**Title and Subtitle**

**More Choices for Connecting Prefabricated Bridge Deck Elements**

**Abstract**

Full-depth precast bridge deck panels are widely used to expedite bridge deck erection and accelerated bridge construction (ABC). They can also provide cost savings, better durability, and solve many constructability challenges. Full-depth deck joints with concave shear key type are the most common field joint used in bridge decks. Currently, ultra-high performance concrete (UHPC) is widely used for closure joints for precast bridge deck connections. UHPC has superior mechanical properties but has some cost and bidding challenges due to the limited number of suppliers and the special expertise needed to work with it. Poly Methyl Methacrylate Polymer concrete (PMMA-PC) is another popular advanced construction material commonly used for overlays and can provide a potential alternative to UHPC for deck joints. PMMA-PC provides high early bond and shear strength, adequate flowability, and high durability, which makes it a good candidate for ABC connections. The objective of this study is to investigate and compare the structural performance of using PMMA-PC and UHPC for closure joints in full-depth field joints. An experimental program was conducted to test five full-scale specimens under monotonic vertical loading at the University of Nevada, Reno. Three specimens represent transverse deck panel joints and the other two specimens represent longitudinal connections in deck-bulb-tee girders (DBTs). The structural response was evaluated for connection performance under the AASHTO service and ultimate loads, peak strength, damage progression and mode of failure, and local steel reinforcement strains. The results of the study demonstrated that PMMA-PC could be successfully used for bridge deck field joints with comparable performance to UHPC deck joints.
More Choices for Connecting Prefabricated Bridge Deck Elements

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Principal Investigator: Mohamed A. Moustafa
Department of Civil and Environmental Engineering
University of Nevada, Reno, NV

Co-Principal Investigator: Ahmad M. Itani
Department of Civil and Environmental Engineering
University of Nevada, Reno, NV

Authors
Mohamed Abokifa, Mohamed A. Moustafa, and Ahmad M. Itani

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A report from
University of Nevada, Reno
Department of Civil and Environmental Engineering, MS 258
1664 N. Virginia St.
Reno, NV 89557
www.unr.edu/cee
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Abstract

Full-depth precast bridge deck panels are widely used to expedite bridge deck erection and accelerated bridge construction (ABC). They can also provide cost savings, better durability, and solve many constructability challenges. Full-depth deck joints with concave shear key type are the most common field joint used in bridge decks. Currently, ultra-high performance concrete (UHPC) is widely used for closure joints for precast bridge deck connections. UHPC has superior mechanical properties but has some cost and bidding challenges due to the limited number of suppliers and the special expertise needed to work with it. Poly Methyl Methacrylate Polymer concrete (PMMA-PC) is another popular advanced construction material commonly used for overlays and can provide a potential alternative to UHPC for deck joints. PMMA-PC provides high early bond and shear strength, adequate flowability, and high durability, which makes it a good candidate for ABC connections. The objective of this study is to investigate and compare the structural performance of using PMMA-PC and UHPC for closure joints in full-depth field joints. An experimental program was conducted to test five full-scale specimens under monotonic vertical loading at the University of Nevada, Reno. Three specimens represent transverse deck panel joints and the other two specimens represent longitudinal connections in deck-bulb tee girders (DBTs). The structural response was evaluated for connection performance under the AASHTO service and ultimate loads, peak strength, damage progression and mode of failure, and local steel reinforcement strains. The results of the study demonstrated that PMMA-PC could be successfully used for bridge deck field joints with comparable performance to UHPC deck joints.
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1. Introduction

1.1 Background

Time, cost, and quality are the three main aspects of every engineering project. Reducing bridge construction time using high-quality materials to enhance bridge durability, will accomplish less construction and maintenance costs. The growing need for durable highway bridge construction and rehabilitation systems compels research agencies and departments of transportation (DOTs) to search for better construction methods to facilitate rapid completion of on-site activities to minimize the impact on road commuters. There are over 617,000 bridges in the United States. More than 46,000 bridges, which are almost 7.5% of the total count, are considered structurally deficient as they need to be repaired or replaced. These numbers were based on a report on the state of the nation's roads and bridges from the American Road and Transportation Builders Association (ARTBA).

There are many issues associated with the low pace of the traditional cast-in-place bridge construction or repair techniques such as the traffic shutdowns and delays, higher labor costs, and the less quality control associated with the on-site concrete casting. As a result, this has paved the way for wider use of precast elements to expedite bridge erection and accelerated bridge construction (ABC). Prefabricated bridge elements and systems (PBES) can enhance constructability issues, provide higher quality, accelerated and safer construction, and minimize traffic disruption and safety concerns arising from detours.

Bridge decks are traditionally fabricated using cast-in-place (CIP) conventional concrete, creating a monolithic deck slab with many expansion joints. Nonetheless, CIP decks were shown to be susceptible to rapid degradation due to many factors. The implementation of the precast deck panels in bridge decks has improved the quality and durability because of the higher quality control associated with the precast fabrication techniques. However, the precast bridge deck connections among the other ABC connections usually require field casting and could form a weak link that affects the overall system performance. Moreover, bridge decks are heavily stressed throughout their life span by both structural loadings and severe environmental conditions, such as the use of deicing salts in cold environments, wearing due to traffic, etc. The relatively wide field connections between the precast deck slabs with the use of the traditional closure joints have often experienced interface leakage issues, resulting in less desirable overall system performance.

Currently, ultra-high performance concrete (UHPC) is commonly used for closure joints between precast bridge deck panels as it has superior mechanical properties and high durability, but is expensive and requires special expertise to work with it. Moreover, robust UHPC mixes are currently proprietary which may limit DOTs that are trying to avoid sole-sourcing and bidding issues from using UHPC. Therefore, identifying other alternative materials that can provide small-
sized connections with simpler reinforcement configurations in addition to the elimination of deck panel system post-tensioning can be extremely beneficial.

In this project, poly methyl methacrylate polymer concrete (PMMA-PC) has been selected and proposed as a potential alternative to UHPC in field joints. This is attributed to PMMA-PC high bond strength, high early strength, high shear strength, excellent flowability, and high durability. Polymer concrete (PC) in general is well known for its high bond strength to the existing surfaces and reinforcement compared to conventional concrete. Furthermore, PC is corrosion resistant, has a superior cracking resistance, and has shown excellent durability over the years in various applications around the nation (ACI 548.1R, 2009).

PMMA-PC has been used for decades as bridge deck overlay and as a fast-permanent repair to runways and taxiways in many airports around the United States. The superior properties of the PC make it a favorable material for use in bridge deck overlays (Whitney and Fowler, 2015). A recent study was conducted at the University of New Mexico (Mantawy et al., 2019) investigating the applicability of using the PMMA-PC as a material for closure joints between the precast bridge deck slabs. This study focused on the development length and lap splice length of uncoated bars inside the PMMA-PC and demonstrated that the PMMA-PC could provide shorter development and lap splice lengths than UHPC. Moreover, the study showed that the PMMA-PC has almost double the shear strength of the UHPC and one-half the cost of the typical UHPC with 2% steel fibers, as the cubic yard of the UHPC costs around $4,000 while a cubic yard of the PMMA-PC costs around $2,000. The study concluded that PMMA-PC has the potential to be a good candidate or alternative for closure joints in ABC deck field connections. The latter research project among other applications of PC motivated this study to further investigate and demonstrate the structural behavior of precast deck joints filled with PMMA-PC.

1.2 Research Objectives and Approach

The main objective of this study is to investigate the structural performance of PMMA-PC as compared to UHPC for precast bridge deck joints under static vertical loading. This is to determine whether PMMA-PC can be effectively used for closure joints in full-depth precast deck panel connections. The core of this study is an experimental program that consists of five full-scale bridge deck specimens with PMMA-PC and UHPC field joints tested under static vertical loading as explained in detail in this report.

The research discussed herein focuses on evaluating the structural performance of the PMMA-PC joints for precast bridge deck slabs in addition to comparing the overall performance with that of the UHPC joints. The precast bridge deck field joints including the longitudinal and transverse directions of the bridge were fabricated and tested under static wheel patch loading up to failure. The research goal and objectives have been accomplished through a set of research activities and tasks as follows.
• Review the relevant data, previous researches, experimental and analytical test results, and other information related to the design of deck slabs, fabrication, and erection of CIP connections at the bridge deck level.

• Investigate several classes of materials for demonstrating the selection of PC as a potential closure joint material and alternative to UHPC. A detailed literature search has been conducted to determine one possible alternative to replace the UHPC in precast deck closure joints.

• Conduct several material tests of PMMA-PC samples to characterize the mechanical properties and explore its applicability to use as a closure material for field joints. Various material tests were performed to determine the mechanical properties of the PMMA-PC including compressive strength, tensile strength, elastic modulus, flexural strength, and typical compressive stress-strain curves were obtained.

• Develop a detailed experimental research plan, which includes testing a total of five full-scale specimens under static loading, with variables including joint orientation, reinforcement configuration, slab thickness, shear key type, and closure joint material. Two transverse connection specimens represented a typical field connection detail between precast deck panels. These specimens were identical except for the closure joint material type, which included UHPC and PMMA-PC. An additional PMMA-PC transverse connection with a new shear key shape and loop type reinforcement splice was proposed and tested as well. The two longitudinal connection specimens represented the field joints between the top flanges of deck-bulb-tee (DBT) girders. These two specimens were again identical except for the closure joint material type, i.e., UHPC versus PMMA-PC.

• Conduct flexure tests on precast deck slab specimens under static loading and unloading. The specimens were supported to form a one-way simple span slab with a vertical load applied through a simulated wheel patch placed adjacent to the field joint near the midspan. The specimens were loaded up to failure to assess the static performance of the field connections, and in turn, come up with a conclusion whether the connections effectively emulated the anticipated performance from a monolithic deck.

• Perform a detailed experimental test results interpretation, evaluation, and discussion. The structural response of the specimens is evaluated for peak strength, deflection, damage schemes, and steel reinforcement strains. Finally, conclusions and recommendations are then proposed.

1.3 Outline of the Report

This report consists of seven chapters. The first chapter provides an overview and introduction. Chapter 2 summarizes the relevant background information and research findings from previous studies about the deck-panel connections in addition to the literature review of the PMMA-PC
mechanical properties. Chapter 3 presents the mixing process and the results of the material testing of the UHPC and the PMMA-PC used in this study. Chapter 4 presents the geometric details of the test specimens along with the mechanical properties of the PMMA-PC, UHPC, and the conventional concrete included in the study. Moreover, Chapter 4 provides the details of the experimental program including the fabrication steps, instrumentation plan, and test setup. Chapter 5 presents the test results and research findings along with a detailed discussion. Chapter 6 presents a comparison between the different tested specimens. Chapter 7 summarizes the significant conclusions of this research program and suggestions for future work.
2. Literature Review

2.1 Overview

Modern construction techniques have led to the use of prefabricated elements in the construction of nowadays bridges. However, these prefabricated members are very challenging as they need to be joined together on-site using simple and durable connections. These types of field connections are required to be simple to reduce the time needed for bridge erection and should be durable to eliminate the risk of structural deterioration before reaching the targeted life span of the bridge. The conventional ways of constructing these connections can lead to the rapid deterioration of the precast members.

One of the common applications of the PBES is the precast bridge deck systems. The use of the precast bridge deck systems has led to the development of several types of field cast connections that can be classified into two main types depending on the orientation of the connection with respect to the traffic flow over the bridge as shown in Figure 2-1. Transverse connections run perpendicular to the direction of traffic over the bridge, while longitudinal connections run parallel to the longitudinal axis of the bridge and the traffic direction.

![Figure 2-1: Joint orientations in a precast bridge deck system (NCHRP, 2011).](image)

Currently, the most common transverse precast deck connection has a diamond-shaped shear key with a full-depth UHPC closure material and a non-contact lap splice. The longitudinal field connections are also commonly used in the DBT girders and they also have almost the same configuration as the transverse joints (see Figure 2-2).
As mentioned before, the main overarching objective of this study is to identify a potential alternative closure material to be used on the transverse and longitudinal field connections of the precast bridge decks. Then, test this alternative material in the deck field joints of full-scale precast bridge deck specimens/subassemblies under static vertical loading and compare the structural performance against identical specimens that use UHPC closure joints. Thus, the literature search conducted in this study is divided into two main parts. (1) Investigating several classes of materials to select a proper closure joint material to be a potential alternative to UHPC and then identify the available mechanical properties of this alternative material from previous research works. (2) Reviewing the previous experimental studies conducted on precast bridge deck panels with transverse and longitudinal field connections.

### 2.2 Alternative Closure Joint Material

Over the last decade, most of the relevant research studies on precast deck panel joints have not focused on testing alternative closure materials but rather focused on the experimental testing and demonstration of the use of UHPC for such joints (e.g., Graybeal, 2010; Vitek et al., 2016; Hartwell et al., 2011; Hwang and Park, 2014). However, other research studies considered using other materials and focused on investigating the behavior of such connections with advanced grouts (Zhu et al., 2012; Li and Jiang, 2015), high-performance concrete (HPC) (Zhu et al., 2012; Leboeuf et al., 2017), and HPC with fiber reinforcement (Leboeuf et al., 2017). But many of these research efforts included special considerations for the reinforcement inside the joint such as rebar post-tensioning, using of mechanical splices, using of headed rebars, or adding of rebar confinement inside the joint.

Figure 2-3 shows some of the special requirements for the field joints that were reported in the literature. Some other studies considered increasing the joint width to maintain the desired
performance of the deck field joints. One of the experimental studies that implemented the use of wider joints is (Choi et al., 2015). The study tested 20 in wide joints using high-strength concrete reinforced with polyamide fibers with a design strength of 17.4 ksi. The study by Choi et al. (2015) showed that under static loading, the cracking load and the maximum load were at least 90% of those of the control specimen of the same dimensions without joints and the maximum deflection recorded in the cyclic loading was 26% larger.

Porter et al. (2011) performed flexure testing on 15 full-scale specimens with transverse field joints to determine the cracking and ultimate strengths of five different connection details. The study concluded that the welded stud connection (see Figure 2-3c) did not perform as well as the welded rebar connection (see Figure 2-3d) and the ultimate capacity of the welded rebar connection was 0.44 times that of a continuous panel. The study did not recommend using both types of connections in negative moment regions. Therefore, these types of connections do not seem to be attractive for implementation on the bridge decks as it requires more time and labor during construction and it does not satisfy the strength capabilities requirements.
Some of the previously mentioned ideas could be acceptable in the past as most of the research was trying to minimize the joint widths by implementing the conventional closure non-shrink grouts. However, these ideas can be eliminated using simple connections after the breakthrough in advanced construction materials, especially the UHPC. Consequently, thinking of using these complicated alternative solutions in nowadays bridge deck joints is not ideal as they may oppose the main concept of accelerating the bridge construction that mainly focuses on having durable bridge systems with less erection time, optimum cost, and less labor.

Currently, UHPC is used for closure joints between the precast bridge deck panels because of its superior mechanical and durability properties. However, UHPC poses cost and bidding challenges due to the limited number of suppliers and the proprietary nature of this robust mix. This often limits state departments of transportation (DOTs) that are trying to avoid sole-sourcing among other bidding issues to use UHPC. Another challenge for using UHPC is that it requires special expertise to mix and place due to the steel fibers in addition to the required long mixing time for this material. Moreover, UHPC has high early shrinkage issues that require more time and additional labor costs to have an acceptable surface finish. Because of the proprietary nature of the UHPC, many state DOTs started to develop their non-proprietary UHPC mixes using locally and domestically available materials that could provide a more desirable option for the closure joint material implementation (Qiao et al., 2017).

It is noted that in line with the search for more options and alternatives for ABC connections, the authors are currently working through the ABC-UTC on testing precast bridge deck joints with non-proprietary UHPC mix which uses local materials sourced from Nevada or the western region in general. The study is considered an extension or phase two of the research work reported herein. One other drawback of the UHPC is that it requires more setting time after pouring in cold weather that may lead to fiber segregation and accumulation at the bottom of the joints as reported by one of the project engineers at the Pulaski Skyway project (McDonagh and Foden through discussion at the ABC-UTC international conference in Miami, 2019).

The aforementioned major drawbacks of the previously mentioned systems motivated the authors to search for another alternative advanced material to be used inside the field connections between the precast bridge deck panels. The implementation of an alternative durable closure joint material with superior mechanical properties in such connections can simplify the connection details, enhance the connection durability, narrow down the width of the connections, and save the time needed for the formwork erection.

PMMA-PC is another advanced construction material that is conveniently used nationwide over the past decades in the bridge deck overlays and is proposed in this study as an alternative closure material. The mechanical properties of the PMMA-PC concrete make it an adequate candidate to be widely used as a closure material inside the precast field joints. The PC in general has a high bond strength, high early strength, high shear strength, adequate flowability, and high freeze and
thaw resistance (ACI 548.1R, 2009; Ribeiro et al., 2003). Moreover, PC is corrosion-resistant, fast curing, has very low permeability, and superior cracking resistance (ACI 548.1R, 2009; Ribeiro et al., 2002; Reis and Ferreira, 2003). The superior properties of PC make it a popular material for use in bridge deck overlays (Whitney and Fowler, 2015) and make it a very attractive material for use in structural engineering applications and many industries (ACI 548.1R, 2009; Fowler and Paul, 1978; Fontana et al., 1978; Kukacka and Fontana, 1977; Fowler et al., 1983).

