deck top cover: \( \text{Cover}_t := 2.5 \text{in} \) AASHTO LRFD Table 5.12.3-1

Deck bottom cover: \( \text{Cover}_b := 1.0 \text{in} \) AASHTO LRFD Table 5.12.3-1

Reinforced concrete density: \( w_c := 150 \text{pcf} \)

Concrete compressive strength (final) \( f'_c := 6.5 \text{ksi} \)

Rebar Young's modulus: \( E_s := 29000 \text{ksi} \)

Reinforcement strength: \( f_y := 60 \text{ksi} \)

Bituminous wearing surface: \( w_{fws} := 140 \text{pcf} \) AASHTO LRFD Table 3.5.1-1

2. 3d finite element method--- Loads and load effects:

2.1 Dead load:

DC :

parapet self weight : \( w_p := 443 \text{plf} \)

Parapet self weight is uniformly distributed to the outer two beams. Therefore for each beam mentioned above,

\[
\frac{w_b}{b_8^2} = 0.028 \cdot \frac{\text{kip}}{\text{ft}^2} \quad \text{where,} \quad b_8 = 8 \text{ft}
\]

Beam self weight:

\[
\frac{w_g}{b_8} = 0.149 \cdot \frac{\text{kip}}{\text{ft}^2} \quad \text{where,} \quad A_8 = 1.147 \times 10^3 \cdot \text{in}^2 \quad w_c = 150 \cdot \text{pcf}
\]
Wearing surface (4in on average):

\[
DW := (4\text{in} \cdot w_{fws}) = 0.047 \frac{\text{kip}}{\text{ft}^2} \quad \text{where,} \quad w_{fws} = 140 \text{pcf}
\]

The load from wearing surface is distributed from face to face of the two parapets.

In the finite element model, the above area loads are distributed to the shell element of the slab. For the frame element of the shear key, the area load is converted to uniformly distributed line load to each element. An example is provided for the 40ft-span NEXT8 bridge.

Bridge span length: \( l := 40\text{ft} \)

Number of frame elements for each shear key: \( n := 81 \)

Parapet self weight is distributed to the outer two shear keys:

\[
w_{bk} := \frac{w_b}{n} = 0.014 \frac{\text{kip}}{\text{ft}} \quad \text{where,} \quad w_b = 0.028 \frac{\text{kip}}{\text{ft}^2}
\]

Beam self weight:

\[
w_{gk} := \frac{w_g}{n} = 0.074 \frac{\text{kip}}{\text{ft}} \quad \text{where,} \quad w_g = 0.149 \frac{\text{kip}}{\text{ft}^2}
\]

Wearing surface load:

\[
w_{dwk} := \frac{DW}{n} = 0.023 \frac{\text{kip}}{\text{ft}} \quad \text{where,} \quad DW = 0.047 \frac{\text{kip}}{\text{ft}^2}
\]

Simulation results show insignificant influence of the loads from the shear key on demands.

2.2 Live load

LRFD Art 3.6.1.3.3 specifies that when the refined methods are used to analyze decks, if the slab spans primarily in the transverse direction, only the axles of the design truck of Article 3.6.1.2.2 or design tandem of Article 3.6.1.2.3 shall be applied to the deck slab.

In order to obtain the most critical demand, a single design tandem specified in LRFD Art 3.6.1.2.3 is applied to the finite element model, moving across the bridge model transversely and longitudinally. The position of the tandem follows that specified in LRFD Art. 3.6.1.3: the design tandem shall be positioned transversely such that the center of any wheel load is not closer than: For the design of components other than deck overhang—2 ft from the edge of the design lane. In this case, the axle load is positioned no closer than 2ft from the face of the parapet.
2.3 Moment distribution

The total unfactored transverse moment demand on the deck for the 40ft span bridge resulting from the dead loads and live loads are displayed as follows, in which the black lines represent the location of the shear key:

![Moment effects of dead loads](image1)

**Figure F.20: Moment effects of dead loads**

![Moment effects of design tandem](image2)

**Figure F.21: Moment effects of design tandem**

2.4 Distribution strip width:

The distribution width for the design tandem is 10ft
2.5 Load combination

LRFD Table 3.4.1-1

strength I limit state:
Maximum Q = 1.25(DC)+1.50(DW)+1.75(LL+IM)
Minimum Q = 0.90(DC)+0.65(DW)+1.75(LL+IM)

service I limit state:
Q = 1(DC)+1(DW)+1(LL+IM)

where
LL = live load effect including multiple presence factor 1.2
IM = dynamic load allowance IM := 1.33 LRFD Table 3.6.2.1-1

2.6 Design demands for a 1ft strip:

The total dead load effect is divided by the total span length to get the normalized demand on a 1ft strip. And the live load effect is divided by the strip width 10ft to get the normalized demand on the 1ft strip. The resulting normalized transverse moment demands combined for Strength I and Service I limit states for the 40ft span are displayed below. Positive moment means the bottom deck fiber is in tension.

Figure F.22: Demand distribution in Strength I limit state
A summary of the demands on a 1ft strip for the 22ft, 30ft, and 40ft spans are listed in Table 1 and Table 2:

![Figure F.23: Demand distribution in Service I limit state](image)

Table 1: Strength I limit state

<table>
<thead>
<tr>
<th>Location</th>
<th>22 ft</th>
<th>30 ft</th>
<th>40 ft</th>
</tr>
</thead>
<tbody>
<tr>
<td>M+ (kip*ft/ft)</td>
<td>8.87</td>
<td>14.25</td>
<td>20.65</td>
</tr>
<tr>
<td>3 ft span</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>M+ (kip*ft/ft)</td>
<td>7.00</td>
<td>11.31</td>
<td>17.12</td>
</tr>
<tr>
<td>5 ft span</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>M- (kip*ft/ft)</td>
<td>-5.56</td>
<td>-6.70</td>
<td>-8.45</td>
</tr>
<tr>
<td>Exterior beam</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>M- (kip*ft/ft)</td>
<td>-4.52</td>
<td>-4.19</td>
<td>-4.34</td>
</tr>
<tr>
<td>Interior beam</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>
### Table 2

<table>
<thead>
<tr>
<th>Location</th>
<th>22 ft</th>
<th>30 ft</th>
<th>40 ft</th>
</tr>
</thead>
<tbody>
<tr>
<td>M+ (kip*ft/ft) 3 ft span</td>
<td>5.10</td>
<td>8.29</td>
<td>12.08</td>
</tr>
<tr>
<td>M+ (kip*ft/ft) 5 ft span</td>
<td>4.06</td>
<td>6.62</td>
<td>10.04</td>
</tr>
<tr>
<td>M- (kip*ft/ft) Exterior beam</td>
<td>-3.25</td>
<td>-3.94</td>
<td>-4.98</td>
</tr>
<tr>
<td>M- (kip*ft/ft) Interior beam</td>
<td>-2.61</td>
<td>-2.34</td>
<td>-2.15</td>
</tr>
</tbody>
</table>

Notice that within the 3ft span, the reinforcement is doubled by the development length, which will be considered in checking the moment capacity and cracking control requirement.

3. Demands based on 1d AASHTO FEM:

The final design demands are calculated based on the results from 1d AASHTO FEM using a finite element software SAP2000. In the FEM, the deck (including shear key) is modeled as a continuous beam using frame elements and the stems are modeled as rigid supports. Refer to the deck design guideline in Chapter 9.2.1 for details.