Among the different types of PC, the PMMA-PC has been used for decades as bridge deck overlay and fast-permanent repair for runways and taxiways in many airports around the nation (Dinitz and Ferri, 1985). PMMA-PC beams proved to be superior to Portland cement concrete beams in fatigue strength (Hsu and Fowler, 1985). A recent study was conducted at the University of New Mexico (Mantawy et al., 2019) to determine the mechanical properties of the PMMA-PC to investigate the applicability of using it as a closure joint material between precast bridge deck panels. This study focused mostly on the development length and lap splice length and demonstrated that the PMMA-PC could provide shorter development and lap splice lengths than the UHPC.

The minimum development length required for the steel rebars embedded in PMMA-PC was found to be almost one-half that of the UHPC and ranges between 3.6 and 4.1 times the reinforcing bar diameter (d_b) (Mantawy et al., 2019). The minimum lap splice length required for the uncoated rebars embedded in PMMA-PC was slightly less than that of the UHPC as only 4.1 d_b lap splice length is sufficient to achieve yielding in the rebars (Mantawy et al., 2019). Moreover, that same study showed that the PMMA-PC has almost double the shear strength of the UHPC and one-half the cost of the UHPC as the cubic yard of the UHPC costs $4,000 while a cubic yard of the PMMA-PC costs $2,000 (Mantawy et al., 2019). The study concluded that PMMA-PC has the potential to be a good candidate or alternative closure material in the bridge deck field connections and proposed a potential optimized PMMA-PC joint of 3 inches width compared to a UHPC joint of 4 inches width as shown in Figure 2-4. The latter research project among other applications of the PC motivated this study to further investigate the structural behavior of the bridge deck field connections filled with PMMA-PC.

![Figure 2-4: PMMA-PC proposed field joint (Mantawy et al., 2019).](image)

### 2.3 Precast Bridge Deck Field Connections

Many research efforts considered testing different parameters of the field joints such as types of field connections, shear key shapes, details of reinforcement, types of reinforcement and tested the
various connections details under different loading schemes to come up with the best and simple connection detail that can emulate a monolithic bridge deck in terms of distribution of the load.

As a result of a large number of studies in this area, several reports were published through the national cooperative highway research program (NCHRP) to summarize and synthesize the state-of-the-art for prefabricated concrete components and deck connections and detailing. For example, the NCHRP report 10-71 (NCHRP, 2011) focused on static and fatigue testing of both transverse and longitudinal field connections between precast concrete deck panels. Consequently, the synthesized practical and research projects converged on typical UHPC field joints that are 6 to 8 inches wide without post-tensioning and use simplified reinforcement configuration inside the joint with diamond-shaped shear key type.

This typical connection has been demonstrated to develop sufficient shear and bending capacities to link the precast deck components together (Graybeal, 2010; Perry et al., 2012; Sritharan et al., 2012). The experimental study done by Graybeal (2010) was the main reference for the present work as it implemented approximately the same flexural static testing of specimens of similar dimensions and joint details. However, the major differences were the cyclic loading applied before the static push-over test and the proprietary UHPC closure joints. Graybeal (2010) tested four full-scale transverse specimens and two longitudinal specimens under cyclic and static loading as shown in Figure 2-5.

Graybeal (2010) experimentally showed that a UHPC joint of 6 inches width with a non-contact lap splice could be sufficient to emulate a monolithic bridge deck in terms of load distribution capabilities across different precast members. Graybeal (2010) also tested different reinforcement configurations inside the joint such as headed reinforcement, loop splice, and the ordinary straight splice. Moreover, various types of steel such as black steel, epoxy-coated steel, and galvanized steel were considered in that study as well. The study showed that there is a little difference in the
load capacities of the four transverse specimens as they all ranged from 105.9 kips to 113 kips. On the other hand, the mid-span displacements were different as the least deflection reading at failure was 2.16 in and it was for the specimen with the headed black reinforcement while the largest deflection at failure was 2.9 in for the specimen with epoxy-coated reinforcement and loop splice detail inside the joint. That study concluded that there is no observance of any interface cracks during the cyclic loading however, three transverse specimens experienced short-length interface cracks at the heavily cracked regions near mid-span during the static loading. The study also concluded that individual cracks in the precast segments were transformed into multiple micro-cracks inside the UHPC joint as shown in Figure 2-6.

![Cracking pattern observed at the bottom of a transverse specimen after static loading (Graybeal, 2010).](image)

As mentioned earlier, Graybeal (2010) tested two longitudinal specimens but one of the specimens was unintentionally pre-cracked before testing. This affected the results but also the other specimen experienced a system malfunction that resulted in an unintentional overload of the specimen to a load capacity of 70 kips and many rebars were fractured before applying the static loading because of the higher amplitude cyclic loading that was applied for this specimen. Because of such unintentional errors that resulted in a probable inaccuracy of the results from the longitudinal specimens, we included testing two longitudinal specimens in our study to experimentally demonstrate that a UHPC and PMMA-PC joints with 6 inches width can be sufficient to emulate a monolithic bridge deck.
DBT girders are one of the attractive superstructure systems that have been implemented by many DOTs such as WSDOT and NYSDOT over the last decades. The DBT girders can provide fast bridge superstructure erection, less on-site activities, less traffic disruption, and more convenient choices for rural areas. The I-shaped DBT girders have a wide top flange that replaces the need of constructing a bridge deck and reduces the time needed for the from work of the bridge decks. The DBT girders are constructed in a controlled environment at the fabrication facility and then shipped to the bridge location where they can be quickly assembled to form the bridge superstructure. However, this system requires on-site pouring of longitudinal field connections between the top flanges of the girders to provide continuity for the distribution of the load along the transverse direction of the bridge. In the past, these field connections were constructed using a combination of welded clips and grouted joints. However, these connections showed a lack of durability due to cracking observance along the joint interface (Qiao et al. 2017).

As mentioned before, a recent study was conducted by the FHWA that focused on the experimental testing of the UHPC as an advanced material to be used in the longitudinal closure joints between the DBT girders (Graybeal, 2010). A more recent study was done by WSDOT on the experimental testing of non-proprietary UHPC closure joints between the DBT girders (Peruchini et al. 2017). This study physically tested 2-ft wide strips of the deck representing the actual upper flanges of the DBT girders under static bending. The proposed UHPC joint in that study included a non-contact lap splice connection with variable joint widths ranging between 3 inches to 7 inches. The study showed that there is no cracking happened to the body of the UHPC joint, however, a significant cracking was observed at the cold joint interface between the UHPC and the conventional concrete part as shown in Figure 2-7.

![Figure 2-7: Joint-deck interface crack (Peruchini et al. 2017).](image)

The damage scheme illustrated above makes this system less desirable because the moisture and chemicals could ingress through the interface cracks and lead to rapid corrosion of the
reinforcement and deterioration of the bridge deck. However, there could be many solutions that could be provided to enhance the interface bond such as using female-female shear key shape and the use of surface preparation such as exposed aggregate (see Figure 2-8) as proposed by the FHWA (Graybeal, 2014; FHWA Publication No: FHWA-HRT-14-084) to ensure the durability and long-term performance of the joints.

Based on the above summary, none of the previous efforts focused on large or full-scale demonstration of alternative materials such as PMMA-PC for deck joint connections, which is the knowledge gap filled through this study. The main objective of this study was again to investigate the structural performance of PMMA-PC as compared to UHPC for precast bridge deck joints under static vertical loading. This is to determine whether PMMA-PC can be effectively used for closure joints in transverse full-depth precast deck panel connections and to determine whether the PMMA-PC can be effectively used for closure joints in the longitudinal full depth connections with using the simple non-contact straight lap splice with a 6.0 in width with diamond shear key shape.

![Figure 2-8: Exposed aggregate surface on a bridge deck field joint interface (Graybeal, 2014).](image)
3. Material Testing

3.1 Overview

Several materials were used in this study such as UHPC and PMMA-PC that were used inside the field joints, and normal strength concrete (NSC) that was used in the fabrication of the precast concrete panels. In addition, A706 Grade 60 reinforcement was used in this study. This chapter reports the mixing procedures and the mixing proportions of the previously mentioned types of concretes and summarizes the sampling and testing that was done to determine the mechanical properties of these materials.

3.2 Ultra-High Performance Concrete (UHPC)

UHPC is a fiber reinforced, Portland cement-based product composed of an optimized gradation of the particle sizes of the mixing components with a water to cement ratio less than 0.25. The compressive strength of the UHPC is typically greater than 21.7 ksi and the tensile strength is greater than 0.72 ksi as defined by Greybeal (2014). The UHPC became a well-known construction material for the last decade as it is commercially available in pre-packed bags and plasticizer buckets across the United States and Europe. As mentioned earlier, most of the robust UHPC mixes are currently proprietary such as the product provided by Lafarge Holcim, which is the type of UHPC that is used in this study, i.e., Ductal® JS1000. The Ductal® JS1000 UHPC has been already used for closure joints to connect prefabricated structural elements on-site for several projects across the United States. Its flowability, strength, and enhanced bond properties make it ideal for this application.

3.2.1 UHPC Mixing Proportions

The UHPC is commonly packed into three different components as follows: (1) dry components that are packed in 50-lbs bags and consist of proprietary pre-blended cement, sand, ground quartz, and silica fume (see Figure 3-1a); (2) liquid admixture that is packed in 5-gallon pails and consists of a high range water reducer (HRWR) in addition to an accelerator (see Figure 3-1b); (3) steel fibers that are packed in 44-lbs bags and the single steel fiber has dimensions of 0.008 in diameter and 0.51 in long with tensile strength greater than 290 ksi (see Figure 3-1c). Table 3-1 shows the mixing proportions of the Ductal® JS1000 UHPC based on the number of dry premix bags used on a single batch. Table 3-2 shows the mix components and quantities per cubic yard and cubic meter of the Ductal® JS1000 UHPC.

The UHPC was mixed at the fabrication yard outside the Earthquake Engineering Laboratory at UNR using a high shear mixer that is readily available at UNR (see Figure 3-2). The type of high shear mixer used in this study is “Imer 750” with a total capacity of 5.12 ft³ (0.145 m³). Only half of the mixer capacity was used for all the developed batches used for this study to ensure the consistency of the UHPC batches and to better control the quality of the mixes.
Figure 3-1: Ductal® UHPC components (a) dry premix, (b) superplasticizer, and (c) steel fibers.

Table 3-1: Ductal® UHPC mixing proportions according to the number of dry premix bags.

<table>
<thead>
<tr>
<th>No. of bags per batch</th>
<th>5</th>
<th>6</th>
<th>7</th>
<th>8</th>
<th>9</th>
</tr>
</thead>
<tbody>
<tr>
<td>Dry premix (lb)</td>
<td>250</td>
<td>300</td>
<td>350</td>
<td>400</td>
<td>450</td>
</tr>
<tr>
<td>Water or ice (lb)</td>
<td>14.37</td>
<td>17.24</td>
<td>20.12</td>
<td>22.99</td>
<td>25.86</td>
</tr>
<tr>
<td>Superplasticizer (lb)</td>
<td>3.43</td>
<td>4.11</td>
<td>4.80</td>
<td>5.48</td>
<td>6.17</td>
</tr>
<tr>
<td>Steel fibers (lb)</td>
<td>17.77</td>
<td>21.32</td>
<td>24.88</td>
<td>28.43</td>
<td>31.99</td>
</tr>
<tr>
<td>Batch volume (ft³)</td>
<td>1.82</td>
<td>2.19</td>
<td>2.55</td>
<td>2.92</td>
<td>3.28</td>
</tr>
</tbody>
</table>

Table 3-2: Ductal® UHPC mixing proportions per cubic yard and cubic meter.

<table>
<thead>
<tr>
<th></th>
<th>kg/m³</th>
<th>lb/yd³</th>
<th>Percentage per weight (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Dry premix</td>
<td>2195</td>
<td>3700</td>
<td>87.6</td>
</tr>
<tr>
<td>Water or ice</td>
<td>130</td>
<td>219.1</td>
<td>5</td>
</tr>
<tr>
<td>Superplasticizer</td>
<td>30</td>
<td>50.6</td>
<td>1.2</td>
</tr>
<tr>
<td>Steel fibers (2% volume)</td>
<td>156</td>
<td>263</td>
<td>6.2</td>
</tr>
</tbody>
</table>

3.2.2 UHPC Mixing Procedure

The mixing procedure of the UHPC is divided into three main steps as shown in Figure 3-3. The three steps are as follows: (1) the dry premix blend is added to the mixer to mix for 1 minute; (2) the superplasticizer is then added to the water and then both are added gradually over the course of 2 minutes to the dry premix and then leave the mixing continues for another more 10-13 minutes till the UHPC started to be like a dough; (3) finally, the steel fibers are then added gradually to the UHPC paste over the course of 1 minute and then left for 2-4 minutes to ensure uniform
distribution of the fibers inside the mix. It is strictly recommended to use 100% ice instead of the required quantity of water if the daytime temperature exceeded 25°C (77°F) and this is to ensure that the mix will have sufficient working time for mixing, placing, and forming. As mentioned before that the UHPC is a flowable material, and hence there is no need to use a vibrator to avoid any fiber segregation.

Figure 3-2: UHPC high shear mixer “Imer 750”.

Figure 3-3: UHPC mixing procedure.
It is also recommended by Aboukifa et al. (2018) to place the UHPC inside the field joints from the same place to prevent the formation of any cold joints between separate flows. Three UHPC batches are used to form the field joints of two of our specimens. One of the specimens used two UHPC batches and the typical mixing time for one UHPC batch is around 25-30 minutes. This time difference between both batches could lead to a formation of a little crust over the surface of the UHPC consequently, a steel rod was used to break this little crust before pouring the second batch to avoid any formation of cold joints between any two successive UHPC batches.

3.2.3 **UHPC Material Testing**

3.2.2.1 **Flowability Test**

A static and dynamic flow test was conducted on each UHPC batch to ensure the appropriate flowability of the mix before pouring into the field joints. The flowability of the mix was measured using a spread cone mold in accordance with ASTM C 1437/230 as shown in Figure 3-4. The measured static flow tests ranged between 8.25 in to 8.75 in for all the three UHPC batches while the dynamic flow tests were typically greater than or equal to 10 in.

![Figure 3-4: UHPC static flow test.](image)

3.2.2.2 **Compression Test**

As mentioned earlier, three UHPC batches were used in this study. A 3x6 in cylinders were prepared from each batch for testing at different ages (e.g., 28 days and test days). The preparation of the 3x6 in cylinders followed two steps according to the UHPC supplier recommendations as only half of the cylinder was initially filled with UHPC then the cylinder was hit on the ground 3-5 times to make sure that no voids or gaps were entrapped inside then finally the cylinder was filled with UHPC with repeating the same process. The cylinders were then left to cure besides the precast slab specimen under the normal site temperature.
The top surface of the cylinders typically has many voids that could affect the results of the compression testing. Hence, approximately a quarter-inch of the top surface was removed using a saw cutting machine that is readily available at UNR. Finally, both ends of the UHPC cylinders were smoothly ground using a manually operated grinding machine to ensure a smooth and flat surface at both ends of the UHPC cylinder.

A SATEC compression machine with a maximum compression capacity of 500 kips was then used to test the UHPC cylinders. The test cylinders were preloaded to approximately 40% of the expected compressive strength and then loaded to failure at 150 psi/sec. The UHPC cylinders preparation and compression testing are shown in Figure 3-5. Samples of the damage of the compression cylinders damage are shown in Figure 3-6. A summary of the measured compressive strength of the three UHPC batches at different ages is reported in Table 3-3. The reported compressive strength is the average of a minimum of three-cylinder samples tested at each testing date.

Figure 3-5: Preparation and compression testing of UHPC 3x6 in cylinders.
Figure 3-6: Sample of tested 3×6 in UHPC cylinders after compression tests at 28 Days.

Table 3-3: Measured compressive strength of the UHPC.

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Test Date</th>
<th>Batch No.</th>
<th>Machine load (lb)</th>
<th>Average load (lb)</th>
<th>Average stress (ksi)</th>
<th>Average / specimen</th>
</tr>
</thead>
<tbody>
<tr>
<td>Longitudinal</td>
<td>28 Days</td>
<td>1</td>
<td>180,740</td>
<td>184,820</td>
<td>26.15</td>
<td>26.15</td>
</tr>
<tr>
<td></td>
<td></td>
<td>1</td>
<td>186,695</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>1</td>
<td>187,025</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Transverse</td>
<td>28 Days</td>
<td>2</td>
<td>175,570</td>
<td></td>
<td></td>
<td>24.82</td>
</tr>
<tr>
<td></td>
<td></td>
<td>2</td>
<td>167,885</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>2</td>
<td>178,485</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Transverse</td>
<td>28 Days</td>
<td>3</td>
<td>181,135</td>
<td></td>
<td>176,855</td>
<td>25.02</td>
</tr>
<tr>
<td></td>
<td></td>
<td>3</td>
<td>167,960</td>
<td></td>
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<td></td>
<td>3</td>
<td>181,470</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Longitudinal</td>
<td>Test Day</td>
<td>1</td>
<td>205,995</td>
<td></td>
<td>195,300</td>
<td>27.63</td>
</tr>
<tr>
<td></td>
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<td></td>
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<td>1</td>
<td>187,665</td>
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<tr>
<td>Transverse</td>
<td>Test Day</td>
<td>2</td>
<td>211,390</td>
<td></td>
<td>211,770</td>
<td>29.96</td>
</tr>
<tr>
<td></td>
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<td>214,045</td>
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<td>2</td>
<td>209,865</td>
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</tr>
<tr>
<td>Transverse</td>
<td>Test Day</td>
<td>3</td>
<td>191,075</td>
<td></td>
<td>185,220</td>
<td>26.21</td>
</tr>
<tr>
<td></td>
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<tr>
<td></td>
<td></td>
<td>3</td>
<td>173,115</td>
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</table>

3.3 Poly-Methyl Methacrylate Polymer Concrete (PMMA-PC)

PMMA-PC has physical and mechanical material properties very comparable to that of the high strength concrete and a gray color same as the cementitious based types of concrete however the PMMA-PC is not cementitious based material. PMMA-PC is formed up by mixing Methyl
Methacrylate resin with a Benzoyl Peroxide initiator in addition to an optional graded aggregate of 3/8 x 3/16 nominal maximum size. Adding aggregate to the mix is only recommended for applications in which the thickness of the patch exceeds about 1.25 in and concrete should be used rather than a mortar (e.g., full-depth field joints in bridge decks). However, the PMMA-PC paste without aggregate can be useful for repairing and patching purposes.

The type of PMMA-PC used in this study is Transpo® T-17 PC and it is one of the commercial products of Transpo Industries Incorporation. The specified compressive strength of the PMMA-PC is 8.0-9.0 ksi after 24 hours and the specified tensile strength is 1.0-1.2 ksi as addressed on the technical data sheet of the T-17 PC product. The T-17 PC is designed for use on new construction and rehabilitation of bridge decks, expansion joints, bearing pads, airport runways, and other concrete structures due to its fast setting, high bond strength, and early strength. The T-17 PC has been used for decades around the nation and it is locally available and can be distributed all over the United States and Canada. The T-17 PC is commercially available and usually shipped in pre-packed bags and pails.

3.3.1 PMMA-PC Mixing Proportions

The PMMA-PC is commonly packed into two different components as follows: (1) the T-17 powder components are packed in 50-lbs bags and consists of a proprietary blend of sand, inert fillers, polymers, and initiators (see Figure 3-7a); (2) the T-17 liquid admixture is packed in 5-gallon pails and consists of a solvent-free 100% reactive, low viscosity methyl methacrylate (MMA) (see Figure 3-7b). As mentioned earlier, a small-sized coarse aggregate could be used inside the T-17 PC mix depending on the depth of the PC patch, the coarse aggregate is also commercially available and offered from the same material supplier in 50-lbs bags.
Figure 3-7: Transpo® T-17 PC components (a) T-17 powder, and (b) T-17 liquid component.

Table 3-4 shows the mixing proportions of the Transpo® T-17 PC based on the number of powder bags that are used on a single batch. Table 3-5 shows the mix components and quantities per cubic yard and cubic meter of the Transpo® T-17 PC. The T-17 PC was mixed at the fabrication yard outside the Engineering building at UNR using a rotary drum mixer that is readily available at UNR (see Figure 3-8). The rotary drum mixer has a total capacity of 1.5 ft³ (0.043 m³). The T-17 PC can also be mixed in large quantities up to 4.0 ft³. Due to the limited capacity of the utilized drum mixer, twelve batches were mixed to pour the field joints of three specimens in addition to the sample preparation for material characterization tests.
Figure 3-8: Rotary drum mixer used for mixing T-17 PC.

Table 3-4: Transpo® T-17 PC mixing proportions according to the number of T-17 powder bags.

<table>
<thead>
<tr>
<th>No. of T-17 powder bags per batch</th>
<th>1</th>
<th>2</th>
<th>3</th>
<th>4</th>
<th>5</th>
<th>6</th>
</tr>
</thead>
<tbody>
<tr>
<td>T-17 powder (lb)</td>
<td>50</td>
<td>100</td>
<td>150</td>
<td>200</td>
<td>250</td>
<td>300</td>
</tr>
<tr>
<td>T-17 liquid (lb)</td>
<td>5.72</td>
<td>11.44</td>
<td>17.16</td>
<td>22.88</td>
<td>28.6</td>
<td>34.32</td>
</tr>
<tr>
<td>T-17 liquid (gallon)</td>
<td>0.75</td>
<td>1.5</td>
<td>2.25</td>
<td>3</td>
<td>3.75</td>
<td>4.5</td>
</tr>
<tr>
<td>Aggregate (3/8 x 3/16) (lb)</td>
<td>25</td>
<td>50</td>
<td>75</td>
<td>100</td>
<td>125</td>
<td>150</td>
</tr>
<tr>
<td>Batch volume (ft³)</td>
<td>0.56</td>
<td>1.12</td>
<td>1.68</td>
<td>2.24</td>
<td>2.8</td>
<td>3.36</td>
</tr>
</tbody>
</table>

Table 3-5: Transpo® T-17 PC mixing proportions per cubic yard and cubic meter.

<table>
<thead>
<tr>
<th></th>
<th>kg/m³</th>
<th>lb/yd³</th>
<th>Percentage per weight (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>T-17 powder (lb)</td>
<td>3153</td>
<td>2411</td>
<td>61.95</td>
</tr>
<tr>
<td>T-17 liquid (lb)</td>
<td>361</td>
<td>276</td>
<td>7.10</td>
</tr>
<tr>
<td>T-17 liquid (gallon)</td>
<td>47</td>
<td>36</td>
<td>-</td>
</tr>
<tr>
<td>Aggregate (3/8 x 3/16) (lb)</td>
<td>1577</td>
<td>1205</td>
<td>30.95</td>
</tr>
</tbody>
</table>
3.3.2 PMMA-PC Mixing Procedure

The mixing procedure of the T-17 PC is divided into three main steps (see Figure 3-9). Prior to mixing, the inside of the mixer should be clean and dry, then the mixer should be pre-wet with a quart of the T-17 liquid component. The three mixing stages of the T-17 PC are as follows: (1) the T-17 liquid component is added to the mixer; (2) the T-17 powder bags are then added gradually over the course of ½ minute and then left to mix with the liquid for about 3 minutes until uniform consistency to ensure appropriate PC paste; (3) finally, the coarse aggregate is then added gradually to the mix over the course of ½ minute and then left for 1 minute to ensure uniform distribution of the aggregate inside the concrete.

![Figure 3-9: PMMA-PC mixing.](image)

It is recommended not to over mix the T-17 PC after adding the coarse aggregate to avoid segregation. The total mixing time of the PMMA-PC batch is about 5-6 minutes compared to that of the UHPC that ranges between 25-30 minutes. It is worth noting that one major advantage of the T-17 PC is that it has a wide application temperature range of 14-100 °F. However, the UHPC needs more setting time in cold environments and requires additional heat curing to speed up the strength gaining. In addition, UHPC requires replacement of the mixing water to the ice during hot temperatures that could limit the use of UHPC in some cases.

T-17 PC is a flowable material, hence there is no need to use a vibrator to avoid any coarse aggregate segregation. It is worth noting that the T-17 PC does not require any special considerations for surface finishing and it follows the same finishing procedure as the typical conventional concrete. Steel trowels, floats, or screeds can be used to obtain a closed PC surface while tinning or broom finishing is not recommended. The T-17 PC has less mixing time and hence there are no cold joints formed between successive batches.
3.3.3 PMMA-PC Material Testing

3.3.2.1 Flowability Test

A static and dynamic flow test was conducted on each PMMA-PC batch to ensure the appropriate flowability of the mix before pouring into the field joints. The flowability of the mix was measured using a spread cone mold in accordance with ASTM C1437/230 as shown in Figure 3-10. The measured static flow tests ranged between 6 in to 6.5 in for all the PMMA-PC batches while the dynamic flow tests range between 8.5 in to 9.5 in.

![Figure 3-10: PMMA-PC dynamic flow test.](image)

3.3.2.2 Compression Test

As previously mentioned, eleven PMMA-PC batches were used in this study. A 3x6 in cylinders were prepared randomly from all the batches to be tested at different ages and to test the consistency of this material. The 3x6 in cylinders were prepared in two steps as only half of the cylinder was initially filled with PMMA-PC and then the cylinder was hit on the ground 3-5 times to make sure that there were no voids or gaps inside the cylinder. Finally, the cylinder was filled with PMMA-PC and the previous process was repeated. The cylinders were then left to cure besides the precast slab specimen under the normal site temperature.

Unlike the UHPC, the PMMA-PC cylinders do not have air voids or weak crust at the top surface. Hence, there was no need to cut the top part of the cylinder. Instead, both ends of the PMMA-PC cylinders were smoothly ground using a manually operated grinding machine to ensure a smooth and flat surface at both ends.

Again, the SATEC compression machine with a maximum compression capacity of 500 kips and a Tinius Olsen machine was used to test the PMMA-PC cylinders according to the ASTM
standards C469. The test cylinders were preloaded for the first two cycles to approximately 40% of the expected ultimate compressive strength of the cylinder and then loaded up to failure at a rate of 250 lb/sec. The main aim of using this type of loading was to get the full stress-strain curve of this new material and determine the static modulus of elasticity. The compression testing of the PMMA-PC cylinders and a sample of the compression damage of cylinders at 7 Days are shown in Figure 3-11. A summary of the measured compressive strength of the PMMA-PC at different ages is reported in Table 3-6. The reported compressive strength is the average of a minimum of three-cylinder samples tested at each testing date.

Figure 3-11: Preparation and compression testing of the PMMA-PC 3x6 in cylinders.
Table 3-6: Measured compressive strength of the PMMA-PC.

<table>
<thead>
<tr>
<th>Test Date</th>
<th>Machine load (lb)</th>
<th>Average load (lb)</th>
<th>Average stress (ksi)</th>
</tr>
</thead>
<tbody>
<tr>
<td>7 Days</td>
<td>70,890</td>
<td>74,827</td>
<td>10.60</td>
</tr>
<tr>
<td></td>
<td>75,770</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>77,820</td>
<td></td>
<td></td>
</tr>
<tr>
<td>28 Days</td>
<td>64,460</td>
<td>61,833</td>
<td>8.75</td>
</tr>
<tr>
<td></td>
<td>61,880</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>59,160</td>
<td></td>
<td></td>
</tr>
<tr>
<td>70 Days</td>
<td>60,540</td>
<td>63,968</td>
<td>9.05</td>
</tr>
<tr>
<td></td>
<td>67,020</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>63,710</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>69,590</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>65,910</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>57,040</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

3.4 Conventional Concrete

The precast bridge deck panels were fabricated using conventional concrete with 5 ksi specified compressive strength. The conventional concrete was delivered by Cemex Incorporation. All the precast panels were cast using a single batch. A total number of 9 compression test cylinders were prepared from the conventional concrete and left on-site to cure till the specified testing date. The test cylinders were poured into three layers and a tamping steel rod was used to blow each layer 25 times to remove any entrapped air and finally the top surface is flattened using a spatula. The cylinders were of dimensions 6x12 in. Figure 3-12 shows the pouring of the conventional concrete and the preparation of the test cylinders. A Satic compression machine with a maximum compression capacity of 500 kips was used to test the concrete cylinders according to the ASTM C39. The test cylinders were loaded to failure at a rate of approximately 35 psi/sec. The compression testing of the conventional concrete cylinders and a sample of compression damage are shown in Figure 3-13.

A summary of the measured compressive strength of the conventional concrete cylinders at different ages is reported in Table 3-7. The reported compressive strength is the average of a minimum of three-cylinder samples tested at each testing date.
Figure 3-12: Preparation and compression testing of the PMMA-PC 3x6 in cylinders.

Figure 3-13: Compression testing of the conventional concrete 6x12 in cylinders.

Table 3-7: Measured compressive strength of the conventional concrete.

<table>
<thead>
<tr>
<th>Test Date</th>
<th>Machine load (lb)</th>
<th>Average load (lb)</th>
<th>Average stress (ksi)</th>
</tr>
</thead>
<tbody>
<tr>
<td>7 Days</td>
<td>76,965</td>
<td>82,725</td>
<td>2.93</td>
</tr>
<tr>
<td></td>
<td>80,490</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>90,720</td>
<td></td>
<td></td>
</tr>
<tr>
<td>28 Days</td>
<td>132,690</td>
<td>122,130</td>
<td>4.32</td>
</tr>
<tr>
<td></td>
<td>121,500</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>112,195</td>
<td></td>
<td></td>
</tr>
<tr>
<td>70 Days</td>
<td>141,600</td>
<td>147,210</td>
<td>5.2</td>
</tr>
<tr>
<td></td>
<td>145,525</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>154,505</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>
3.5 Reinforcing Steel

All the reinforcement that was used in this study is of grade 60 and it is conforming to the ASTM A706 Grade 60 specification. Direct tension tests were performed on 5 steel coupons from the #5 bars from the same reinforcement batch that was used in our study to determine the actual tensile properties of the reinforcing bars. The samples were tested using the Instron testing machine that has an ultimate tensile capacity of 56 kips (250 kN). The tensile strains were recorded throughout the test using a laser extensometer and two laser targets attached to each specimen to represent the gage length. All the tested samples were failed inside the gage length except the second specimen. Hence, most of the results were considered acceptable. The results of the tension tests are reported in Table 3-8. The measured yield and ultimate tensile strengths are 63.5 ksi and 103 ksi, respectively. The measured yield and ultimate tensile strains are 0.278 % and 20.08 %, respectively. The tensile stress-strain curve of sample number 4 is shown in Figure 3-14 for reference.

Table 3-8: Measured compressive strength of the conventional concrete.

<table>
<thead>
<tr>
<th>Sample</th>
<th>Ultimate tensile strength (ksi)</th>
<th>Tensile yield strength (ksi)</th>
<th>Tensile yield strain (%)</th>
<th>Tensile rupture strain</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>103.7</td>
<td>64.0</td>
<td>0.282</td>
<td>19.55</td>
</tr>
<tr>
<td>2</td>
<td>102.6</td>
<td>63.1</td>
<td>0.250</td>
<td>---</td>
</tr>
<tr>
<td>3</td>
<td>103.1</td>
<td>63.7</td>
<td>0.308</td>
<td>20.05</td>
</tr>
<tr>
<td>4</td>
<td>102.2</td>
<td>63.2</td>
<td>0.286</td>
<td>20.62</td>
</tr>
<tr>
<td>5</td>
<td>103.7</td>
<td>63.6</td>
<td>0.265</td>
<td>20.08</td>
</tr>
<tr>
<td>Average</td>
<td>103.1</td>
<td>63.5</td>
<td>0.278</td>
<td>20.08</td>
</tr>
</tbody>
</table>

Figure 3-14: Tensile stress-strain curve of the #5 reinforcing bars.
4. Deck Panel Test: Experimental Program

The general design of the test specimens in addition to the structural details of the deck slab specimens are reported in this chapter. This chapter also describes the fabrication process of the deck slab specimens, details of the instrumentation plan, and the static bending test setup.

4.1 Specimen Design

There are two main types of field joints in a full-depth precast bridge deck system as mentioned earlier depending on the orientation of the joints with respect to the traffic direction of the bridge. The first type of joint is the transverse joint and it runs perpendicular to the traffic direction while the second type is the longitudinal joint and it runs through the longitudinal axis of the bridge, parallel to the traffic direction. Both types of joints were experimentally tested in this study so that the results could be applied to many precast deck systems including DBT girder bridges and full-depth precast deck panel systems. Accordingly, the experimental program included testing of two different types of specimens as illustrated in Figure 4-1. A transverse specimen represents a part of a full-scale bridge deck system with an 8 in thick slab and a 6 in wide transverse joint at the middle. While the longitudinal specimen represents a section of a DBT girder bridge with a top flange of a 6 in thickness and a 6 in wide longitudinal joint at the middle.

![Figure 4-1: Illustration of test specimen’s orientation in a precast bridge deck system.](image)

The experimental program included testing a total of five full-scale specimens. The test parameters are as follows; joint orientation (e.g., transverse versus longitudinal), reinforcement lap splice type
(e.g., straight splice versus loop splice), slab thickness (e.g., 6 in versus 8 in), shear key shape (e.g., diamond versus modified diamond), and closure joint material (e.g., UHPC versus PMMA-PC). Three of the specimens emulated the transverse field joints, while the other two specimens emulated the longitudinal field joints in a DBT girder system. The general dimensions and test parameters of the five tested specimens presented and discussed herein are shown in Table 4-1.