From the 1d AASHTO FEM, the maximum positive moment and critical negative moment (unfactored) resulting from a single 32kip axle of design truck as specified in LRFD Article 3.6.1.2.2 are:

\[
M_p_{\text{total}} := 156.73 \text{kip-in} \quad M_n_{\text{total}} := -95.11 \text{kip-in}
\]

The strip widths for positive and negative moment for cast-in-place deck without stay-in-place concrete formwork are: LRFD Table 4.6.2.1.3-1

\[
M_p_{\text{width}} = 26 + 6.6 \cdot s_{\text{design}} \quad M_n_{\text{width}} = 48 + 3 \cdot s_{\text{design}}
\]

where,

\[s_{\text{design}} = \text{average stem spacing (ft)} \quad s_{\text{design}} := 4\]

Therefore,

\[
M_p_{\text{width}} := (26 + 6.6 \cdot s_{\text{design}}) \text{in} = 52.4 \text{in}
\]

\[
M_n_{\text{width}} := (48 + 3 \cdot s_{\text{design}}) \text{in} = 60 \text{in}
\]

The normalized demands for a 1ft load strip are:

\[
M_p_{\text{AASHTO}} := \frac{M_p_{\text{total}}}{M_p_{\text{width}}} = 2.991 \text{kip-ft/ft}
\]
Based on the formula proposed, the normalized design demands are:

Positive design moment:

For the 3ft span: \[ M_{\text{positive}} = M_{p,\text{AASHTO}} \left[ 0.77 + 0.0027 \left( l_{\text{design}}^{1.3} s_{\text{design}}^{1.4} - 244 \right) \right] \]

For the 5ft span: \[ M_{\text{positive}} = M_{p,\text{AASHTO}} \left[ 0.58 + 0.0196 \left( l_{\text{design}}^{1} s_{\text{design}}^{0.8} - 51 \right) \right] \]

Negative design moment:

For the exterior beam: \[ M_{\text{negative}} = M_{n,\text{AASHTO}} \left[ 0.4 + 6.28 \left( l_{\text{design}}^{0.1} s_{\text{design}}^{0.2} - 1.69 \right) \right] \]

For the interior beam: \[ M_{\text{negative}} = M_{n,\text{AASHTO}} \]

The negative moment demand in the interior beam shall not exceed that in the exterior beam.

where,

\[ l_{\text{design}} = \text{design span length, (ft)} \]

\[ l_{\text{design}} = 39\text{ft for the 40ft span bridge} \]

\[ M_{\text{positive}} = \text{unfactored positive moment demand from live load (kip*ft/ft)} \]

\[ M_{\text{negative}} = \text{unfactored negative moment demand from live load (kip*ft/ft)} \]

Based on the formula, the normalized live load demands (kip*ft/ft) are:

<table>
<thead>
<tr>
<th>Table 3</th>
<th>Location</th>
<th>22 ft</th>
<th>30 ft</th>
<th>40 ft</th>
</tr>
</thead>
<tbody>
<tr>
<td>( M_{\text{positive}} ) (kip*ft/ft)</td>
<td>3 ft span</td>
<td>3.28</td>
<td>4.81</td>
<td>6.92</td>
</tr>
<tr>
<td>( M_{\text{positive}} ) (kip*ft/ft)</td>
<td>5 ft span</td>
<td>2.48</td>
<td>3.90</td>
<td>5.68</td>
</tr>
<tr>
<td>( M_{\text{negative}} ) (kip*ft/ft)</td>
<td>Exterior beam</td>
<td>−1.62</td>
<td>−2.20</td>
<td>−2.76</td>
</tr>
<tr>
<td>( M_{\text{negative}} ) (kip*ft/ft)</td>
<td>Interior beam</td>
<td>−1.59</td>
<td>−1.59</td>
<td>−1.59</td>
</tr>
</tbody>
</table>

The dead load effect is obtained based on the 1d AASHTO FEM. The moment demands from beam self weight, barrier self weight, and wearing surface are combined for the Strength I limit state and Service I limit state. For each limit state, both the maximum and minimum combined moment demands are obtained and listed in Table 4.
The total load effect is calculated as:

\[ M_{p,\text{total}} = M_{\text{positive}} \cdot IM \cdot m \cdot LF + M_{p,\text{DL}} \]
\[ M_{n,\text{total}} = M_{\text{negative}} \cdot IM \cdot m \cdot LF + M_{n,\text{DL}} \]

where,

- \( M_{p,\text{total}} \) = final design positive moment demand for either Strength I or Service I limit state (kip*ft/ft)
- \( M_{n,\text{total}} \) = final design negative moment demand for either Strength I or Service I limit state (kip*ft/ft)
- \( M_{\text{positive}} \) = unfactored positive moment demand from live load (kip*ft/ft)
- \( M_{\text{negative}} \) = unfactored negative moment demand from live load (kip*ft/ft)
- \( M_{p,\text{DL}} \) = factored positive moment demand from dead load (kip*ft/ft)
- \( M_{n,\text{DL}} \) = factored negative moment demand from dead load (kip*ft/ft)
- \( IM \) = dynamic load allowance percent, \( IM = 1.33 \) LRFD Table 3.6.2.1-1
- \( m \) = multiple presence factor, \( MP = 1.2 \) LRFD Table 3.6.1.1.2-1
- \( LF \) = live load factor, \( LL = 1.75 \) for Strength I limit state, and \( 1 \) for Service I limit state LRFD Table 3.4.1-1

The final design demands for Strength I and Service I limit states are listed in Table 5 and 6. The values given in bracket are the demands based on 3d FEM results. The percentage errors for the unconservative demands are also given by taking the 3d FEM results as the 'correct' values. The unconservative values are all within six percentage of the FEM results.
4. Moment capacity

4.1 The moment capacity of the U-bar configuration used in experiments: #4@8in c2c

Rebar diameter: $d_4 := 0.5\text{in}$

Area of top layer of rebar: $A'_s := 0.29\text{in}^2/\text{ft}$

Area of bottom layer of rebar: $A_s := A'_s$

Distance from the bottom beam fiber to the centroid of the top bar:

$d' := h - \text{Cover}_t - 0.5\cdot d_4 = 5.25\text{in}$ where, $\text{Cover}_t = 2.5\text{in}$ $h = 8\text{in}$

Distance from the top beam fiber to the centroid of the bottom bar:

$d := h - \text{Cover}_b - 0.5\cdot d_4 = 6.75\text{in}$ where, $\text{Cover}_b = 1\text{in}$

$\beta_1 := 0.85 - \left(\frac{f'_c - 4000\text{psi}}{1000\text{psi}}\right)0.05 = 0.725$ where, $f'_c = 6.5\text{ksi}$ LRFD Article 5.7.2.2

4.1.1 Negative moment capacity

For negative moment capacity, assume both sides of bars are in tension and yield

$A_s\cdot f_y + A'_s\cdot f'_y = 0.85\cdot f'_c\cdot \beta_1 c_o$

$c_o := \frac{1.17647055822352941176\cdot(A_s\cdot f_y + A'_s\cdot f'_y)}{f'_c\cdot \beta_1} = 0.724\text{in}$

$h - d = 1.25\text{in}$ Therefore both layers of bars are in tension
Strain in the bottom layer of rebar:

\[ \varepsilon := \frac{c_o - (h - d)}{c_o} \cdot 0.003 = 2.18 \times 10^{-3} > \frac{f_y}{E_s} = 2.069 \times 10^{-3} \]

Strain in the top layer of rebar:

\[ \varepsilon := \frac{c_o - d'}{c_o} \cdot 0.003 = 0.019 > 0.005 \]

Therefore cross section is tension-controlled \( \text{LRFD Article 5.7.2.1} \)

Therefore both layers of rebars yield. Assumption is valid

\[ a_o := \beta_1 \cdot c_o = 0.525 \cdot \text{in} \quad \text{where,} \quad \beta_1 = 0.725 \]