Table 4-1: Test matrix and specimen details.

<table>
<thead>
<tr>
<th>Specimen Name</th>
<th>Joint orientation</th>
<th>Overall specimen dimensions (ft)</th>
<th>Closure material</th>
<th>Lap splice type</th>
<th>Joint width (in)</th>
<th>Splice length (in)</th>
</tr>
</thead>
<tbody>
<tr>
<td>S1-T-UHPC</td>
<td>Transverse</td>
<td>9 x 8 x 0.67</td>
<td>UHPC</td>
<td>Straight</td>
<td>6</td>
<td>5</td>
</tr>
<tr>
<td>S2-T-PC</td>
<td>Transverse</td>
<td>9 x 8 x 0.67</td>
<td>PMMA-PC</td>
<td>Straight</td>
<td>6</td>
<td>5</td>
</tr>
<tr>
<td>S3-T-PC-Loop</td>
<td>Transverse</td>
<td>9 x 8 x 0.67</td>
<td>PMMA-PC</td>
<td>U-bar</td>
<td>6</td>
<td>4.5</td>
</tr>
<tr>
<td>S4-L-UHPC</td>
<td>Longitudinal</td>
<td>8 x 7 x 0.5</td>
<td>UHPC</td>
<td>Straight</td>
<td>6</td>
<td>5</td>
</tr>
<tr>
<td>S5-L-PC</td>
<td>Longitudinal</td>
<td>8 x 7 x 0.5</td>
<td>PMMA-PC</td>
<td>Straight</td>
<td>6</td>
<td>5</td>
</tr>
</tbody>
</table>

The notations used for specimens’ names are as follows: “S#’’ stands for the number of the specimen; “T’’ represents the transverse field joints, while “L’’ represents the longitudinal field joints. All the specimen has the ordinary straight non-contact lap splice except the third specimen that has a “Loop” splice (U-bar). “UHPC” and “PC” represent the type of closure material inside the joints. All specimens were designed so that the main reinforcement was aligned in the transverse direction of the bridge while the secondary reinforcement was aligned in the longitudinal direction. Therefore, the main (transverse) reinforcement was connecting the precast deck slabs in the longitudinal joints while the secondary (longitudinal) reinforcement was connecting the precast deck slabs in the transverse joints.

The design of the specimens was done according to the AASHTO LRFD Bridge Design Specification (AASHTO, 2012). The design moments were determined based on the AASHTO equivalent strip method. The moment values provided in this method takes into account the largest values that could be experienced by the deck slabs with respect to the numerous loading conditions. The bridge example that was used to analyze the reinforced concrete deck slab for the transverse specimens has a cross-section consisted of five steel girders spaced at 12 ft on center and a deck slab of 8 in thickness. The slab thickness was determined based on the effective span length in addition to satisfying the minimum required depth of a 7 in as specified in the AASHTO (2012). The deck slab thickness of the longitudinal specimens was determined based on the typical AASHTO/PCI standard DBTs top flange thickness, i.e., 6 in (PCI bridge design manual, 3rd edition, 2011). The deck slabs were designed based on a class two exposure that controls the
spacing between the bottom main reinforcement to avoid the positive flexural cracking under the service limit state.

Many design and practical considerations were taken into account during the design of the field joints to enhance constructability, accelerate bridge deck erection, and enhance the structural performance and durability of the field joints. (1) Two layers of steel were used inside the joints to transfer shear and provide positive and negative moment continuity across the joint; (2) staggered reinforcement was used inside the connection; (3) all the precast slabs were joined together through passive field joints without using any post-tensioning or welding to the reinforcement inside the joints; (4) straight and loop bar splices were used to provide simpler reinforcement details inside the connection; (5) female-female diamond-shaped shear key shape was used given its superior durability and performance for transferring shear and bending as compared to flat ends shear keys.

The concrete cover for the main reinforcement was selected based on the minimum specified cover required for the unprotected reinforcing steel. The minimum bottom concrete cover is taken as 1 in for bar sizes up to No. 11 bar. While the minimum top concrete cover is taken as 2 in as specified for the deck surfaces that are subjected to normal conditions and located in a non-corrosive atmosphere.

The transverse reinforcement was placed at the outer layers of the deck slabs to maximize the flexural capacity in the transverse direction and increase the demands in the field joints. A Grade 60 reinforcing steel as specified by the ASTM A706 was used in this study in accordance with the AASHTO (2012). The conventional concrete used for casting of the precast deck panels has a 28-day specified compressive strength of 5 ksi which is larger than the specified minimum strength value of 4 ksi in the AASHTO (2012). In a bridge deck system rested over longitudinal main girders, the moment distribution over a transverse segment of the bridge deck due to the vertical loading can be as follows; negative moment over the girder locations due to the continuity of the bridge deck and a positive moment at the middle section in-between the adjacent girders. To finalize the geometry, boundary conditions, and dimensions of the specimens, the effective span length of the test specimens was selected based on the expected distance between the flexural inflection points of the original bridge deck slabs. Usually, in the case of the bridge decks, it is hard to determine the exact location of such bending inflection points because of the numerous loading conditions. Thus the effective span of the transverse test specimens was estimated based on the dead loads for simplicity and was adopted to be 8 ft, which is approximately two-thirds of the original span. While in the longitudinal specimens, 6 ft was used which corresponds to the DBT flange width of 8 ft. Since the effective span represents the positive flexural part of the slab and there is no negative moment in this part, the amount of negative reinforcement was determined to be as the main positive reinforcement.
Figure 4-2 shows the overall dimensions and reinforcement details of the first two transverse specimens where a straight bar splice was used. The outer concrete dimensions for both specimens are 8 by 9 ft. Both specimens were identical except for the closure material type. One specimen had a UHPC joint while the other specimen had a PMMA-PC joint. The specimens were set to be similar in all aspects except the closure material type in order to establish a direct comparison of the structural performance of both specimens. The lap splice length for the top and bottom mesh reinforcement is 5 in. The spacing between the bottom continuous reinforcement at the edge of the connection is 10 in, while the reinforcement at the middle of the connection became denser as the spacing decreased to 5 in.

Figure 4-3 shows the reinforcement details of the third transverse specimen that has a loop bar splice. This specimen is different from the other two transverse specimens by two main factors: (1) the shear key shape is different as a modified diamond-shaped shear key was implemented in this specimen with a 6 in width at the top surface and only a half-inch gap at the bottom surface. This type of shear key was proposed to eliminate the time and labor needed for the formwork erection at the bottom surface of the connection; (2) the straight non-contact lap splice was changed to loop (U-bar) splice to investigate the difference in the structural performance between using a straight lap splice and a loop splice inside the joint. U-bar splices can provide less splice length compared to the straight splice in addition to a slightly better performance in force transfer between precast panels than the straight splice (NCHRP, 2011). For a #5 bar and smaller made of conventional steel, the minimum bend diameter should be six times the bar diameter $6d_b$ (ACI 318-08, 2008). But because of the tight deck slab thickness and the required minimum hook condition of $6d_b$, #4 bars were used as the main reinforcement instead of the #5 bars.

The bend diameter used in this study is $5.5d_b$, this bend diameter slightly violated the minimum allowable values. This specified value for the minimum bend diameters was mainly established for two main reasons: feasibility of bending the reinforcement without breaking it and possible crushing of the concrete within the tight bend. By addressing these two factors, it is found that there is a weak possibility to crush the concrete within the bar bends as we are using a closure material with high compressive strength compared to the NSC. Furthermore, a literature study showed that it is feasible to use a $3d_b$ bend diameter within the deck field joints (NCHRP, 2011). One more advantage of using the #4 bars is the smaller spacing between reinforcement within the joint that can provide more load resistance and better load distribution across the precast panels (Li and Jiang, 2015).

Figure 4-4 shows the overall dimensions and reinforcement details of the two longitudinal specimens with a straight bar splice. They have smaller outer dimensions of 7 ft by 8 ft. The reason for the relatively smaller dimensions compared to the transverse specimens was mainly because of the typical dimensions of the DBT girders. Both specimens were identical except for the closure material type as one of them used UHPC while the other specimen used PMMA-PC. The lap splice length for the top and bottom mesh reinforcement was 5 in. The spacing between the bottom
continuous reinforcement at the edge of the joint was 7 in, while the reinforcement at the middle of the joint became denser as the spacing decreased to 3.5 in.

As illustrated in Figure 4-2 through Figure 4-4, all specimens consisted of two precast panels that are connected together through a diamond-shaped shear key with a 6 in width field joints. The selected shear key dimensions and the required lap splice lengths were selected based on the previous work by Graybeal (2010) on similar bridge deck slabs with UHPC closure joints that demonstrated a very good structural performance. Two bars were added inside the joint that runs through the joint length and is called lacer bars. These lacer bars were added to provide confinement of concrete within the joint and serve as restraints for the U-bar splice in addition to allowing ductile failure to the U-bar splice transverse specimen. The reinforcement of the precast panels runs through the joint in a staggered formation to transfer shear and moment between both panels.
Figure 4-2: Dimensions and structural details of specimens (S1-T-UHPC) and (S2-T-PC).
Figure 4-3: Dimensions and structural details of specimen (S3-T-PC-Loop).
Figure 4-4: Dimensions and structural details of specimens (S4-L-UHPC) and (S5-L-PC).
4.2 Fabrication Procedure

All specimens were constructed at the fabrication yard outside the EEL building at UNR. The construction followed a multi-phase process as briefly explained below:

4.2.1 Fabrication of Precast Deck Panels

The precast deck panels were constructed first. Each specimen consisted of two precast slab panels joined together with a field joint. This stage included doing the instrumentation of the slab reinforcing bars and installation, formwork preparation, and pouring the conventional concrete. All precast panels were cast using a single batch from a ready-mix concrete truck. A total number of 10 deck panels were fabricated on October 9, 2018. Figures 4-5 to 4-7 show the steps of constructing the precast deck panels. Figure 4-8 shows the clear concrete surface and the shear key shape after stripping off the formwork.

![Installation of the reinforcement strain gages.](image)

*Figure 4-5: Installation of the reinforcement strain gages.*

![Reinforcement assembly and formwork of the precast deck panels.](image)

*Figure 4-6: Reinforcement assembly and formwork of the precast deck panels.*
Figure 4-7: Casting of the precast deck panels and concrete samples preparation.

Figure 4-8: Close-up views of the constructed deck panels and the shear key shapes.
4.2.2 Precast Panel Alignment

The precast slab panels were cured under normal conditions for approximately three weeks before the slabs were moved. Each pair of two adjacent panels was aligned together with a gap of 6 in width to allow for pouring the pouring of the field joints later. Figure 4-9 shows the alignment and assembly of the test specimens.

![Figure 4-9: Alignment and assembly of test specimens at UNR.](image)

4.2.3 Shear Key Preparation

This construction stage included the installation of the lacer bars inside the joints, installation of the field joint side formwork, and cleaning the surface of the shear key using strong air pressure to remove dust and debris. A backer rod with 1 in diameter was added at the bottom of the field
joint of specimen “S3-T-PC-Loop” to fill the 0.5 in gap between the bottom tips of the precast panels. Finally, concrete strain gages were installed inside the joint near the top surface.

4.2.4 Pouring UHPC

Three UHPC batches were used in casting the field joints of two specimens “S1-T-UHPC” and “S4-L-UHPC” on November 14, 2018. The shear key surface was sprinkled with water about half an hour before pouring UHPC to prevent the old precast panels from absorbing the water out of the freshly mixed UHPC. The mixing process for UHPC was done using a high-shear mixer that is readily available at UNR. Figure 4-10 shows the casting of UHPC inside the field joints.

![Figure 4-10: Casting of UHPC and sample preparation.](image)

4.2.5 Pouring PMMA-PC

The mixing process of the PMMA-PC was also done at the fabrication yard using a 1.5 ft³ rotary drum mixer. A primer liquid was initially applied to the shear key surfaces about 10 minutes before pouring PMMA-PC. The primer was applied using a brush to increase the adhesion between the PC and the old precast concrete panels. Twelve batches of the PMMA-PC were used in casting the field joints of the other three specimens “S2-T-PC”, “S3-T-PC-Loop” and “S5-L-PC” in addition to several types of test samples. The pouring of the PMMA-PC joints took place on November 27, 2018. The application of the primer was recommended by the manufacturer of the PMMA-PC. The commercial name of the primer is “T-41”. After then, the primer was left for approximately 10 minutes to harden and to be stickier before pouring the PMMA-PC inside the field joints. Figure 4-11 shows the casting of the PMMA-PC inside the joints.
4.3 Test Setup

The experimental testing was conducted at the EEL at UNR. The test setup in this study was prepared to include static testing of five full-scale bridge deck specimens in three-point bending assembly with a single point loading applied adjacent to the connection. The test setup assembly included the alignment of two seat beams parallel to the longitudinal direction of the bridge. The spacing between the seat beams was adjusted to mimic the assumed spacing between the inflection points of the deck slabs as will be illustrated later in Figures 4-12 and 4-14.

A long strip of unreinforced 2 in thick and 8 in wide rubber pad was used to allow for rotation of the deck slab at the simple support axis. As previously mentioned, precast bridge decks usually span across multiple girders, which provides negative moment regions over the girders and positive moment almost halfway between the girders. The test setup was adjusted to provide simply supported deck slabs with allowing some rotations at the ends only to represent a positive moment case and the portion of the deck slab transverse span between two adjacent bending inflection points. The test setup did not consider any axial restraint, i.e., arching action, at the ends and ignored the associated contribution of this arching action to the load-bearing capacity of the bridge deck. The applied mid-span vertical static load simulates the types of stresses usually imparted through the field joints to provide continuity of load distribution between the precast panels. Flexure and shear are the main stresses that are usually required to be transferred through the connection. Thus the point load position was determined to be at the mid-span and adjacent to the field joint.

The load was applied using a 220 kip hydraulic actuator (see Figures 4-12 through 4-15 for details) through a 1 in thick elastomeric rubber pad of 20×10 in plan dimensions. The loading pad dimensions were determined based on the contact area of the typical AASHTO truck wheel. It is noted that the rubber pad was backed by a 1 in thick steel plate to achieve uniform load distribution.
4.3.1 Test Setup of the Transverse Specimens

Figure 4-12 shows a schematic drawing of the test setup of the transverse specimens. While Figure 4-13 shows a photograph of the test setup used in this study at UNR Laboratory. Figure 4-13 also shows the seat beams and seat rubber pads, loading steel plate, and loading rubber pad. Three deck slabs were designed to mimic the full-depth transverse joint between the precast deck panels in a multi-girder bridge. The objective of the test is to investigate the local flexural behavior of the interior transverse joints in a non-composite deck/girder bridge. The position of the interior transverse joint is located away from the main girder supports. The transverse joints over the girder supports (negative moment region) in a composite deck/girder bridge are mainly subjected to uniform tensile stresses throughout the depth of the connection. These types of stresses along with any global flexural behavior experienced by composite deck/girder bridges were not investigated in this study. The transverse specimens were supported over the entire 8 ft length seat beams. The spacing between the seat beams was adopted to be 8 ft to represent the assumed effective span of the transverse specimens as mentioned earlier.
Figure 4-12: Schematic drawing of the transverse specimen’s test setup.
Figure 4-13: Photograph of the actual transverse specimen’s test setup at UNR.
4.3.2 Test Setup of the Longitudinal Specimens

Figure 4-14 shows a schematic drawing of the longitudinal specimen’s test setup. While Figure 4-15 shows a photograph of the actual test setup used in this study at UNR Laboratory. Two deck slabs were designed to mimic the top flange section of the DBT girder bridges. The DBT girders are connected in the longitudinal direction of the bridge through field joints at the top flanges. The objective of the test is to investigate the local flexural behavior of the longitudinal field joints in a DBT girder bridge without considering the composite action between the DBT flanges and the girder's web parts. Only the local flexural behavior of the deck slabs was investigated by applying a patch load adjacent to the joint to investigate the performance of the longitudinal joint in transferring the shear and flexure stresses across the different DBT girders. The longitudinal specimens were supported over the entire 8 ft length seat beams. The spacing between the seat beams was adopted to be 6 ft to represent the assumed effective span of the longitudinal specimens as mentioned earlier. The field joint was aligned parallel to the seat beams in the longitudinal direction of the bridge. The portion of the deck slab, which spans transversely between two adjacent bending inflection points was implemented in this setup.
Figure 4-14: Schematic drawing of the longitudinal specimen’s test setup.
Figure 4-15: Photograph of the actual longitudinal specimen’s test setup at UNR.
4.4  Loading Protocol

The loading protocol is divided into five cycles that combined four cycles of loading and unloading then monotonic loading until failure. The first two cycles included loading of the deck slabs up to a certain load that represents approximately the cracking load of the deck slabs, then unloading. The second two cycles included loading of the deck slabs up to a load limit which is slightly below the AASHTO ultimate load, then unloading. The last cycle included the monotonic loading of the deck slabs until failure. The first four cycles were applied using the force-control loading method at a loading rate of 5 kips/min. The same loading rate was also used for the unloading to avoid any sudden load drops or loss of contact at the loading pads. Then, a displacement control at a rate of 0.075 in/min was used for the monotonic loading of the last cycle to capture the post-peak response through failure. The measured mid-span deflections of the specimens were used for the closed-loop displacement control applied for the last cycle.