Negative moment capacity:

\[ \phi M_n = -\phi \left[ A_s f_y \left( h - d - \frac{a_o}{2} \right) + A'_s f_y \left( d' - \frac{a_o}{2} \right) \right] \]

where

\[ \phi = \text{resistance factor} \quad \text{LRFD Art. 5.5.4.2.1} \]

\[ \phi := 0.9 \quad \text{tension controlled reinforced concrete sections} \]

\[ \phi M_n := -\phi \left[ A_s f_y \left( h - d - \frac{a_o}{2} \right) + A'_s f_y \left( d' - \frac{a_o}{2} \right) \right] = -7.798 \frac{\text{kip-ft}}{\text{ft}} \]

4.1.2 Positive moment capacity

Assume both sides of bars are in tension and yield

\[ A_s f_y + A'_s f_y = 0.85 f'_c \beta_1 c_o \]

\[ c_o := \frac{1.1764705882352941176 \cdot \left( A_s f_y + A'_s f_y \right)}{f'_c \beta_1} = 0.724 \cdot \text{in} \]

\[ h - d' = 2.75 \cdot \text{in} \quad \text{Therefore both layers of bars are in tension} \]
Strain in the top layer of rebar:

\[ \varepsilon := -\frac{c_o - (h - d')}{c_o} \cdot 0.003 = 8.395 \times 10^{-3} > \frac{f_y}{E_s} = 2.069 \times 10^{-3} \]

where, \( d' = 5.25 \text{ in} \)

Strain in the bottom layer of rebar:

\[ \varepsilon := -\frac{c_o - d}{c_o} \cdot 0.003 = 0.025 > 0.005 \quad \text{where,} \quad d = 6.75 \text{ in} \]

Cross section is tension-controlled

Therefore both layers of rebars yield. Assumption is valid

\[ a_o := \beta_1 c_o = 0.525 \text{ in} \]

Positive moment capacity:

\[ \phi M_n := \phi \left[ A_s f_y \left( h - d' - \frac{a_o}{2} \right) + A_s f_y \left( d - \frac{a_o}{2} \right) \right] = 11.713 \frac{\text{kip} \cdot \text{ft}}{\text{ft}} \]

4.2 Capacities provided by various rebar configurations:

Like the calculation procedures above, several different rebar configurations are considered. The capacities are provided below:

<table>
<thead>
<tr>
<th>Table 7</th>
<th>#4@9in</th>
<th>#4@8in</th>
<th>#4@7in</th>
<th>#4@6in</th>
<th>#4@5in</th>
</tr>
</thead>
<tbody>
<tr>
<td>M+ (kip*ft/ft)</td>
<td>10.56</td>
<td>11.71</td>
<td>13.59</td>
<td>15.43</td>
<td>18.29</td>
</tr>
<tr>
<td>M- (kip*ft/ft)</td>
<td>-7.05</td>
<td>-7.80</td>
<td>-8.90</td>
<td>-9.87</td>
<td>-11.47</td>
</tr>
</tbody>
</table>

5. 40 ft bridge design--- #4@5

The normalized moment demands in strength I limit state, according to Table 5, is:

<table>
<thead>
<tr>
<th>Table 8</th>
<th>Location</th>
<th>Demand</th>
<th>Capacity provided by #4@5in Ubar</th>
</tr>
</thead>
<tbody>
<tr>
<td>( M_{p_{total}} ) (kip*ft/ft)</td>
<td>3 ft span</td>
<td>21.18</td>
<td>N/A</td>
</tr>
<tr>
<td>( M_{p_{total}} ) (kip*ft/ft)</td>
<td>5 ft span</td>
<td>17.72</td>
<td>18.29</td>
</tr>
<tr>
<td>( M_{n_{total}} ) (kip*ft/ft)</td>
<td>Exterior beam</td>
<td>-8.42</td>
<td>-11.47</td>
</tr>
<tr>
<td>( M_{n_{total}} ) (kip*ft/ft)</td>
<td>Interior beam</td>
<td>-5.15</td>
<td>-11.47</td>
</tr>
</tbody>
</table>
For the 3ft-span deck —which is the deck in-between the two stems of a beam— due to the development length of the rebar, the actual capacity is larger than that in the 5ft-span deck.

5.1 Development length

Rebar area: \( A_4 := 0.2 \text{in}^2 \)

Rebar diameter: \( d_4 := 0.5 \text{in} \)

**Straight development:**

\[
l_{hb} := \max \left( \frac{1.25 \cdot A_4 \cdot f_y \cdot \text{in}}{0.4 \cdot \frac{f'_c \cdot \text{kip}}{\text{ksi}}} \right) = 12 \text{ in} \quad \text{where,} \quad f'_c = 6.5 \text{ksi} \quad f_y = 60 \text{ksi}
\]

\[
f_{modS} := 1
\]

\[
l_{dh} := \max \left( f_{modS} l_{hb}, 12 \text{ in} \right) = 12 \text{ in}
\]

According to LRFD Art. 5.11.2.1, except at supports of simple spans and at the free ends of cantilevers, reinforcement shall be extended beyond the point at which it is no longer required to resist flexure for a distance calculated above.

Interior beam:

For the positive reinforcement, since additional moment capacity is needed within the 3ft deck but not within the stem, this point is taken at the inner face of each stem.

For the negative reinforcement, the point where the development length begins is taken at the centerline of the beam. The development length is at least 12in as determined above. In the design, the two legs of the U-bar will be made of the same length.

5.2. Limits of reinforcement

5.2.1 Maximum reinforcement

The check of maximum reinforcement limits was removed from the LRFD Specifications in 2005.  

LRFD Art. 5.7.3.3.1

5.2.2 Minimum reinforcement

At any section of a noncompression-controlled flexural component, the amount of prestressed and nonprestressed tensile reinforcement shall be adequate to develop a factored flexural resistance, \( M_r \), at least equal to the lesser of:

1. 1.33 times the factored moment required by the applicable strength load combination specified in Table 3.4.1-1 (Strength I); and

2. \( M_{cr} = \gamma_3 (\gamma_1 f_y) S_f \)  

LRFD Eq. 5.7.3.3.2-1
The above equation is a simplified form of LRFD Equation 5.7.3.3.2-1 because no composite section exists, therefore the composite and noncomposite section modulus are the same. Also since there is no transverse post-tensioning, the cracking moment capacity won't be increased.

where,

\[ f_r = \text{modulus of rupture of concrete} \quad \text{LRFD Art 5.4.2.6} \]

\[ f_r = \frac{f'_c \times 0.37}{\text{ksi}} = 0.943 \text{ ksi} \]

\[ S_r = \text{section modulus for the extreme fiber of the section where tensile stress is caused by externally applied loads (in}^3) \]

\[ S_r = 128 \text{ in}^3 \]

\[ \gamma_1 = \text{flexural cracking variability factor} \]

\[ \gamma_1 = 1.2 \quad \text{for precast segmental structures} \]

\[ \gamma_3 = \text{ratio of specified minimum yield strength to ultimate tensile strength of the reinforcement} \]

\[ \gamma_3 = 0.75 \quad \text{for A 706, Grade 60 reinforcement} \]

Therefore,

\[ M_{cr} := \gamma_3 \left( \gamma_1 \cdot f_r \right) S_r = 9.056 \text{-kip-ft} \]

The above \( M_{cr} \) applies to any cross section in the deck.

5.2.2.1 Positive moment

The factored positive moment required by strength I load combination within the 5ft deck for a 1ft strip, according to Table 8, is:

\[ M_u := 17.72 \text{kip-ft} \]

Thus,

\[ 1.33 \times M_u = 23.568 \text{-kip-ft} > M_{cr} = 9.056 \text{-kip-ft} \]

Therefore \( M_{cr} \) requirement controls.

The factored capacity provided by #4@5in is:

\[ M_r := 18.29 \text{kip-ft} \]

\[ M_r > M_{cr} = 9.056 \text{-kip-ft} \quad \text{OK} \]

This criteria can be met at every deck section.

5.2.2.2 Negative moment

5.2.2.2.1 Interior beam
The factored negative moment required by strength I load combination for the interior beam, according to Table 8, is:

\[ M_u := -5.15 \text{kip-ft} \]

Thus, \[ 1.33 \cdot M_u = 6.849 \text{-kip-ft} < M_{cr} = 9.056 \text{-kip-ft} \]

Therefore \[ 1.33 \cdot M_u \] requirement controls.

The factored negative moment resistance provided by #4@5in is:

\[ M_r = 11.47 \text{kip-ft} \]

Thus, \[ 1.33 \cdot M_u = 6.849 \text{-kip-ft} \] \( \text{OK} \)

5.2.2.2 Exterior beam

The factored negative moment required by strength I load combination for the exterior beam, according to Table 8, is:

\[ M_u := -8.42 \text{kip-ft} \]

Thus, \[ 1.33 \cdot M_u = 11.199 \text{-kip-ft} > M_{cr} = 9.056 \text{-kip-ft} \]

Therefore \[ M_{cr} \] requirement controls.

The least negative moment capacity comes from the rebar configuration of #4 @5in

\[ M_r = 11.47 \text{kip-ft} > M_{cr} = 9.056 \text{-kip-ft} \text{ OK} \]

Therefore this criteria can be met at every deck section.

5.3 Distribution reinforcement

The required area of secondary reinforcement at the bottom of the deck is a percentage of the primary positive moment reinforcement. For primary reinforcement perpendicular to traffic, LRFD Art. 9.7.3.2 specifies that the percentage should be:

\[
\text{Percentage} = \frac{220}{\sqrt{S}} \leq 67\
\]

where

\[ S = \text{the effective span length (ft)} \]

LRFD Art. 9.7.2.3

\[ S := 5 \text{ft} - 15 \text{-in} = 45 \text{-in} \]

Compared with the 3ft span, the 5ft span gives less percentage

\[
\text{Percentage} := \frac{220}{\sqrt{S}} = 113.608\% \text{ use 67}\%
\]
For both the Interior beam and exterior beam:

Positive moment reinforcement in the transverse direction (#4@5in): \( A_s := \frac{0.47}{\text{ft}} \)

\[ A_s \cdot 67\% = 0.315 \frac{\text{in}^2}{\text{ft}} \]

For longitudinal bottom bar, Use #4@7in, which gives \( 0.34 \frac{\text{in}^2}{\text{ft}} \)

5.4. Shrinkage and temperature reinforcement

The minimum reinforcement area per foot, on each face and in each direction, shall satisfy:

\[ A_{s,TS} \geq \frac{1.3 \cdot b \cdot h}{2 \cdot (b + h) \cdot f_y} \]

LRFD Eq. 5.10.8-1

where

\( A_{s,TS} = \) area of reinforcement in each direction and each face (in²/ft)

\( b = \) least width of component section (in) \( b := 1\text{ft} \)

\( h = \) least thickness of component section (in) \( h = 8\text{ in} \)

\( f_y = \) specified yield strength of reinforcing bars \( f_y = 60\text{ksi} \leq 75\text{ksi} \)

Therefore

\[ A_{s,TS} \geq \frac{1.3 \cdot 12 \cdot 8}{2 \cdot (12 + 8) \cdot 60} \frac{\text{in}^2}{\text{ft}} = 0.052 \frac{\text{in}^2}{\text{ft}} \]

Use \( A_{s,TS} := 0.11 \frac{\text{in}^2}{\text{ft}} \)

LRFD Eq. 5.10.8-2

This requirement can be satisfied in each direction and each face.

Since the deck depth is more than 6in, the shrinkage and temperature rebars need to provided equally on both layers. The maximum spacing of the rebar should not exceed either 3 times the deck depth or 18in. The top longitudinal bars are provided by the bonded reinforcement, which is #4@7in for both interior and exterior beams as calculated below:

From CONSPAN, at release, the tension stress in the top fiber of the interior beam at the transfer cross section is:

\( f_t := -0.472 \text{ksi} \)

The compressive stress in the bottom fiber of the beam at the same cross section is:
The depth of the tensile zone $x$ is:

$$ x := \frac{-f_t \cdot \text{in}}{f_b - f_t} = 3.087 \text{ in} < 8 \text{ in} $$

The tensile force $T$ in the concrete is:

$$ T := \frac{f_t}{2} \cdot b_8 \cdot x $$

where, $b_8$ is the width of the beam at top $b_8 = 96 \text{ in}$

Therefore

$$ T := \frac{f_t}{2} \cdot b_8 \cdot x = 69.937 \text{ kip} $$

The required area of bonded reinforcement is:

$$ A_{\text{req}} = \frac{T}{f_s} $$

where $f_s := 0.5 \cdot f_y = 30 \text{ ksi}$

Therefore

$$ A_{\text{req}} := \frac{T}{f_s \cdot b_8} = 0.291 \text{ in}^2 \text{ ft} $$

Use #4 @ 7 in within the tensile zone $A_s := 0.34 \text{ in}^2 \text{ ft} > A_{\text{req}}$ OK

From CONSPAN, at release, the tension stress in the top fiber of the exterior beam at the transfer cross section is:

$f_t := -0.53 \text{ ksi}$

The compressive stress in the bottom fiber of the beam at the same cross section is:

$f_b := 3.117 \text{ ksi}$

The depth of the tensile zone $x$ is:

LRFD C5.9.4.1.2
The tensile force $T$ in the concrete is:

$$T := \frac{f_t}{2} b_8 x$$

where, $b_8$ is the width of the beam at top \[ b_8 = 96\text{ in} \]

Therefore

$$T := \frac{f_t}{2} b_8 x = 77.638\text{ kip}$$

The required area of bonded reinforcement is:

$$A_{req} = \frac{T}{f_s}$$

where \[ f_s := 0.5 f_y = 30\text{ ksi} \]

LRFD C5.9.4.1.2

Therefore

$$A_{req} := \frac{T}{f_s b_8} = 0.323\text{ in}^2/\text{ft}$$

Use #4 @ 7in within the tensile zone \[ A_s := 0.34\text{ in}^2/\text{ft} > A_{req} \text{ OK} \]

5.5 Control of cracking LRFD Art. A.5.7.3.4

In the longitudinal direction, due to the existence of prestress strands, cracking is assumed to not happen. Therefore only in the transverse direction, cracking is considered.

The spacing of mild steel reinforcement in the layer closest to the tension face shall satisfy the following:

$$s \leq \frac{700 \gamma_e}{\beta_s f_{ss}} - 2d_c$$

in which, \[ \beta_s = 1 + \frac{d_c}{0.7(h - d_c)} \]

where

$\gamma_e$ = exposure factor

= 1.00 for Class 1 exposure condition

= 0.75 for Class 2 exposure condition

$\gamma_e := 0.75$ for deck SCDOT Bridge Design Manual 15.1.7

$d_c$ = thickness of concrete cover measured from extreme tension fiber to center of the flexural reinforcement located closest thereto (in)

$f_{ss}$ = tensile stress in steel reinforcement at the service limit state (ksi)

$h$ = overall thickness or depth of the component (in) \[ h = 8\text{ in} \]

300
The load effects determined for the Service I limit state for the NEXT-8 40ft span, according to Table 6, is displayed below.