The reasons for implementing the first four loading and unloading cycles in the protocol were to investigate the behavior of the deck slab specimens under service loads, determine the crack pattern at the bottom of the specimens at earlier stages of loading, and estimate the average stiffness of the deck slabs based on the load versus mid-span displacement relationships of the first four cycles. Figures 4-16 and 4-17 show the loading protocol of the transverse and longitudinal specimens, respectively. For the transverse specimens, the first two cycles included loading of the specimen with a 20 kips axial load, then applying 40 kips in the third and fourth cycles. While the last cycle included loading of the specimen up to failure. While for the longitudinal specimens, the first two cycles included loading of the specimen with a 15 kips axial load, then applying 30 kips in the third and fourth cycles. While the last cycle included the loading of the specimen up to failure.
Figure 4-16: Loading protocol of the transverse specimens.

Figure 4-17: Loading protocol of the longitudinal specimens.
4.5 Instrumentation Plan

The overall structural response of the deck slabs during the static flexural tests was captured through different sets of instrumentation including reinforcement and concrete strain gages, string potentiometers, LVDTs, and video cameras.

4.5.1 Instrumentation of the Transverse Specimens

The instrumentation plan for the first two identical straight splice transverse specimens included a total of 44 strain gages for reinforcing steel distributed among the longitudinal and transverse reinforcement, three concrete strain gages embedded within the field joint, nine string potentiometers, and six LVDTs for horizontal displacement measurements at the joint interface. Figure 4-18 shows the instrumentation plan for specimens “S1-T-UHPC” and “S2-T-PC”.

The third transverse specimen with loop splice had 46 strain gages for reinforcing steel distributed among the longitudinal and transverse reinforcement, three concrete strain gages embedded within the field joint, nine string potentiometers, and three LVDTs for horizontal displacement measurements at the joint interface. Figure 4-19 shows the instrumentation plan for specimen “S3-T-PC-Loop”.

The strain gages that are mounted on the bottom transverse reinforcement were installed to capture the yielding of the bars due to the bending of the deck slabs and to monitor the load distribution across the connected precast panels. The distribution of the strain gages was denser near the loading position due to the high local flexural stresses expected in this region. For the strain gages of the longitudinal reinforcement, each instrumented bar had two strain gages inside and just outside the joint region to interpret any bar slip if happened. Moreover, the strain gages inside the joint were mainly installed to determine whether the bars yielded within the joint and verify if that the embedded length satisfied the required development length within the closure joint material.

The concrete strain gages were embedded near the top surface of the field joint to capture the concrete compressive strains. The displacement transducers were attached to the bottom of the deck slabs at the joint interface to measure the horizontal displacements between the precast deck panels and the field joint, i.e., monitor the opening of the bottom side of the joint. The string potentiometers were mounted to the bottom side of the deck slabs to measure the vertical deflection of the specimens. The vertical deflection of the specimens was recorded throughout the test at nine different locations, which were distributed uniformly across the span of the specimens. The main aim of using these string potentiometers is to check the uniformity of the load distribution of the specimens. Several cameras were also used below the specimens to capture the crack progress and pattern throughout the test.
Figure 4-18: Instrumentation plan of specimens “S1-T-UHPC” and “S2-T-PC”.

Instrumentation:
- Number of r.f.t strain gages: 44
- Number of concrete strain gages: 3
- Number of string potentiometers: 9
- Number of LVDTs: 6
**Instrumentation:**

- Number of r.f.t strain gages: 46
- Number of concrete strain gages: 3
- Number of string potentiometers: 9
- Number of LVDTs: 6

*Figure 4-19:* Instrumentation plan of specimen “S3-T-PC-Loop”.
4.5.2 Instrumentation of the Longitudinal Specimens

The instrumentation plan of the two identical straight splice longitudinal specimens included a total of 41 strain gages for the reinforcing steel distributed among the longitudinal and transverse reinforcement, three concrete strain gages embedded within the field joint, nine string potentiometers, and six LVDTs for horizontal displacement measurements at the joint interface. Figure 4-20 shows the instrumentation plan for specimens “S4-L-UHPC” and “S5-L-PC”. A photograph of the actual instrumentation plan at UNR is shown in Figure 4-21.

**Figure 4-20:** Instrumentation plan of specimens “S4-L-UHPC” and “S5-L-PC”.

**SP2 & SP4**

**Instrumentation:**
- Number of r.f.t strain gages: 41
- Number of concrete strain gages: 3
- Number of string potentiometers: 9
- Number of LVDTs: 6
Figure 4-21: Photograph of the actual instrumentation plan at UNR.
5. Deck Panel Test: Results and Discussion

This chapter presents the experimental test results of the three transverse specimens and the two longitudinal specimens. The precast deck slabs were tested for flexure under static loading as illustrated in the previous chapter. The specimens were simply supported at the two ends to form a one-way simple span deck slab with a vertical load applied through a simulated typical AASHTO wheel patch that was placed adjacent to the field joint near the mid-span. This chapter illustrates the structural behavior of the tested specimens in addition to the performance of the field joints up to failure.

5.1 Test Results of the Transverse Specimens

The flexure static testing of the transverse specimens was done according to the process described previously. The testing of each specimen was completed over the course of 2 hours. Again, the loading process included four cycles of static axial loading and unloading. After each cycle, the test was stopped to inspect the flexural cracks at the bottom of the slabs and visually determine if there are any signs of interface deboning that happened between the precast slabs and the field joints. The test data was filtered and processed to determine the structural performance of the deck slabs. The results of each specimen are separately presented to provide a detailed illustration of the structural performance of the precast deck slabs throughout the different loading stages. Finally, a comparison between all transverse specimens was done and reported in Chapter 6 to illustrate the difference between using UHPC and PMMA-PC as a closure material inside the field joints in addition to studying the difference in behavior between using loop splice and straight splice inside the joints.

5.1.1 Test Results of Specimen “S1-T-UHPC”

The measured peak load of specimen “S1-T-UHPC” was 117.9 kips at 2.33 in mid-span vertical displacement. The observed mode of failure for the UHPC transverse specimen is yielding in the main reinforcement due to flexure accompanied with crushing of concrete at the top of the west precast panel of the specimen. Deep and wide flexural cracks were observed at the bottom face of the precast panels interrupted by two main localized cracks in the UHPC at the measured peak load. Figure 5-1 shows an overview of the observed extensive flexural cracking at the bottom side of specimen “S1-T-UHPC” at the peak load. Interface cracking at the top surface of the slab was then observed between the east precast panel and the UHPC joint at the peak load as shown in Figure 5-2. The depth and length of this interface de-bonding were then increased as the applied displacements increased. After reaching the peak load, a series of sudden drops in the force capacity was observed along with excessive flexural cracking at the bottom and crushing of concrete of the deck panels at the top.
The test was stopped after noticing a sound of a bar rupture after losing approximately 20% of the peak load, which was found to indicate the rupture of the bottom transverse bar inside the field joint. Figure 5-3 shows the observed concrete crushing at the top of specimen “S1-T-UHPC” at the end of the test. Figure 5-4 shows the conclusion of the flexural cracks at the bottom of specimen “S1-T-UHPC” at the end of the test.

Figure 5-1: Crack pattern at the bottom side of specimen “S1-T-UHPC” at peak load.
Figure 5-2: Interface cracking at the top side of specimen “S1-T-UHPC” at the end of the test.

Figure 5-3: Concrete crushing at the top-west side of specimen “S1-T-UHPC” at the end of the test.
Figure 5-4: (a) Crack pattern at the bottom of specimen “S1-T-UHPC” at the end of the test, (b) concrete bottom cover spalling, and (c) crack localization in the UHPC field joint.

The load-deflection relationships at three different locations at the mid-span of specimen “S1-T-UHPC” are presented in Figure 5-5. Similarly, the load-deflection relationships based on the measurements from the other string potentiometers located at quarter spans of specimen “S1-T-UHPC” are presented in Figure 5-6.
It can be seen that the flexural capacity of the specimen exceeded the two code limit states, i.e., service and ultimate loads specified in the AASHTO using the equivalent strip method. Therefore, the proposed UHPC field joint was capable of providing an ABC precast deck system that emulates the behavior of conventional CIP deck systems in terms of strength requirements.
The load-deflection relationships shown in Figure 5-5 can be divided into different regions of behavior. The first region of behavior represents the elastic behavior of the specimen when the applied load was approximately below 20 kips. During this stage, only hair flexural cracks in the longitudinal direction were observed at the bottom face of the slab. A hair transverse crack was also observed that was located approximately 5 in apart from the edge of the joint and runs parallel to the joint in the loaded west side of the slab. No cracks were observed at the unloaded east side of the slab. No interface de-bonding or concrete crushing was observed during the “service” level loading. No cracks were also observed in the field joint because of the higher tensile strength of the UHPC as compared to the conventional concrete.

The second region of behavior in defining the load-deflection relationship represents the onset and progression of the inelastic behavior as the applied load exceeded the elastic load. This started in the third and fourth cycles of loading around the 40 kips vertical load when the flexural stiffness decreased, and the residual displacements increased. The cracks at this level were still narrow and the main reinforcement was not yet yielded. It is worth noting that there were no flexural or interface cracks observed in the field joint until reaching the AASHTO specified ultimate load as shown in Figure 5-7.

*Figure 5-7:* (a) Crack pattern at the bottom side of specimen “S1-T-UHPC” after the 2nd cycle, and (b) crack pattern after the 4th cycle.
The slight inelastic response continued as the load increased to approximately 85 kips. As the load increased beyond this limit, the third region of the behavior of the load-deflection relationship is defined by the significant reduction in stiffness as the specimen started to soften due to yielding of the middle and west side transverse reinforcement in the bottom mesh. As a result, deep and wide flexural cracks were observed on the bottom side of the slab. No bar slip was observed until reaching the peak load. Following the peak load capacity, the fourth region of behavior is defined by a global softening behavior where the load capacity started to decrease accompanied by larger displacements. It is noted that the goal of pushing the specimen to such high levels of loads and displacements beyond code-required limits was to get a better understanding of the modes of failure and check if the different system components stood intact till the end of the test. It is also observed that the measured deflections of the west side of the specimen and deflections along the field joint were almost identical as can be seen in Figures 5-5 and 5-6. However, the unloaded side of the specimen (east side) attained lower deflections which were expected because of the asymmetry of the specimen loading in the longitudinal direction, i.e. east-west direction.

Another interpretation for this observation is that the field joint did not transfer the full applied loads between the two sides of the slab as there is a discontinuity in the reinforcement between both sides. Moreover, this difference in deflections increased with increasing the applied load due to the influence of the reduction in stiffness of the slabs after the propagation of excessive cracking. This is consistent with results from a previous finite element study (NCHRP, 2011) which showed that the shear and moment forces on the field joint are greatly reduced due to the stiffness reduction accompanied by cracking of the specimen. Figure 5-8 shows the difference between the mid-span deflections and the east and west side deflections versus the applied vertical load of specimen “S1-T-UHPC”.
Figure 5-8: Difference between mid-span deflections and deflections of the east and west sides of specimen “S1-T-UHPC”.

The difference in deflection readings between the middle and the west side is minimal, while the difference in deflections between the middle and east side are comparably higher. The difference between the east and west side deflections at the measured peak load is 0.75 in. The difference between the deflections of the middle and east unloaded side of the slab can give a good indication about the performance of the field joint and the ability of the UHPC joint to transfer the straining actions between both sides of the precast segments. As the difference in deflections decreases, the load distribution over the slab and the joint performance in transferring the loads becomes better.

The load versus strain readings of the bottom transverse reinforcement are shown in Figures 5-9 to 5-11 and they are discussed here to provide more understanding of the flexural behavior of the specimen and the reason for failure. It is noted that some of the strain gages were damaged during the test especially at larger load levels when concrete damage led to strain gages or wire damages.
Figure 5-9: Reinforcement strain gage readings of the bottom transverse bars in the west precast panel of specimen “S1-T-UHPC”.
Figure 5-10: Reinforcement strain gage readings of the bottom transverse bars in the east precast panel of specimen “S1-T-UHPC”.
It can be seen from Figures 5-9 to 5-11 that the largest strain values were recorded in the middle bars near the load application. The onset of yielding was observed in the bar inside the joint at 65 kips. This was followed by yielding in the bars adjacent to the joint at almost 69 kips and 78 kips for the west and east sides, respectively. The strain values at the same applied load level were different between the east and west sides of the slab. The west side had higher values of strains than the east side. Most of the other bars on the west side and the middle part of the slab yielded at approximately 78 kips. After that, the load-deflection behavior of the slab significantly changed to suffer large deflection increments corresponding to small force increments as previously shown in Figure 5-5. Thus, when yielding happened in the reinforcement of the west side while the east side reinforcement was still not yielded, the difference in deflections between the east and west side of the slab increased significantly.

Rebar rupture was indicated at the end of the test in the bottom transverse rebar that was located inside the UHPC joint as can be seen from the strain readings. This bar rupture happened approximately when the specimen reached 100 kips of load and mid-span displacement of 3.75 in where a sudden drop in the load capacity was observed. Finally, a key observation from the behavior depicted in the previous figures is that none of the joint or slab transverse bottom reinforcement yielded before reaching the ultimate and service loads specified in the AASHTO, i.e., the deck system remains elastic up to the code limits as required.

The interface crack width between the field joint and the precast panels was monitored through the test using six LVDTs as mentioned earlier. Figure 5-12 shows the horizontal displacement readings from these LVDTs versus the applied load for specimen “S1-T-UHPC”.

Figure 5-11: Reinforcement strain gage readings for the bottom transverse bar inside the field joint of specimen “S1-T-UHPC”.
The largest crack width was located at the mid-span of the specimen. The reason for that behavior is that the mid-span zone was subjected to maximum applied loads and maximum vertical deflection. The crack opening at the west side was bigger than that of the east side in the middle of the specimen. It is worth noting that the readings from the LVDTs do not accurately represent the interface crack width as these readings were greatly affected by the vertical displacement of the specimen. Hence, the LVDTs were measuring the inclined distance instead of the horizontal distance. But we are reporting these values as a reference for further testing or numerical analysis.

At the early stages of the applied load where the displacements were minimal, the readings from the LVDTs were more accurate in representing the interface crack width. However, the readings were greatly affected by the vertical displacements and the severe crack openings in the slab when approaching the peak load.

The results from the LVDTs are shown herein to investigate the bond performance between the UHPC joint and the precast slab and to compare the crack width with that of the maximum allowable limits specified in the AASHTO LRFD Article 5.6.7. The AASHTO specifies a maximum spacing between the reinforcing bars to control and limit the flexural cracks. These maximum spacing limits were determined based on the maximum allowable crack width of 0.017 in for class (1) exposure and 0.013 in for class (2) exposure. Class (2) exposure is typically used for situations in which the concrete is subjected to severe corrosion conditions. The specimen reached the maximum allowable crack assuming a class (2) exposure at approximately 60 kips of applied load that is almost 1.25 times the specified ultimate design load of the deck slab and 2.25...
times the service load. This is a good conclusion that the interface bond between the UHPC and the precast segments was perfectly satisfying the design requirements and code allowable limits.

5.1.2 Test Results of Specimen “S2-T-PC”

The measured peak load of specimen “S2-T-PC” was 113.2 kips at 2.53 in mid-span vertical deflection. The observed mode of failure for the PMMA-PC transverse specimen was similar in the beginning to the UHPC specimen with yielding in the main reinforcement due to flexure. However, a rebar rupture was observed at the peak load that caused a sudden drop in the ultimate load capacity of the specimen. Crushing of concrete was then observed at the top surface of the loaded west side of the specimen and continued to propagate through the joint until reaching the unloaded east side. Deep and wide flexural cracks were observed at the bottom face of the precast slabs, these cracks continued to propagate through the joint when approaching the peak load. Figure 5-13 shows an overview of the observed extensive flexural cracking at the bottom side of the specimen at the peak load. No interface cracking was observed at the top surface of the slab between the precast slab and the PMMA-PC joint until reaching the peak load. While a short-length interface crack of approximately 5 in wide was observed by the end of the test at the bottom surface of the slab between the unloaded part and the PMMA-PC joint (see Figure 5-14). The bar rupture that caused the sudden drop into the ultimate load capacity of the specimen was anticipated on one of the transverse bottom bars. After that drop in capacity, the stiffness of the slab decreased steadily associated with excessive flexural cracking at the bottom and crushing of conventional concrete at the top surface of the slab.