Table 9

<table>
<thead>
<tr>
<th>Location</th>
<th>Demand</th>
</tr>
</thead>
<tbody>
<tr>
<td>$M_p_{\text{total}}$ (kip*ft/ft) 3 ft span</td>
<td>12.47</td>
</tr>
<tr>
<td>$M_p_{\text{total}}$ (kip*ft/ft) 5 ft span</td>
<td>10.50</td>
</tr>
<tr>
<td>$M_n_{\text{total}}$ (kip*ft/ft) Exterior beam</td>
<td>−4.97</td>
</tr>
<tr>
<td>$M_n_{\text{total}}$ (kip*ft/ft) Interior beam</td>
<td>−3.10</td>
</tr>
</tbody>
</table>

The section is transformed elastic, cracked cross section. LRFD Art. 5.7.1 modulus of elasticity, ksi = 

$$E_c := 33000 \cdot K_1 \cdot w_c^{1.5} \cdot \sqrt{f_c'}$$

where

$K_1 = \text{correction factor for source of aggregate}: \quad K_1 := 1$

$w_c = \text{unit weight of concrete(kcf)}: \quad w_c = 150 \cdot \text{pcf}$

This unit weight is higher than what is given in LRFD Table 3.5.1-1. It is to be used for deck design unless more precise information is provided.

$f_c' = \text{specified compressive strength of concrete, ksi} \quad f_c' = 6.5 \cdot \text{ksi}$

Therefore, the modulus of elasticity:

$$E_c := 33000 \cdot K_1 \left( \frac{w_c}{1000 \text{pcf}} \right)^{1.5} \cdot \sqrt{f_c'} \, \text{ksi} = 4.888 \times 10^3 \, \text{ksi}$$

Modulus ratio:

$$n_c := \frac{E_s}{E_c} = 5.933 \quad \text{where}, \quad E_s = 2.9 \times 10^4 \, \text{ksi}$$

5.5.1 Check of the positive moment reinforcement

For the 5ft deck with #4@5in, according to Table 9, $M_{\text{pos}} := 10.5 \frac{\text{kip-ft}}{\text{ft}}$

5.5.1.1 Cracked moment of inertia

Assume both layers of bars are in tension.

For a 1ft cross section, $b := 1\text{ft}$

Bottom layer of rebar: $A_s := 0.47 \frac{\text{in}^2}{\text{ft}}$
Top layer of rebar: \( A_s' := 0.47 \text{ in}^2 / \text{ft} \)

Distance from center of bottom rebar to the top fiber of the beam is:
\[
d := h - \text{Cover}_b - \frac{d_4}{2} = 6.75 \text{-in}
\]

Distance from center of top rebar to the bottom fiber of the beam is:
\[
d' := h - \text{Cover}_t - \frac{d_4}{2} = 5.25 \text{-in}
\]

The location of neutral axis \( x \) measured from the top fiber of the beam is determined as below:

Sum of statical moments about the neutral axis gives:
\[
\frac{1}{2} \cdot x^2 = n_c \cdot A_s' \cdot (d - x) + n_c \cdot A_s' \cdot (h - d' - x)
\]
\[
x := \sqrt{n_c \left( A_s^2 n_c + A_s' \frac{2}{3} n_c + 2 \cdot A_s \cdot d - 2 \cdot A_s' \cdot d' + 2 \cdot A_s' \cdot h + 2 \cdot A_s' \cdot A_s' \cdot n_c \right) - A_s' \cdot n_c - A_s' \cdot n_c = 1.687 \text{-in}}
\]

The cracked moment of inertia is:
\[
I_{cr} := \frac{b \cdot x}{3} + n_c \cdot A_s \cdot b \cdot (d - x)^2 + n_c \cdot A_s' \cdot b \cdot (h - d' - x)^2 = 93.839 \text{-in}^4
\]

5.5.1.2 Tensile stress in the bottom steel

\[
f_{ss} := n_c \cdot \frac{M_{\text{pos}}}{I_{cr} \cdot b} \cdot (d - x) = 40.333 \text{-ksi}
\]

5.5.1.3 Rebar spacing

The thickness of concrete cover measured from extreme tension fiber to center of the flexural reinforcement located closest thereto (in) is:
\[
d_c := h - d = 1.25 \text{-in} \quad \text{where,} \quad h = 8 \text{-in}
\]

Therefore:
\[
\beta_s := 1 + \frac{d_c}{0.7 (h - d_c)} = 1.265
\]

So that the maximum rebar spacing is:
\[
s_{\text{max}} := \frac{700 \cdot \gamma_e \text{-in} - 2 \cdot d_c}{f_{ss} \frac{\gamma_e}{\text{ksi}}} = 7.793 \text{-in} > 5 \text{-in} \quad \text{OK} \quad \text{where,} \quad \gamma_e = 0.75
\]
5.5.2 Check of the negative moment reinforcement

Check the most critical negative moment against #4@5in

According to Table 9, \( M_{\text{neg}} = 4.97 \text{kip-ft} \)

5.5.2.1 Cracked moment of inertia

Assume both layers of bars are in tension.

For a 1ft cross section, \( b := 1 \text{ft} \)

Bottom layer of rebar: \( A_s := 0.47 \text{ in}^2 \)

Top layer of rebar: \( A'_s := 0.47 \text{ in}^2 \)

The location of neutral axis \( x \) measured from the bottom fiber of the beam is determined as below.

Sum of statical moments about the neutral axis gives:

\[
\frac{1}{2}x^2 = n_c A_s (h - d - x) + n_c A'_s (d' - x)
\]

where, \( n_c = 5.933 \quad d = 6.75 \text{ in} \quad d' = 5.25 \text{ in} \)

\[
x := \sqrt{n_c \left( A_s^2 n_c + A'_s^2 n_c - 2 A_s A'_s + 2 A_s A'_s d' + 2 A_s A'_s d + 2 A'_s A'_s h + 2 A_s A'_s n_c \right) - A_s n_c - A'_s n_c = 1.334 \text{ in}}
\]

The cracked moment of inertia is:

\[
I_{\text{cr}} := \frac{b x^3}{3} + n_c A_s b (h - d - x)^2 + n_c A'_s b (d' - x)^2 = 52.279 \text{ in}^4
\]

5.5.2.2 Tensile stress in the top steel

\[
f_{ss} := n_c \frac{M_{\text{neg}} b}{I_{\text{cr}}} (d' - x) = 26.503 \text{ ksi}
\]

5.5.2.3 Rebar spacing

The thickness of concrete cover measured from extreme tension fiber to center of the flexural reinforcement located closest thereto (in) is:

\( d_c := h - d' = 2.75 \text{ in} \)

Therefore:

\[
\beta_s := 1 + \frac{d_c}{0.7 (h - d_c)} = 1.748
\]
So that the maximum rebar spacing is:

\[ s_{\text{max}} = \frac{700 \cdot \gamma_e \cdot \beta_s \cdot f_{\text{ss}}}{\ln(2) \cdot d_c} = 5.83 \text{ in} \quad > 5 \text{ in} \quad \text{OK} \]

Therefore this requirement can be satisfied at any cross section.

6. 30ft bridge design--- #4 @ 7in

The normalized moment demands in strength I limit state, according to Table 5, is:

<table>
<thead>
<tr>
<th>Table 10 Capacity provided by #4@7in Ubar</th>
</tr>
</thead>
<tbody>
<tr>
<td>Location</td>
</tr>
<tr>
<td>----------</td>
</tr>
<tr>
<td>( M_{p _\text{total}} ) (kip*ft/ft)</td>
</tr>
<tr>
<td>( M_{p _\text{total}} ) (kip*ft/ft)</td>
</tr>
<tr>
<td>( M_{n _\text{total}} ) (kip*ft/ft)</td>
</tr>
<tr>
<td>( M_{n _\text{total}} ) (kip*ft/ft)</td>
</tr>
</tbody>
</table>

6.1 Development length

The development length for #4 bar is as determined before:

\[ l_{dh} = 12 \text{ in} \]

Interior beam:

For positive moment reinforcement, additional moment capacity is needed within the 3ft deck. Therefore the beginning point of the development length is taken at the inner face of each stem.

For negative moment reinforcement, the point where the development length begins is taken at the centerline of the 3ft span between the two stems. The development length is 12 in as determined above. In the design, the two legs of the U-bar will be made of the same length.

6.2. Minimum reinforcement

LRFD Art. 5.7.3.3.1

At any section of a noncompression-controlled flexural component, the amount of prestressed and nonprestressed tensile reinforcement shall be adequate to develop a factored flexural resistance, \( M_r \), at least equal to the lesser of:

1. 1.33 times the factored moment required by the applicable strength load combination specified in Table 3.4.1-1 (Strength I); and

2. \( M_{cr} = 9.056 \text{ kip*ft} \)
6.2.1 Positive moment

The factored positive moment required by strength I load combination within the 5ft deck for a 1ft strip, according to Table 10, is:

\[ M_u := 12.75 \text{ kip-ft} \]

Thus, \[ 1.33 \cdot M_u = 16.957 \text{ kip-ft} > M_{cr} = 9.056 \text{ kip-ft} \]

Therefore \( M_{cr} \) requirement controls.

The factored capacity provided by #4@7in is:

\[ M_r := 13.59 \text{ kip-ft} > M_{cr} = 9.056 \text{ kip-ft} \quad \text{OK} \]

This criteria can be met at every deck section.

6.2.2 Negative moment

6.2.2.1 Interior beam

The factored negative moment required by strength I load combination for the interior beam, according to Table 10, is:

\[ M_u := -5.15 \text{ kip-ft} \]

Thus, \[ 1.33 \cdot M_u = 6.849 \text{ kip-ft} < M_{cr} = 9.056 \text{ kip-ft} \]

Therefore \( 1.33 \cdot M_u \) requirement controls.

The factored negative moment resistance provided by #4@7in is:

\[ |M_r| = 8.90 \text{ kip-ft} > 1.33 \cdot M_u = 6.849 \text{ kip-ft} \quad \text{OK} \]

6.2.2.2 Exterior beam

The factored negative moment required by strength I load combination for the exterior beam is:

\[ M_u := -6.88 \text{ kip-ft} \]

Thus, \[ 1.33 \cdot M_u = 9.15 \text{ kip-ft} > M_{cr} = 9.056 \text{ kip-ft} \]

Therefore \( M_{cr} \) requirement controls.

The factored negative moment capacity provided by #4@7in is:

\[ |M_r| = 8.90 \text{ kip-ft} < M_{cr} = 9.056 \text{ kip-ft} \]

Therefore additional bars in the overhang need to extend beyond the point of critical demand for a distance of its development length.

6.3 Distribution reinforcement
As determined before, the minimum percentage of bottom longitudinal reinforcement should be:

\[ \text{Percentage} := 67\% \]

For both the Interior beam and exterior beam

Positive moment reinforcement in the transverse direction (#4@7in):
\[ A_s := 0.34 \text{ in}^2 / \text{ft} \]
\[ A_s \times 67\% = 0.228 \text{ in}^2 / \text{ft} \]

For longitudinal bottom bar, Use #4@10in, which gives \( 0.24 \text{ in}^2 / \text{ft} \)

6.4. Shrinkage and temperature reinforcement

As determined before, the minimum reinforcement area per foot, on each face and in each direction, shall be:

\[ A_{s,T} := 0.11 \text{ in}^2 / \text{ft} \]

This criteria can be met on each face and in each direction. The top longitudinal bars are provided by the bonded reinforcement, which is #4 @10in for the exterior beam, and #4@12in for the interior beam. The calculation is provided here.

From CONSPAN, at release, the tension stress in the top fiber of the exterior beam at the transfer cross section is:

\[ f_t := -0.368 \text{ksi} \]

The compressive stress in the bottom fiber of the beam at the same cross section is:

\[ f_b := 2.125 \text{ksi} \]

Refer to Figure F.24, the depth of the tensile zone \( x \) is:

\[ LRFD \ C5.9.4.1.2 \]

\[ \frac{-f_t}{x} = \frac{f_b}{21\text{in} - x} \]

\[ x := \frac{21 \cdot f_t \text{in}}{f_b - f_t} = 3.1 \text{in} \quad < \quad 8\text{in} \]

The tensile force \( T \) in the concrete is:

\[ T := \frac{f_t}{2} \cdot b_8 \cdot x \]

where, \( b_8 \) is the width of the beam at top

\[ b_8 = 96 \text{ in} \]

Therefore

\[ T := \frac{-f_t}{2} \cdot b_8 \cdot x = 54.756 \text{ kip} \]

The required area of bonded reinforcement is:

\[ A_{req} = \frac{T}{f_s} \]
where \( f_s := 0.5f_y = 30\text{ ksi} \) \hspace{1cm} \text{LRFD C5.9.4.1.2}

Therefore \[ A_{req} := \frac{T}{f_s b_8} = 0.228 \text{ in}^2 / \text{ft} \]

Use #4 @ 10in within the tensile zone \( \Lambda_s := 0.24 \text{ in}^2 / \text{ft} > \Lambda_{req} \hspace{1cm} \text{OK} \)

From CONSPAN, at release, the tension stress in the top fiber of the \textit{interior beam} at the transfer cross section is:

\( f_t := -0.314\text{ksi} \)

The compressive stress in the bottom fiber of the beam at the same cross section is:

\( f_b := 1.889\text{ksi} \)

The depth of the tensile zone \( x \) is: \hspace{1cm} \text{LRFD C5.9.4.1.2}

\[ \frac{-f_t}{x} = \frac{f_b}{21\text{in} - x} \]

\( x := \frac{21 \cdot f_t \text{in}}{f_b - f_t} = 2.993 \text{in} < 8\text{in} \)

The tensile force \( T \) in the concrete is:

\[ T := \frac{f_t}{2} b_8 x \]

where, \( b_8 \) is the width of the beam at top \( b_8 = 96\text{in} \)

Therefore \[ T := \frac{f_t}{2} b_8 x = 45.113\text{kip} \]

The required area of bonded reinforcement is:

\[ A_{req} := \frac{T}{f_s} = 0.188 \text{ in}^2 / \text{ft} \]

Use #4 @ 12in within the tensile zone \( \Lambda_s := 0.2 \text{ in}^2 / \text{ft} > \Lambda_{req} \hspace{1cm} \text{OK} \)

6.5. Control of cracking \hspace{1cm} \text{LRFD Art. A.5.7.3.4}

In the longitudinal direction, due to the existence of prestress strands, cracking is assumed to not happen. Therefore only in the transverse direction, cracking is considered.

A summary of the demands on a 1ft strip for the 30ft span in Service I limit state, according to Table 6, is as follows:
Table 11

<table>
<thead>
<tr>
<th>Location</th>
<th>Demand</th>
</tr>
</thead>
<tbody>
<tr>
<td>Mp_total (kip*ft/ft)</td>
<td>3 ft span</td>
</tr>
<tr>
<td>Mp_total (kip*ft/ft)</td>
<td>5 ft span</td>
</tr>
<tr>
<td>Mn_total (kip*ft/ft)</td>
<td>Exterior beam</td>
</tr>
<tr>
<td>Mn_total (kip*ft/ft)</td>
<td>Interior beam</td>
</tr>
</tbody>
</table>

The section is transformed elastic, cracked cross section. LRFD Art. 5.7.1

6.5.1 Check of the positive moment reinforcement

For the 5ft deck with #4@7in, according to Table 11, 

\[ M_{pos} = 7.66 \text{ kip-ft/ft} \]

6.5.1.1 Cracked moment of inertia

For a 1ft cross section, \( b = 1\text{ ft} \)

Bottom layer of rebar: \( A_s := 0.34 \text{ in}^2/\text{ft} \)

Top layer of rebar: \( A'_s := 0.34 \text{ in}^2/\text{ft} \)

The location of neutral axis \( x \) measured from the top fiber of the beam is determined as below.