The test was again stopped after losing approximately 20% of the peak load. Figure 5-15 shows the overall damage and final state of specimen “S2-T-PC” at the end of the test. Figure 5-14 shows the crack pattern and damage at the bottom side of specimen “S2-T-PC”. It is worth noting that the specimen had two transverse cracks located 5 in away from the joint and parallel to the joint before testing. These cracks are probably interpreted due to improper storing or lifting. It was observed that the PMMA-PC field joint reached the AASHTO ultimate load without cracking due to the high tensile properties of the PMMA-PC that makes this material suitable in developing more durable deck field joints.
Figure 5-13: Overview of the bottom side of specimen “S2-T-PC” at peak load.
Figure 5-14: (a) Crack pattern at the bottom of specimen “S2-T-PC” after 3.9 in displacement, (b) spalling of the bottom concrete cover, and (c) bottom interface crack.
Figure 5-15: Overview of the concrete crushing at the top surface of specimen “S2-T-PC” at the end of the test.

The load-deflection relationships at three different locations at mid-span are presented in Figure 5-16. Similarly, the load-deflection relationships measured by the other six-string potentiometers located at quarter spans are presented in Figure 5-17.
It can be seen that the data acquisition readings of the third cycle had some noise that partially affected the quality of the load-deflection relationships. The flexural capacity of the specimen exceeded the two code limit states, i.e., service and ultimate loads specified in the AASHTO LRFD using the equivalent strip method. Therefore the PMMA-PC field joint was capable of providing
an ABC precast deck system that emulates the behavior of conventional CIP deck systems in terms of strength requirements.

The load-deflection relationships shown in Figure 5-16 can be divided into different regions of behavior. The first region of behavior represents the elastic behavior of the specimen when the applied load was approximately below 20 kips. During this elastic range, only hair longitudinal flexural cracks were observed at the bottom face of the slab. Two-hair transverse cracks located approximately 5 in apart from the edge of the joint and ran parallel to the joint in the west and east sides of the specimen were also observed. These cracks were initially identified before testing due to improper storage. Some cracks were also observed on the unloaded (east) side of the slab. No interface de-bonding or concrete crushing was observed during the service level loading. No cracks were also observed in the field joint because of the higher tensile strength of the PMMA-PC as compared to the conventional concrete.

The second region of behavior in defining the load-deflection relationship represents the onset and progression of the inelastic behavior as the applied load exceeded the elastic load. This inelastic behavior started in the third and fourth cycles of loading around the 40 kips vertical load when the flexural stiffness decreased and the residual displacement increased. The crack pattern at this level did not change from that of the previous 20 kips cycles, also at this level the cracks were still narrow and the main reinforcement was not yet yielded. It is worth noting that there were no flexural or interface cracks observed in the field joint until reaching the AASHTO specified ultimate load as shown in Figure 5-18.
Figure 5-18: (a) Crack pattern at the bottom side of specimen “S2-T-PC” after the 2nd cycle, and (b) crack pattern after the 4th cycle.

The slight inelastic response continued as the load increased to approximately 75 kips. After that, the third region of the load-deflection curve is defined by the significant reduction in stiffness as the specimens started to soften due to yielding of the middle and west side transverse reinforcement in the bottom mesh. As a result, deep and wide flexural cracks were observed on the bottom side of the slab. No bar slip was observed until reaching the peak load.

Following the peak load capacity, the fourth region is defined by a global softening behavior where the load capacity started to decrease with larger displacements. A sudden drop was observed at the peak load due to a bar rupture. It was hard to trace where the bar rupture happened due to the damage that happened to some of the reinforcement strain gages before reaching the peak load. The descending branch of the load-deflection relationship was accompanied by concrete crushing at the top surface of the slab, concrete cover spalling, and finally, a punching shear failure was observed below the loading pad when the elastomeric pad was compressed to a limit that allowed the loading steel plate to punch through the concrete. The measured deflections of the west side of the specimen and the deflections along the field joint were almost identical as can be seen in Figures 5-16 and 5-17. However, the unloaded side of the specimen (east side) attained smaller deflection values. Figure 5-19 shows the difference between the mid-span deflections and the east and west side deflections versus the applied vertical load of specimen “S2-T-PC”.

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Figure 5-19: Difference between mid-span deflections and deflections of the east and west sides of specimen “S2-T-PC”.

The difference in the deflection readings between the middle and the west side of the specimen is minimal, while the difference in deflections between the middle and east side are comparably higher. The difference between the east and west side deflections at the measured peak load is 0.75 in.

The load versus strain readings of the bottom transverse reinforcement are shown in Figures 5-20 to 5-22 and they are discussed here to provide more understanding of the flexural behavior of the specimen and the reason for failure. It is noted that some of the strain gages were damaged during the test especially at larger load levels when concrete damage led to strain gages and wire damages.
**Figure 5-20:** Reinforcement strain gage readings of the bottom transverse bars in the west precast panel of specimen “S2-T-PC”.
**Figure 5-21:** Reinforcement strain gage readings of the bottom transverse bars in the east precast panel of specimen “S2-T-PC”.
It is worth noting that some of the strain gages were damaged during the test and they were reported in the previous figures. It can be seen from Figures 5-20 to 5-22 that the largest strain readings were recorded in the middle bars near the applied load location. The onset of yielding was observed on the bar inside the joint at 46 kips. This was followed by yielding in the bars adjacent to the joint at almost 72 kips and 75 kips for the west and east sides, respectively. The strain values at the same applied load level were different between the east and west sides of the slab. The west side had higher values of strains than the east side until reaching the peak load where the east side had more strains due to the redistribution of forces as a result of the bar rupture. Most of the other bars on the west side and the middle part of the slab yielded at approximately 75 kips. After that, the load-deflection behavior of the slab significantly changed with large deflection increments corresponding to small force increments as previously shown in Figure 5-16. It can also be concluded from Figure 5-19 that when yielding happened in the reinforcement of the west side at 73 kips while the east side reinforcement was still not yielded, the difference in deflections between the east and west side of the specimen increased significantly.

Finally, a key observation from the behavior depicted in the previous figures is that none of the joint or slab transverse bottom reinforcement yielded before reaching the ultimate and service loads specified in the AASHTO, i.e., the deck system remains elastic up to the code limits as required. The only bar that yielded before the AASHTO ultimate load was the bottom transverse bar that yielded slightly below the limit at approximately 46 kips.
The interface crack width between the field joint and the precast panels was monitored through the test using six LVDTs as mentioned earlier. Figure 5-23 shows the horizontal displacement readings from these LVDTs versus the applied load.

![Interface Crack Width Diagram](image)

**Figure 5-23**: Interface crack opening versus applied load for specimen “S2-T-PC”.

It was observed that the crack width from the west side at mid-span is significantly higher than anywhere else along the joint. The reason for this large crack width is that the mid-span zone is subjected to the maximum applied load and maximum vertical deflection. Furthermore, the readings of the “LVDT W” were significantly affected by the flexural cracks at the bottom of the specimen as it was mounted over one of the wide flexural cracks that run parallel to the joint in the transverse direction as shown in Figure 5-24.

The interface bonding can also be investigated through the other LVDTs along the joint (i.e., LVDT NW). It was observed that the interface crack opening reached the maximum allowable crack assuming a class (2) exposure at approximately 48 kips of vertical load which is slightly higher than the ultimate load of the slab. This is a good conclusion that the interface bond between the PMMA-PC and the precast panels was sufficient to prevent interface cracking up to the AASHTO design load requirements.
5.1.3 Test Results of Specimen “S3-T-PC-Loop”

The measured peak load of specimen “S3-T-PC-Loop” was 122.5 kips at 2.63 in mid-span deflection. Similarly, the observed mode of failure for the PC-Loop transverse specimen was yielding in the main reinforcement due to flexure accompanied with crushing of the concrete at the top of the loaded west side of the slab. Figure 5-25 shows an overview of the observed extensive flexural cracking at the bottom side of the specimen at peak load. After reaching the peak load, concrete crushing started to propagate across the east and west sides of the slab with excessive crushing damage observed on the west side as shown in Figure 5-26. Deep and wide flexural cracks were observed at the bottom face of the precast panels, these cracks continued to propagate across the joint after reaching the peak load as shown in Figure 5-27. No interface cracks were observed at the top surface of the slab between the precast panel and the PMMA-PC joint at peak load. A series of drops in the force capacity was indicated beyond the peak load associated with excessive flexural cracking at the bottom and crushing of the conventional concrete at the top. The test was again stopped after losing approximately 20% of the peak load.
Figure 5-25: Crack pattern at the bottom side of specimen “S3-T-PC-Loop” at peak load.
Figure 5-26: Concrete crushing at the top of specimen “S3-T-PC-Loop” at the end of test.
Figure 5-27: Crack pattern at the bottom of specimen “S3-T-PC-Loop” at the end of the test.

The load-deflection relationships at mid-span measured at three different locations are presented in Figure 5-28. The load-deflection relationships measured by the other six string potentiometers located at quarter spans are presented in Figure 5-29.
The flexural capacity of the specimen exceeded the two code limit states, i.e. service and ultimate loads specified in the AASHTO LRFD using the equivalent strip method. Therefore, the proposed PMMA-PC field joint was capable of providing an ABC precast deck system that emulates the behavior of conventional CIP deck systems in terms of strength requirements.
The load-deflection relationships shown in Figure 5-28 are divided into different regions of behavior. The first portion of behavior represents the elastic behavior of the specimen when the applied load is approximately below 20 kips. During this stage, only hair flexural cracks in the longitudinal direction were observed at the bottom face of the west side of the specimen. A hair transverse crack located approximately 5 in apart from the edge of the joint and ran parallel to the connection in the loaded west side of the slab was also observed. No cracks were observed in the unloaded east side of the slab. No interface de-bonding or concrete crushing was observed at the top surface during the service level loading. The second region in defining the load-deflection curve represents the onset and progression of the inelastic behavior as the applied load exceeded the elastic load. This inelastic behavior started in the third and fourth cycles of loading around 40 kips vertical load when the flexural stiffness decreased and residual displacements increased. The cracks at this level were still narrow and the main reinforcement was not yet yielded. The crack patterns of specimen “S3-T-PC-Loop” after the second and fourth cycles are shown in Figure 5-30.

![Figure 5-30: (a) Crack pattern at the bottom side of specimen “S3-T-PC-Loop” after the 2nd cycle, and (b) crack pattern after the 4th cycle.](image)

The slight inelastic response continued as the load increased to approximately 80 kips. After that, the third region of behavior is defined by the significant reduction in stiffness as the specimen
started to soften due to yielding of the middle and west side transverse reinforcement in the bottom mesh. Hence, deep and wide flexural cracks were observed on the bottom side of the slab. No bar slip was observed until reaching the peak load. Following the peak load capacity, the fourth region of behavior is defined by a global softening behavior where the load capacity started to decrease with larger displacements. A series of sudden drops was then observed during the descending branch of the load-deflection relationship. Each sudden drop was accompanied by concrete crushing at the top surface in the loaded part of the slab (west side) followed by crushing in the polymer concrete part and east side.

Similarly, the measured deflections of the west side of the specimen and the deflections along the field joint were almost identical as can be seen in Figures 5-28 and 5-29. However, the unloaded side of the specimen (east side) attained lower deflections. Figure 5-31 shows the difference between the mid-span deflections and the east and west sides deflections versus the applied vertical load. The difference in string potentiometer readings between the middle and the west side of the specimen is minimal, while the difference in deflections between the middle and east side are comparably higher. The difference between the east and west side deflections at the measured peak load is 0.72 in.

![Figure 5-31: Difference between mid-span deflections and deflections of the east and west sides of specimen “S3-T-PC-Loop”](image)

The load versus strain readings of the bottom transverse reinforcement are shown in Figures 5-32 to 5-34 and they are discussed here to provide more understanding of the flexural behavior of the specimen and the reason for failure. It is noted that some of the strain gages were damaged during the test especially at larger load levels when concrete damage led to strain gages and wire damages.
Figure 5-32: Reinforcement strain gage readings of the bottom transverse bars in the west precast panel of specimen “S3-T-PC-Loop”.
Figure 5-33: Reinforcement strain gage readings of the bottom transverse bars in the east precast panel of specimen “S3-T-PC-Loop”.
Figure 5-34: Reinforcement strain gage readings for the bottom transverse bars inside the field joint of specimen “S3-T-PC-Loop”.
It can be seen from Figures 5-32 to 5-34 that the largest strains were recorded in the middle bars near the location of the applied load. The onset of yielding was observed in the bar adjacent to the joint on the west side at 67.5 kips. This was followed by yielding in the bar adjacent to the joint on the east side at 74 kips. Most of the bars on the west side and the middle part of the slab yielded at approximately 80 kips. Again, it was observed that the strain values at the same applied load level were different between the east and west sides of the slab. In most of the cases, the west side had higher values of strains than the east side. It is worth noting that the bars inside the joint were not placed near the top and bottom edges of the joint as they were added at mid-depth of the joint as lacer bars to provide some bearing forces for the loop splices inside the joint. Hence, the strain readings of these bars were slightly lower than the adjacent bars outside the joint. Moreover and similar to the other two transverse specimens, the transverse bottom reinforcement inside the joint and slab remained elastic up to the service and ultimate loads specified in AASHTO, which further verifies the acceptable behavior of the PMMA-PC deck joints with loop splices.

The joint opening at the bottom of the slab was monitored through the test using three LVDTs. Figure 5-35 shows the horizontal displacement results from these LVDTs. It is worth noting that the readings of the middle LVDT had some noise at the beginning of the test.

**Figure 5-35:** Joint opening versus applied load for specimen “S3-T-PC-Loop”.

It can be seen that the joint opening is minimal and it perfectly satisfies the crack width limits with respect to the design loads specified in AASTO LRFD. The specimen has reached the maximum allowable crack width assuming a class (2) exposure at approximately 83 kips of vertical mid-span load that is almost 1.75 times the specified ultimate design load of the deck slab and 3.1 times the service load. These values were calculated based on the readings from the LVDT located at the
north side of the slab. This is a good conclusion that the interface bond between the PMMA-PC and the precast segments was perfectly satisfying the design requirements and code allowable crack width limits.

5.2 Test Results of the Longitudinal Specimens

The flexure static testing of the longitudinal specimens was completed according to the process described previously in Chapter 4. The testing was completed over the course of 1.25 hours. The loading process included four cycles of static axial loading and unloading. After each cycle, the test was stopped to inspect the flexural cracks at the bottom of the slabs and visually determine if there are any signs of interface deboning that happened between the precast panels and the field joints. The test data was filtered and processed to determine the structural performance of the deck slabs. The results of each specimen are separately presented to provide a detailed illustration of the structural performance of the precast deck slabs throughout the different loading stages. Finally, a comparison between both longitudinal specimens was done and reported in Chapter 6 to illustrate the difference between using UHPC and PMMA-PC as a closure material inside the longitudinal field joints of the DBT girders.

5.2.1 Test Results of Specimen “S4-L-UHPC”

The measured peak load of specimen “S4-L-UHPC” is 115.8 kips at 1.51 in mid-span vertical displacement. The observed mode of failure for the UHPC longitudinal specimen was yielding in the main reinforcement due to flexure accompanied with crushing of concrete at the top side of the loaded south precast panel. Deep and wide flexural cracks were observed at the bottom face of the precast slabs interrupted by three main localized cracks in the UHPC at the measured peak load. Figure 5-36 shows an overview of the observed extensive flexural cracking at the bottom side of the specimen at peak load. The crack pattern at the bottom of the loaded (south) precast panel is shown only in Figure 5-36. Crushing of concrete was initially triggered under the loading location at approximately 103 kips and then propagated through the east and west sides of the loaded slab part as shown in Figure 5-37 a. Signs of punching shear failure were also observed by the end of the test under the loading pad. After reaching the peak load, chunks of conventional concrete popped up at the top surface due to the excessive concrete crushing that was accompanied by the crack localization at the bottom of the UHPC joint that led up to a sudden drop in the load capacity of the specimen. Shear cracking was observed at the west side of the loaded panel, the crack propagated from the bearing pads to the UHPC joint in a diagonal formation as shown in Figure 5-37 b.

The flexural cracks at the bottom side were mainly localized in the loaded (south) precast panel under the applied load location as shown in Figure 5-38 a. Limited flexural cracking was observed under the unloaded side of the slab. The first flexural crack in the UHPC was observed when the specimen reached approximately 100 kips. A long length and narrow width interface crack was
observed at the end of the test at the bottom between the loaded south side and the UHPC joint as shown in Figure 5-38 b. No bar rupture or bar slip was observed during the test. The test was again stopped after losing approximately 20% of the peak load.

Figure 5-36: Crack pattern at the bottom side of specimen “S4-L-UHPC” at peak load.
Figure 5-37: (a) Concrete crushing at the top-south side of specimen “S4-L-UHPC” at the end of the test, and (b) diagonal shear cracking at the west side.
Figure 5-38: (a) Flexural cracking at the bottom-south side of specimen “S4-L-UHPC” at the end of the test, and (b) Interface cracking at the bottom between the loaded south side and the UHPC joint.