Sum of statical moments about the neutral axis gives:

\[
\frac{1}{2}x^2 = n_c \cdot A_s \cdot (d - x) + n_c \cdot A'_s \cdot (h - d' - x)
\]

where, \( n_c = 5.933 \) \( d = 6.75\text{ in} \) \( d' = 5.25\text{ in} \)

\[ x = \sqrt{n_c \left( A_s^2 \cdot n_c + A'_s^2 \cdot n_c + 2 \cdot A_s \cdot d - 2 \cdot A'_s \cdot d' + 2 \cdot A'_s \cdot h + 2 \cdot A_s \cdot A'_s \cdot n_c \right) - A_s \cdot n_c - A'_s \cdot n_c} = 1.482\text{ in} \]

The cracked moment of inertia therefore is:

\[
I_{cr} := \frac{b \cdot x^3}{3} + n_c \cdot A_s \cdot b \cdot (d - x)^2 + n_c \cdot A'_s \cdot b \cdot (h - d' - x)^2 = 72.247\text{ in}^4
\]

6.5.1.2 Tensile stress in the bottom steel

\[
f_{ss} := n_c \cdot \frac{M_{pos} \cdot b}{I_{cr}} \cdot (d - x) = 39.765\text{ ksi}
\]

6.5.1.3 Rebar spacing

The thickness of concrete cover measured from extreme tension fiber to center of the flexural reinforcement located closest thereto (in) is:

\[ d_c := h - d = 1.25\text{ in} \]

where, \( h = 8\text{ in} \)

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Therefore: \[ \beta_s := 1 + \frac{d_c}{0.7(h - d_c)} = 1.265 \]

So that the maximum rebar spacing is:

\[ s_{\text{max}} := \frac{700 \cdot \gamma_e \cdot \text{in} - 2 \cdot d_c}{\beta_s \cdot \frac{f_{ss}}{\text{ksi}}} = 7.941 \cdot \text{in} > 7 \text{in} \quad \text{where,} \quad \gamma_e = 0.75 \quad \text{OK} \]

6.5.2 Check of the negative moment reinforcement

For the interior beams with #4@7in, according to Table 11, \( M_{\text{neg}} := 3.10 \frac{\text{kip}\cdot\text{ft}}{\text{ft}} \)

6.5.2.1 Cracked moment of inertia

For a 1ft cross section, \( b := 1 \text{ft} \)

Bottom layer of rebar: \( A_s := 0.34 \frac{\text{in}^2}{\text{ft}} \)

Top layer of rebar: \( A'_s := 0.34 \frac{\text{in}^2}{\text{ft}} \)

The location of neutral axis \( x \) measured from the bottom fiber of the beam is determined as below.

Sum of statical moments about the neutral axis gives:

\[ \frac{1}{2}x^2 = n_c \cdot A_s \cdot (h - d - x) + n_c \cdot A'_s \cdot (d' - x) \quad \text{where,} \quad n_c = 5.933 \quad d = 6.75 \cdot \text{in} \quad d' = 5.25 \cdot \text{in} \]

\[ x := \sqrt{n_c \left( A_s^2 \cdot n_c + A'_s^2 \cdot n_c - 2 \cdot A_s \cdot d + 2 \cdot A'_s \cdot d' + 2 \cdot A_s^2 \cdot h + 2 \cdot A_s \cdot A'_s \cdot n_c \right) - A_s \cdot n_c - A'_s \cdot n_c} = 1.18 \cdot \text{in} \]

The cracked moment of inertia therefore is:

\[ I_{\text{cr}} := \frac{b \cdot x^3}{3} + n_c \cdot A_s \cdot b \cdot (h - d - x)^2 + n_c \cdot A'_s \cdot b \cdot (d' - x)^2 = 39.998 \cdot \text{in}^4 \]

6.5.2.2 Tensile stress in the top steel

\[ f_{ss} := n_c \cdot \frac{M_{\text{neg}} \cdot b}{I_{\text{cr}}} \cdot (d' - x) = 22.46 \cdot \text{ksi} \]

6.5.2.3 Rebar spacing

The thickness of concrete cover measured from extreme tension fiber to center of the flexural reinforcement located closest thereto (in) is:

\[ d_c := h - d' = 2.75 \cdot \text{in} \quad \text{where,} \quad h = 8 \cdot \text{in} \]
Therefore: \[ \beta_s := 1 + \frac{d_c}{0.7(h - d_c)} = 1.748 \]

So that the maximum rebar spacing is:

\[ s_{\text{max}} := \frac{700 \cdot \gamma_e \cdot \text{in}}{\beta_s f_{ss} \cdot \text{ksi}} - 2 \cdot d_c = 7.87\text{-in} > 7\text{in} \quad \text{OK} \]

7.22ft bridge design using #4@7in

Compared with the 30ft bridge design, with the same reinforcement configuration and smaller demands (Table 5 and 6), the requirements of minimum reinforcement and cracking control reinforcement are definitely satisfied.

7.1 Development length

The development length for #4 bar is as determined before: \[ l_{dh} = 12\text{-in} \]

For both the positive and negative moment reinforcement, this point where the development length begins is taken at the centerline of the beam.

7.2 Distribution reinforcement

For longitudinal bottom bar, Use #4@10in (refer the 30ft bridge design)

7.3. Shrinkage and temperature reinforcement

As determined before, the minimum reinforcement area per foot, on each face and in each direction, shall be:

\[ A_{s,TS} := 0.11 \text{in}^2/\text{ft} \]

This criteria can be met on each face and in each direction. But this reinforcement need to be provided equally on both layers. The maximum spacing of the rebar shall not exceed either 3 times the deck depth or 18in. For the top longitudinal bar, use #4@18in. There is no bonding reinforcement needed as shown below:

From CONSPAN, at release, the tension stress in the top fiber of the beam at the transfer cross section is:

\[ f_t := -0.152\text{ksi} \]

which is smaller than the limiting tensile stress of concrete 0.2ksi
8. Design summary

8.1 Reinforcement configuration

<table>
<thead>
<tr>
<th>Main bar (transverse) length (in)</th>
<th>Development length (in)</th>
<th>Cut-off point (Interior beam)</th>
<th>Distribution bottom rebar (longitudinal)</th>
<th>Top rebar (longitudinal)</th>
</tr>
</thead>
<tbody>
<tr>
<td>40 ft</td>
<td>#4@5in</td>
<td>12in Inner face of each stem</td>
<td>#4@7in</td>
<td>#4@7in</td>
</tr>
<tr>
<td>30 ft</td>
<td>#4@7in</td>
<td>12in Inner face of each stem</td>
<td>#4@10in</td>
<td>#4@10in (exterior beam)</td>
</tr>
<tr>
<td>22 ft</td>
<td>#4@7in</td>
<td>12in Centerline between two stems</td>
<td>#4@10in</td>
<td>#4@18in Use #4@10in</td>
</tr>
</tbody>
</table>

Note: bonded reinforcement needs to be provided within the tension zone of concrete at the transfer cross section at the time of release.

8.2 Criteria check

8.2.1 Demand VS Capacity

<table>
<thead>
<tr>
<th>40 ft span Location</th>
<th>Demand (kip*ft/ft)</th>
<th>Capacity provided by #4@5in Ubar</th>
</tr>
</thead>
<tbody>
<tr>
<td>M+ (kip*ft/ft) 3 ft deck</td>
<td>21.18</td>
<td>N/A</td>
</tr>
<tr>
<td>M+ (kip*ft/ft) 5 ft deck</td>
<td>17.72</td>
<td>18.29</td>
</tr>
<tr>
<td>M- (kip*ft/ft) exterior beam</td>
<td>−8.42</td>
<td>−11.47</td>
</tr>
<tr>
<td>M- (kip*ft/ft) interior beam</td>
<td>−5.15</td>
<td>−11.47</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>30 ft span Location</th>
<th>Demand (kip*ft/ft)</th>
<th>Capacity provided by #4@7in Ubar</th>
</tr>
</thead>
<tbody>
<tr>
<td>M+ (kip*ft/ft) 3 ft deck</td>
<td>15.30</td>
<td>N/A</td>
</tr>
<tr>
<td>M+ (kip*ft/ft) 5 ft deck</td>
<td>12.75</td>
<td>13.59</td>
</tr>
<tr>
<td>M- (kip*ft/ft) exterior beam</td>
<td>−6.88</td>
<td>−8.90</td>
</tr>
<tr>
<td>M- (kip*ft/ft) interior beam</td>
<td>−5.15</td>
<td>−8.90</td>
</tr>
<tr>
<td>Location</td>
<td>Demand</td>
<td>Capacity provided by #4@7in Ubar</td>
</tr>
<tr>
<td>--------------------------</td>
<td>--------</td>
<td>---------------------------------</td>
</tr>
<tr>
<td>M+ (kip*ft/ft) 3 ft deck</td>
<td>11.02</td>
<td>N/A</td>
</tr>
<tr>
<td>M+ (kip*ft/ft) 5 ft deck</td>
<td>8.78</td>
<td>13.59</td>
</tr>
<tr>
<td>M- (kip*ft/ft) exterior beam</td>
<td>−5.25</td>
<td>−8.90</td>
</tr>
<tr>
<td>M- (kip*ft/ft) interior beam</td>
<td>−5.15</td>
<td>−8.90</td>
</tr>
</tbody>
</table>

Note: the positive moment capacity in the 3ft deck span, due to the development length of the reinforcing steel, is larger than that in the 5ft deck span.

8.2.2 Minimum reinforcement check

40 ft span

| M+ rebar | Any cross section | #4@5in | $M_{cr} = 18.29\text{kip} \cdot \text{ft}$ | > | $M_{cr} = 9.056\text{kip} \cdot \text{ft}$ | OK |

| M- rebar | Interior beam | #4@5in | $|M_T| = 11.47\text{kip} \cdot \text{ft}$ | > | $|1.33M_{cr}| = 6.849\text{kip} \cdot \text{ft}$ | OK |

| M- rebar | Exterior beam | #4@5in | $|M_T| = 11.47\text{kip} \cdot \text{ft}$ | > | $M_{cr} = 9.056\text{kip} \cdot \text{ft}$ | OK |

30 ft span

| M+ rebar | Any cross section | #4@7in | $M_{cr} = 13.59\text{kip} \cdot \text{ft}$ | > | $M_{cr} = 9.056\text{kip} \cdot \text{ft}$ | OK |

| M- rebar | Interior beam | #4@7in | $|M_T| = 8.90\text{kip} \cdot \text{ft}$ | > | $|1.33M_{cr}| = 6.849\text{kip} \cdot \text{ft}$ | OK |

| M- rebar | Exterior beam | #4@7in | $|M_T| = 8.90\text{kip} \cdot \text{ft}$ | < | $M_{cr} = 9.056\text{kip} \cdot \text{ft}$ |

Overhang reinforcement needs to extend beyond the critical negative moment demand point for a length of its development length.

22 ft span

With the same rebar configuration with 30ft span and less demands, this requirement is satisfied at any cross section.
8.2.3 Cracking control check

40 ft span

\[ s_{\text{max}} := \frac{700 \cdot \gamma_e}{f_{ss}} \text{in} - 2 \cdot d_c = \frac{7.933}{5} \text{in} > 5 \text{in} \quad \text{OK} \]

Note: Positive moment demand in the 5ft span deck is the most critical, and negative moment demand in the exterior beam is most critical.

30 ft span

\[ s_{\text{max}} := \frac{700 \cdot \gamma_e}{f_{ss}} \text{in} - 2 \cdot d_c = \frac{5.83}{5} \text{in} > 5 \text{in} \quad \text{OK} \]

For exterior beams, due to the extension of the overhang reinforcement (see the minimum reinforcement check), this criteria can be satisfied.

22 ft span

With the same rebar configuration with 30ft span and less demands, this requirement can be satisfied at any cross section.
Appendix E

Bridge Drawings
Figure E.1: NEXT-6 22 ft. deck and beam detail
Figure E.2: NEXT-6 30 ft. deck and beam detail
Figure E.3: NEXT-6 40 ft. deck and beam detail
Figure E.4: NEXT-8 20 ft. deck and beam detail
Figure E.5: NEXT-8 30 ft. deck and beam detail
Figure E.6: NEXT-8 40 ft. deck and beam detail
Figure E.7: NEXT-6 22 ft. deck and beam cross-section detail

Figure E.8: NEXT-6 30 ft. deck and beam cross-section detail
Figure E.9: NEXT-6 40 ft. deck and beam cross-section detail

Figure E.10: NEXT-8 22 ft. deck and beam cross-section detail
Figure E.11: NEXT-8 30 ft. deck and beam cross-section detail
Figure E.12: NEXT-8 40 ft. deck and beam cross-section detail

Cross Section 1-1 Anchorage zone

Cross Section 2-2 Other zone
Notes:

1. * denotes straight strands, only straight strands are used
2. Strand type is 1/2-270k-LL low relaxation strands
3. Except for the T&S bar, the rebar dimensions shown to the left apply to all the other spans of NEXT-6
   For the length and splice of T&S bar, follow the usual practice.
4. The distribution of T&S bar should not follow that given in CONSPAN, because the beam geometry in CONSPAN
   does not consider shear key, or the asymmetrical geometry of the exterior beam.
5. The prestressing strand patterns shown are the final patterns
6. The top two prestressing strands are at a fully prestressed.
7. The vertical bars are all #4

Bars:

Exterior beam:

Top overhang bar: #6

U bar: #4

Interior beam:

U bar: #4

Others:

Vertical bar #4

Figure E.13: NEXT-6 notes and bar details

Notes:

1. * denotes straight strands, only straight strands are used
2. Strand type is 1/2-270k-LL low relaxation strands
3. For the length and splice of T&S bar, follow the usual practice.
4. The distribution of T&S bar should not follow that given in CONSPAN, because the beam geometry in CONSPAN
   does not consider shear key, or the asymmetrical geometry of the exterior beam.
5. The prestressing strand patterns shown are the final patterns
6. The top two prestressing strands are at a fully prestressed.
7. The vertical bars are all #4

Figure E.14: NEXT-8 notes
Bars:

Exterior beam:

Top overhang bar: #5

U bar: #4

Interior beam:

U bar: #4

Others:

Vertical bar #4

Figure E.15: NEXT-8 22 ft. bar details
Bars:

Exterior beam:

**Top overhang bar:** #5

**U bar:** #4

Interior beam:

**U bar:** #4

Others:

**Vertical bar #4**

Figure E.16: NEXT-8 30 ft. bar details
Bars:

Exterior beam:

Top overhang bar: #5

U bar: #4

Interior beam:

U bar: #4

Others:

Vertical bar #4

Figure E.17: NEXT-8 40 ft. bar details


ACI (2011). *ACI 318-11: Building Code Requirements for Structural Concrete and Commentary*. American Concrete Institute, Farmington Hills, MI.