The load-deflection relationships developed from the readings of the three middle string potentiometers are presented in Figure 5-39. The load-deflection relationships developed from the other six string potentiometers located at quarter spans in addition to the middle string potentiometer are presented in Figure 5-40.
Figure 5-39: Load-deflection relationships at mid-span of specimen “S4-L-UHPC”.

Figure 5-40: Load-deflection relationships at quarter-spans of specimen “S4-L-UHPC”.
The flexural capacity of the specimen exceeded the two code limit states, i.e., service and ultimate loads specified in the AASHTO LRFD using the equivalent strip method. Therefore, the UHPC field joint was capable of providing continuity of load transfer between the adjacent DBT girders up to the AASHTO strength requirements. The load-deflection relationships shown in Figure 5-39 are divided into different regions of behavior. The first region of behavior represents the elastic behavior of the specimen when the applied load was approximately below 15 kips. During this linear response, no flexural cracks were observed at the bottom side of the slab as the specimen did not yet reach its cracking moment. No interface debonding or concrete crushing was observed during the service level load. No cracks were also observed in the field joint.

The second region in defining the load-deflection relationship represents the onset and progression of the inelastic behavior as the applied load exceeded the elastic load. This started through the third and fourth cycles of loading around 30 kips when the flexural stiffness decreased and the residual displacement increased. A hair transverse crack located approximately 12 inches apart from the edge of the joint and ran parallel to the joint in the loaded south side of the slab was observed. However, no flexural cracks were observed on the north precast panel. The cracks at this level were still narrow and the main reinforcement was not yet yielded. It is worth noting that there were no flexural or interface cracks observed in the field joint because of the higher tensile strength of the UHPC as compared to the conventional concrete. Figure 5-41 shows the crack pattern at the bottom of the specimen at the AASHTO ultimate load.

Figure 5-41: Crack pattern at the bottom side of specimen “S4-L-UHPC” at AASHTO ultimate load.
Figure 5-41 shows that there are only flexural cracks at the bottom of the south precast panel especially under the loading pad and there is no interface cracking or flexural cracking in the UHPC joint.

The slight inelastic response continued as the load increased to approximately 80 kips where many of the middle transverse bars at the bottom reinforcement of the south precast panel have yielded. After that, the third region of the behavior of the load-deflection relationship is defined by the significant reduction in stiffness as the specimens started to soften due to the yielding of transverse reinforcement of the bottom mesh. Hence, deep and wide flexural cracks were observed on the bottom side of the slab. No bar slip was observed until reaching the peak load. Following the peak load capacity, the fourth region of behavior is defined by sudden drops in the load capacity of the specimen with small increments of applied displacements. The sudden loss of the load capacity was mainly due to the crack localization in the UHPC joint in addition to the excessive concrete crushing at the top surface of the south precast panel. The goal of pushing the specimen to such high levels of loads and displacements beyond code-required limits was to provide more understanding of the modes of failure and check whether the different system components will stay intact till the end of the test. It is worth noting that no bar rupture was observed throughout the test.

Figure 5-39 shows that the middle deflections are more than the west and east sides deflections as the load were applied in the middle of the longitudinal direction of the specimen. Moreover, the east and west sides deflections were almost identical throughout the test. This symmetry in deflections is a good indication that the UHPC joint was capable of uniformly distribute the applied load to both east and west sides till the end of the test despite the extensive flexural cracking of the specimen. The difference in deflections between the middle and east or west sides increased with the increase of the applied load. In the beginning, the loads were too small and there were only limited flexural cracks and hence this allowed the slab to bend uniformly. However, with increasing the load, the flexural stiffness of the slab was decreased because of the extensive flexural cracking that led to an increase of the difference in deflections between the middle and the east or west sides.

The measured deflections of the south precast panel and the deflections at the middle of the field joint were almost identical as can be seen in Figure 5-40. While the unloaded side of the specimen (north side) attained lower deflections, which was expected because of the asymmetry of the specimen’s loading in the transverse direction, i.e., north-south direction.

Another interpretation for this observation is that the field joint was not transferring the full applied loads between the two sides of the slab as there was a discontinuity in the reinforcement between both sides. Moreover, this difference in deflections increased with the increase of the applied load due to the influence of the reduction in stiffness of the slabs after the propagation of the flexural excessive cracking. This is consistent with results from a previous finite element study (NCHRP,
2011) that showed that the shear and moment forces on the field joint are greatly reduced due to the stiffness reduction accompanied by cracking of the specimen. Figure 5-42 shows the difference between the mid-span deflections and the north and south side deflections versus the applied vertical load of specimen “S4-L-UHPC”. It can be seen that the difference in deflections between the middle and south sides was minimal, while the difference in deflections between the middle and north side was comparably higher. The difference between the middle and north side deflections at the measured peak load is 0.66 in.

![Figure 5-42: Difference between mid-span deflections and the deflections of the north and south sides of specimen “S4-L-UHPC”.

The load versus strain readings of the bottom transverse reinforcement are shown in Figures 5-43 and 5-44 and they are discussed here to provide more understanding of the flexural behavior of the specimen and the reason for failure. It is noted that some of the strain gages were damaged during the test especially at larger load levels when concrete damage led to strain gages and wire damages...
Figure 5-43: Reinforcement strain gage readings of the bottom transverse bars in the south precast panel of specimen “S4-L-UHPC”.

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Figure 5-44: Reinforcement strain gage readings of the bottom transverse bars in the north precast panel of specimen “S4-L-UHPC”.

It can be seen from Figures 5-43 and 5-44 that the largest strains were observed in the middle bars near the applied load location. The onset of yielding was observed in one of the bars at the middle of the south precast panel at 64 kips. This was followed by yielding in the adjacent bar at 70 kips. After this limit, the load-deflection behavior of the slab was significantly changed to a more inelastic response associated with more deflections corresponding to small force gain increments as previously shown in Figure 5-39.
Most of the bottom transverse bars of both precast panels were yielded at approximately 100 kips, after then a concrete crushing was initiated at the top surface of the south precast panel. The strains of the reinforcement in the south precast panel were slightly higher than the values of strains of the reinforcement in the north precast panel. No bar rupture was indicated throughout the test. The transverse reinforcement has yielded within the UHPC joint. This is good proof that a UHPC joint with 6 in width was sufficient to provide a proper development length for the bars inside the joint. Consequently, this provided better load transfer between both precast panels. Finally, a key observation from the behavior depicted in the previous figures is that none of the joint or slab transverse bottom reinforcement yielded before reaching the ultimate and service loads specified in the AASHTO, i.e., the deck system remained elastic up to the code limits as required.

The interface crack width between the field joint and the precast panels was monitored through the test using six LVDTs as previously addressed in Chapter 4. Figure 5-45 shows the horizontal displacement readings from these LVDTs versus the applied load. It is worth noting that the readings of “LVDT NE” were not provided because of an error that happened in the DAQ system.

![Figure 5-45: Interface crack opening versus applied load for specimen “S4-L-UHPC”](image)

The interface crack width at mid-span from the south side of the specimen was larger than any other location along the joint. The reason for this larger crack width is that the mid-span zone is subjected to the maximum applied load and maximum vertical deflection. The interface crack opening reached the maximum allowable crack assuming a class (2) exposure at approximately 35 kips of vertical load which is higher than the AASHTO service load of the slab but still below the AASHTO ultimate load. As mentioned before, the readings from the LVDTs do not provide an accurate representation of the interface crack width as the accuracy of the readings decreases with
increasing the applied load. As the LVDTs are measuring the inclined distance instead of the horizontal distance.

5.2.2 Test Results of Specimen “S5-L-UHPC”

The measured peak load of specimen “S5-L-PC” was 98.2 kips at 1.41 in mid-span vertical displacement. The observed mode of failure for the PC longitudinal specimen was yielding in the main reinforcement due to flexure accompanied with interface cracking between the loaded south panel and the PC joint. This interface crack started to appear at the bottom of the slab at approximately 88 kips of the vertical applied load. After the peak load, conventional concrete crushing was observed at the top side of the loaded south precast panel that caused a sudden drop to the load capacity of the specimen. Figure 5-46 shows an overview of the observed extensive flexural cracks at the bottom side of specimen “S5-L-PC” at the peak load.

At failure, deep and wide flexural cracks were observed at the bottom face of the precast panels with multiple transverse cracks also observed in the PC joint. Again, the flexural cracks at the bottom face were mainly localized in the loaded (south) precast panel under the patch loading as shown in Figure 5-47. However, limited flexural cracking was only observed under the unloaded side of the slab. Crushing of concrete initially happened around the loading pad after the observation of a punching shear under the loading pad at approximately 94 kips and then the cracks propagated through the east and west sides of the loaded (south) precast panel as shown in Figure 5-48 a. Crushing of concrete was initially observed after reaching the peak load while approaching the descending branch of the load-deflection relationship.

The first transverse crack in the PC field joint was observed at approximately 75 kips. The expected reason for failure was the interface cracking at mid-span between the loaded south side and the PC joint as shown in Figure 5-47. Another interface crack was also observed at the east side of the specimen, this crack was developed between the north side of the slab and the PC joint as shown in Figure 5-48 b. No bar rupture or bar slip was observed during the test. The test was again stopped after reaching approximately 80% of the peak load.
Figure 5-46: Crack pattern at the bottom side of specimen “S5-L-PC” at peak load.
Figure 5-47: Flexure and interface cracks at the bottom side of specimen “S5-L-PC” at the end of the test.
Figure 5-48: (a) Concrete crushing at the top-south side of specimen “S5-L-PC” at the end of the test, and (b) interface cracking at the east side.

The load-deflection relationships based on the readings from the three middle string potentiometers are presented in Figure 5-49. Similarly, the load-deflection relationships based on
the readings from the other six string potentiometers located at quarter spans in addition to the middle string potentiometer are presented in Figure 5-50.

**Figure 5-49:** Load-deflection relationship at mid-span of specimen “S5-L-PC”.

![Load-deflection relationship at mid-span of specimen “S5-L-PC”](image-url)
It can be seen from the previous figures that the flexural capacity of the specimen exceeded the two code limit states, i.e., service and ultimate loads specified in the AASHTO LRFD using the equivalent strip method. Therefore, the PMMA-PC field joint was capable of providing continuity of load transfer between the adjacent DBT girders up to the AASHTO strength requirements. The load-deflection relationships shown in Figure 5-49 can be divided into different regions of behavior. The first region of behavior represents the elastic behavior of the specimen when the applied load was approximately below 15 kips. During this stage, only a hair longitudinal crack located approximately 5 in apart from the edge of the joint and ran parallel to the joint in the loaded south side of the slab was observed. No interface de-bonding or concrete crushing was observed during the service level loading. No cracks were also observed in the field joint because of the higher tensile strength of the PC as compared to the conventional concrete.

The second region in defining the load-deflection relationship represents the onset and progression of the inelastic behavior as the applied load exceeded the elastic load. This started in the third and fourth cycles of loading around 30 kips of vertical load when the flexural stiffness decreased and the residual displacement increased. Hair cracks under the loading pad at the south precast panel were observed. However, no flexural cracks were observed on the north precast panel. The cracks at this level were still narrow and the main reinforcement was not yet yielded. It is worth noting that there were no flexural or interface cracks observed in the field joint until reaching the AASHTO specified ultimate load as shown in Figure 5-51.
The slight inelastic response continued as the load increased to approximately 75 kips where some of the bottom transverse bars of the south precast panel were yielded followed by flexural cracking in the PC joint. After that, the third region of the behavior of the load-deflection relationship is defined by the significant reduction in stiffness as the specimen started to soften due to the yielding of the transverse reinforcement of the bottom mesh. Hence, deep and wide flexural cracks were observed on the bottom side of the slab. No bar slip was observed until reaching the peak load. Following the peak load capacity, the fourth region of behavior is defined by a gradual decrease in the load capacity of the specimen. The loss in the load capacity was mainly due to the extensive cracking that happened under the loading patch and then these cracks extended to the PC joint at approximately 75 kips that lead to redistribution of the applied loads to the west and east sides of the slab until finally interface de-bonding was observed. This load redistribution allowed more ductile failure of the PC specimen. It is noted that the goal of pushing the specimens to such high levels of loads and displacements beyond code-required limits was to have a better understanding of the modes of failure and check whether the different system components will stay intact till the end of the test. It is worth noting that no bar rupture was indicated throughout the test.
Figure 5-49 shows that the middle deflections were larger than the west and east sides deflections as the load were applied in the middle of the longitudinal direction of the specimen. Moreover, the east and west sides deflections are almost identical throughout the test. This symmetry in deflections is a good indication that the PC joint was capable of uniformly distribute the applied load to both east and west sides till the end of the test despite the extensive flexural cracking of the specimen. It was also observed that the difference in deflections between the middle and the east or west sides increased with the increase of the applied load.
It was also observed that the measured deflections of the south side were slightly lower than the deflections at the middle of the field joint as can be seen in Figure 5-50. However, the deflections of the south precast panel exceeded the deflections in the middle of the specimen after reaching the peak load due to the interface crack that happened between the PC joint and the south side of the specimen. It was also observed that the unloaded side of the specimen (north side) attained lower deflections which were expected because of the asymmetry of the specimen loading in the transverse direction, i.e., north-south direction. Figure 5-52 shows the difference between the mid-span deflections and the north and south sides deflections versus the applied vertical load of the slab.

![Figure 5-52: Difference between mid-span deflections and the deflections of the north and south sides of specimen “S5-L-PC”.

The difference in string potentiometer readings between the middle and the south side of the specimen is minimal till reaching the peak load but this difference was increased because of the interface cracking as mentioned before. On the other side, the difference in deflections between the middle and north side was comparably higher. The difference between the middle and north side deflections at the measured peak load is 0.62 in.

The load versus strain readings of the bottom transverse reinforcement are shown in Figures 5-53 and 5-54 and they are discussed here to provide more understanding of the flexural behavior of the specimen and the reason for failure. It is noted that some of the strain gages were damaged during the test especially at larger load levels when concrete damage led to strain gages and wire damages.
Figure 5-53: Reinforcement strain gage readings of the bottom transverse bars in the south precast panel of specimen “S5-L-PC”.

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Figure 5-54: Reinforcement strain gage readings of the bottom transverse bars in the north precast panel of specimen “S5-L-PC”.
It can be clearly seen from Figures 5-53 and 5-54 that the largest strains were recorded in the middle bars close to where the load was applied until reaching approximately 75 kips. After that, the PC joint started to crack in the middle and the load was redistributed to the east and west sides. Consequently, the strains at the west and east side bars were much higher than the strains in the middle at the end of the test. The onset of yielding was observed in one of the bars at the east of the south precast panel at 92 kips. However, it is expected that the onset of yielding has happened at approximately 75 kips, but it was not captured as it was located on one of the non-instrumented bars. The bar that was expected to yield at 75 kips was one of the bars in the middle of the south precast panel and adjacent to the middle instrumented bars. Most of the bottom transverse bars at both precast panels were yielded at approximately 98 kips, the load-carrying capacity of the specimen was then started to decrease because of the interface cracking and the loss of stiffness that happened to the PC joint that was the main reason for failure. The strains of the reinforcement in the south precast panel were slightly higher than the values of strains on the north side. No bar rupture was indicated throughout the test.

It is worth noting that only the middle transverse bars have strain gages inside the PC joint. Hence, none of the strain gages that were installed inside the PC joint has reached the yield strain because of the load redistribution that happened after the PC cracking as mentioned before. Consequently, it was hard to determine the adequacy of the joint width in terms of the required development length. However, it can be concluded that the 6 in width joint can provide adequate development length that can transfer the applied load across different precast panels up to 75 kips which is more than 1.5 times the required AASHTO ultimate load. Finally, a key observation from the behavior depicted in the previous figures is that none of the joint or slab transverse bottom reinforcement yielded before reaching the ultimate and service loads specified in the AASHTO, i.e., the deck system remained elastic up to the code limits as required.

The interface crack width between the field joint and the precast panels was monitored through the test using six LVDTs as mentioned before in Chapter 4. Figure 5-55 shows the horizontal displacement readings from these LVDTs versus the applied load. The crack width at mid-span from the south side of the specimen was larger than any other location along the joint. The reason behind that is that the mid-span zone is subjected to the maximum applied load and maximum vertical deflection. The interface crack opening reached the maximum allowable crack assuming a class (2) exposure at approximately 31 kips of vertical load which is higher than the AASHTO service load of the slab but still below the AASHTO ultimate load.
Figure 5-55: Interface crack opening versus applied load for specimen “S5-L-PC”.
6. Deck Panel Test: Comparison

This chapter provides a comparative assessment of the structural performance of the deck slabs with UHPC and PMMA-PC closure joints used for the field joints. The comparison was mainly discussed in terms of the load versus deflection responses of the three transverse specimens and the two longitudinal specimens as evaluated at various loading levels in addition to evaluating the reasons for failure and the reinforcement strains. This chapter is divided into two main sections; the first section includes the comparison between the three transverse specimens and the other section includes the comparison between the other two longitudinal specimens.

6.1 Comparison of the Transverse Specimens

Specimen “S1-T-UHPC” is considered the control specimen for the other two transverse PMMA-PC specimens. The load-deflection results of both PMMA-PC specimens were presented in a comparative format with respect to specimen “S1-T-UHPC” results. Hence, the load-deflection relationship is divided into two sections that include a comparison between specimens “S1-T-UHPC” and “S2-T-PC”, and another comparison between specimens “S1-T-UHPC” and “S3-T-PC-Loop”.

6.1.1 Summary of Results

The summary of the experimental results for the three transverse specimens is reported in Table 6.1.

<table>
<thead>
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<th>Specimen Name</th>
<th>Peak Load (kips)</th>
<th>Middle Deflection at Peak Load (in)</th>
<th>Middle Deflection at Service Load (in)</th>
<th>Middle Deflection at Ultimate Load (in)</th>
<th>Initial Stiffness (kips/in)</th>
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</thead>
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<td>0.175</td>
<td>0.384</td>
<td>240</td>
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<tr>
<td>S2-T-PC</td>
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<td>2.530</td>
<td>0.205</td>
<td>0.460</td>
<td>190</td>
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<td>2.630</td>
<td>0.177</td>
<td>0.397</td>
<td>220</td>
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</tbody>
</table>

6.1.2 Load-Deflection Relationship

6.1.2.1 Specimen “S1-T-UHPC” versus “S2-T-PC”

The overall load versus the mid-span vertical displacement obtained from three middle string potentiometers of specimens “S1-T-UHPC” and “S2-T-PC” are presented in Figure 6-1. It is noted that the data acquisition readings in the third cycle of specimen “S2-T-PC” had some noise that partially affected the accuracy of the load-deflection relationship results.
Two main observations can be drawn from Figure 6-1: (1) the overall behavior of the two specimens is very comparable, and (2) the flexural capacity of both specimens exceeded the two code limit states, i.e., service and ultimate loads specified in the AASHTO LRFD using the equivalent strip method. Therefore, both UHPC and PC field joints were capable of providing an ABC precast deck system that is capable of emulating the behavior of conventional CIP deck systems in terms of strength requirements. The measured peak load of specimen “S1-T-UHPC” was 117.9 kips at 2.33 in mid-span vertical displacement. While the measured peak load of specimen “S2-T-PC” was 113.2 kips at 2.53 in mid-span vertical displacement. The UHPC specimen exhibited larger flexural stiffness than the PMMA-PC specimen because of the higher stiffness, tensile and compressive strengths of the UHPC. The early measured flexural stiffness of the UHPC specimen was found to be 240 kips/in versus 190 kips/in for the PMMA-PC specimen, i.e. the PMMA-PC specimen stiffness was about 80% of that of the UHPC specimen.

6.1.2.2 Specimen “S1-T-UHPC” versus “S3-T-PC-Loop”

The overall load versus the mid-span vertical displacement obtained from three middle string potentiometers for specimens “S1-T-UHPC” and “S3-T-PC-Loop” are presented in Figure 6-2.
Three main observations can be drawn from Figure 6-2: (1) the overall behavior of both specimens are almost identical, (2) the flexural capacity of both specimens exceeded the two code limit states, i.e., service and ultimate loads specified in the AASHTO LRFD using the equivalent strip method, and (3) the load capacity of specimen “S3-T-PC-Loop” is slightly higher than the other transverse specimens. Therefore, both UHPC and PC-Loop field joints were capable of providing an ABC precast deck system that can emulate the behavior of conventional CIP deck systems in terms of strength requirements. Moreover, the PMMA-PC joints with loop splice configuration enhanced the structural performance and increased the flexural stiffness of the deck slabs. The measured peak load of specimen “S3-T-PC-Loop” was 122.5 kips at 2.63 in mid-span vertical displacement. The PMMA-PC specimen with loop splice exhibited larger flexural stiffness than the PMMA-PC specimen with straight splice, however, the flexural stiffness of specimen “S3-T-PC-Loop” is slightly less than that of “S1-T-UHPC”. The early measured flexural stiffness of specimen “S3-T-PC-Loop” was 220 kips/in.

6.1.3 Difference in Deflections

As discussed earlier in Chapter 5, the difference between the middle deflections and the west or east deflections could be a useful tool to determine the efficiency of the field joints in transferring the applied load across the precast panels. For all the transverse specimens, the loaded west side of the slab and the field joint deflection were almost identical. However, the unloaded east side attained lower deflections. This difference in deflections is mainly because of two reasons; asymmetry of the specimen loading in the longitudinal direction, and capability of the field joint to transfer the full applied loads between the two sides of the slab. Figure 6-3 shows the difference.
between the mid-span vertical displacement and the east and west side displacements versus the applied vertical load.

![Graph showing load vs. difference in vertical displacement for specimens S1-UHPC (MID - W), S1-UHPC (MID - E), S2-PC (MID - W), S2-PC (MID - E), S3-PC-Loop (MID - W), and S3-PC-Loop (MID - E). The graph indicates the difference in deflections between the middle and west side of the specimens are minimal, while the difference in deflections between the middle and east side are comparably higher. The difference between the deflections of the middle and east unloaded side of the slab can provide an indication about the performance of the field joint and the ability of the joint to transfer the applied load between both sides of the precast panels. As the difference in deflections decreases, the load distribution over the slab is better and the joint performance in transferring the loads is even better. Hence, the PMMA-PC specimen with loop splice shows better load distribution than the other two specimens.

It can be concluded from the previous Figures 6-1 to 6-3 that specimen S3-T-PC-Loop attained higher ultimate deflections and higher load capacity than the other two specimens. Therefore, the loop splice configuration and the denser reinforcement inside the field joint increase the flexural ductility of the deck slabs and enhances the load distribution across the precast panels.

6.1.4 Early Flexural Behavior

It is important to compare the early performances of the three proposed transverse field joints based on the early flexural behavior of the specimens. This will reflect on the behavior of the actual bridge decks under the specified AASHTO loads. The main observations that can be concluded
from the early assessment of the transverse specimens under the AASHTO ultimate load are as follows:

- No flexural cracks were observed on the field joints before reaching the AASHTO ultimate load
- No interface cracks were observed between the field joints and the precast panels
- The 6 in width joints were capable of transferring the loads uniformly across the precast panels without any signs of bar bond-slip
- All specimens failed outside the joints in flexure
- None of the bottom transverse reinforcement reached the yield strain before the AASHTO ultimate load, hence all the specimens remained elastic up to this limit.
- The flexural cracks at the bottom of the precast panels were limited and narrow

The crack pattern at the bottom face of the three transverse specimens at the AASHTO ultimate load level is shown in Figure 6-4.

![Figure 6-4](image)

**Figure 6-4:** Crack pattern at the AASHTO ultimate load level at: (a) bottom side of “S1-T-UHPC”; (b) bottom side of “S2-T-PC”; and (c) bottom side of “S3-T-PC-Loop”.

It was observed that the early crack propagation of the PC-Loop specimen was better than the other specimens due to the denser distribution of the bottom and top mesh reinforcement (i.e. #4@7" instead of #5@10" and #4@15") that resulted in better crack control. The crack patterns in Figure 6-4 were highlighted to better identify them as the cracks at the AASHTO ultimate load level were narrow and small that makes them hard to be seen. Again, no flexural or interface cracks were observed in the field joints for the transverse specimens till reaching the AASHTO ultimate load level as shown in the previous figure.
6.1.5 Transverse Reinforcement Strains

This section presents selected results from the measured strain data. The load versus strain readings of selected bottom transverse reinforcement at mid-span location are shown in Figure 6-5 and discussed here.

None of the joint or slab transverse bottom reinforcement has yielded before reaching the ultimate and service loads specified in the AASHTO, i.e., the deck system remained elastic up to the code limits as required. The onset of yielding was observed on the bar inside the joint at 65 kips for the specimen “S1-T-UHPC”. Similarly, the onset of yielding was observed on the bar inside the joint at 46 kips for specimen “S2-T-PC”. However, the onset of yielding was observed in the bar adjacent to the connection on the west side at 67.5 kips for specimen “S3-T-PC-Loop”.

Figure 6-5: Mid-span strain gage readings of the bottom transverse bars for the transverse specimens.
6.1.6  Peak Strain Values

The strain gage readings of the top and bottom transverse reinforcement at peak loads for the three transverse specimens are reported in Table 6.2. These strain values were reported herein to be as a reference or a verification tool for any further finite element or numerical studies that could be conducted in the future.

Table 6-2: Summary of the measured reinforcement strain values at peak load.

<table>
<thead>
<tr>
<th>Reinforcement</th>
<th>Strain Gage Location</th>
<th>SG #</th>
<th>S1-UHPC Strain (%)</th>
<th>S2-PC Strain (%)</th>
<th>S3-PC-Loop Strain (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Bottom Transverse</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>1</td>
<td>ST 1</td>
<td>0.21</td>
<td>0.18</td>
<td>0.23</td>
</tr>
<tr>
<td></td>
<td>2</td>
<td>ST 2</td>
<td>1.27</td>
<td>1.37</td>
<td>1.87</td>
</tr>
<tr>
<td></td>
<td>3</td>
<td>ST 3</td>
<td>0.18</td>
<td>0.22</td>
<td>0.17</td>
</tr>
<tr>
<td></td>
<td>4</td>
<td>ST 4</td>
<td>0.17</td>
<td>NA</td>
<td>0.26</td>
</tr>
<tr>
<td></td>
<td>5</td>
<td>ST 5</td>
<td>1.01</td>
<td>1.22</td>
<td>1.64</td>
</tr>
<tr>
<td></td>
<td>6</td>
<td>ST 6</td>
<td>2.19</td>
<td>NA</td>
<td>2.06</td>
</tr>
<tr>
<td></td>
<td>7</td>
<td>ST 7</td>
<td>1.13</td>
<td>1.42</td>
<td>NA</td>
</tr>
<tr>
<td></td>
<td>8</td>
<td>ST 8</td>
<td>0.15</td>
<td>0.25</td>
<td>NA</td>
</tr>
<tr>
<td></td>
<td>9</td>
<td>ST 9</td>
<td>0.12</td>
<td>0.16</td>
<td>0.06</td>
</tr>
<tr>
<td></td>
<td>10</td>
<td>ST 10</td>
<td>0.61</td>
<td>2.57</td>
<td>3.13</td>
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<tr>
<td></td>
<td>11</td>
<td>ST 11</td>
<td>2.03</td>
<td>NA</td>
<td>0.05</td>
</tr>
<tr>
<td></td>
<td>12</td>
<td>ST 12</td>
<td>0.84</td>
<td>2.54</td>
<td>0.04</td>
</tr>
<tr>
<td></td>
<td>13</td>
<td>ST 13</td>
<td>0.17</td>
<td>0.09</td>
<td>NA</td>
</tr>
<tr>
<td></td>
<td>14'</td>
<td></td>
<td></td>
<td></td>
<td>0.11</td>
</tr>
<tr>
<td></td>
<td>14</td>
<td>ST 14</td>
<td>0.29</td>
<td>0.25</td>
<td>0.15</td>
</tr>
<tr>
<td></td>
<td>15</td>
<td>ST 15</td>
<td>1.13</td>
<td>1.13</td>
<td>1.26</td>
</tr>
<tr>
<td></td>
<td>16</td>
<td>ST 16</td>
<td>1.92</td>
<td>1.57</td>
<td>2.24</td>
</tr>
<tr>
<td></td>
<td>17</td>
<td>ST 17</td>
<td>0.72</td>
<td>1.10</td>
<td>0.91</td>
</tr>
<tr>
<td></td>
<td>18</td>
<td>ST 18</td>
<td>0.23</td>
<td>0.22</td>
<td>0.24</td>
</tr>
<tr>
<td></td>
<td>19</td>
<td>ST 19</td>
<td>0.80</td>
<td>0.49</td>
<td>0.24</td>
</tr>
<tr>
<td></td>
<td>20</td>
<td>ST 20</td>
<td>0.76</td>
<td>1.00</td>
<td>0.54</td>
</tr>
<tr>
<td></td>
<td>21</td>
<td>ST 21</td>
<td>0.26</td>
<td>0.58</td>
<td>0.26</td>
</tr>
<tr>
<td>Top Transverse</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>22</td>
<td>ST 22</td>
<td>0.04</td>
<td>0.07</td>
<td>0.01</td>
</tr>
<tr>
<td></td>
<td>23</td>
<td>ST 23</td>
<td>0.66</td>
<td>0.46</td>
<td>0.26</td>
</tr>
<tr>
<td></td>
<td>24</td>
<td>ST 24</td>
<td>0.01</td>
<td>0.02</td>
<td>0.01</td>
</tr>
<tr>
<td></td>
<td>25</td>
<td>ST 25</td>
<td>0.01</td>
<td>-0.01</td>
<td></td>
</tr>
<tr>
<td></td>
<td>26</td>
<td>ST 26</td>
<td>0.20</td>
<td>0.19</td>
<td></td>
</tr>
<tr>
<td></td>
<td>27</td>
<td>ST 27</td>
<td>0.004</td>
<td>-0.00</td>
<td></td>
</tr>
<tr>
<td></td>
<td>28</td>
<td>ST 28</td>
<td>0.02</td>
<td>0.13</td>
<td>0.02</td>
</tr>
<tr>
<td></td>
<td>29</td>
<td>ST 29</td>
<td>0.18</td>
<td>NA</td>
<td>0.19</td>
</tr>
<tr>
<td></td>
<td>30</td>
<td>ST 30</td>
<td>0.03</td>
<td>0.09</td>
<td>-0.00</td>
</tr>
</tbody>
</table>
It can be seen that the strain readings for the three specimens are very comparable. However, it was also observed that the strain readings of the bars inside the PMMA-PC are higher than that of the UHPC and the bars just outside the joint which is in the NSC.

6.1.7 *Longitudinal Reinforcement Strains*

The load versus strain readings of selected top longitudinal reinforcement at mid-span location are shown in Figure 6-6.

![Figure 6-6: Strain gage readings of the top longitudinal bars at the middle of the specimens.](image)

It can be seen that most of the strain values were typically below the measured yield strain of the reinforcement. However, the strain values of specimen “S1-T-UHPC” exceeded the yield limit and were extensively higher than the reported values in the other two specimens. The reason for this increase was probably due to the interface de-bonding that happened between the UHPC joint and the east side of the slab after reaching the peak load.

It can also be seen that the recorded strain values of the strain gages that were installed inside and outside the field joint were almost identical. This could be a good clue that there is no bar slip that happened till reaching the peak loads. The strain values of specimen “S3-T-PC-Loop” are slightly less than that of specimen “S2-T-PC” because of the denser reinforcement used in specimen “S3-T-PC-Loop” (i.e., #4@7" instead of #5@10" and #4@15") and the continuity of the force transfer between the top and bottom reinforcement because of using the loop splice. Hence, better load distribution is achieved by specimen “S3-T-PC-Loop”.

The measured strain values of the longitudinal bars inside and outside the joint at the ultimate and service loads were typically far below the yield strains. This could be a good conclusion that the demand on the field joints within the AASHTO design loads does not require implementing the full development length of the rebars inside the joints. Hence, the lap splice length of the reinforcement that transversely connects two precast decks slabs is not dominated by the required development length of the bars inside the closure joint material.
### 6.1.8 Interface Opening

The interface opening width at the bottom face of the slab between the field joint and the precast parts was monitored through the test using six LVDTs. The comparison between the interface opening width at service and ultimate loads for specimens “S1-T-UHPC” and “S2-T-PC” are reported in Table 6.3.

**Table 6-3:** Interface opening width of specimens “S1-T-UHPC” and “S2-T-PC” at service and ultimate loads.

<table>
<thead>
<tr>
<th>LVDT Name</th>
<th>S1-T-UHPC Service Load, x 10⁻³ in</th>
<th>S1-T-UHPC Ultimate Load, x 10⁻³ in</th>
<th>S2-T-PC Service Load, x 10⁻³ in</th>
<th>S2-T-PC Ultimate Load, x 10⁻³ in</th>
</tr>
</thead>
<tbody>
<tr>
<td>LVDT NW</td>
<td>2.8</td>
<td>6.2</td>
<td>1.0</td>
<td>12.5</td>
</tr>
<tr>
<td>LVDT NE</td>
<td>2.8</td>
<td>3.9</td>
<td>2.6</td>
<td>7.4</td>
</tr>
<tr>
<td>LVDT W</td>
<td>5.3</td>
<td>9.3</td>
<td>3.5</td>
<td>28.0</td>
</tr>
<tr>
<td>LVDT E</td>
<td>3.6</td>
<td>5.6</td>
<td>2.5</td>
<td>4.2</td>
</tr>
<tr>
<td>LVDT SW</td>
<td>6.3</td>
<td>10.4</td>
<td>5.3</td>
<td>1.2</td>
</tr>
<tr>
<td>LVDT SE</td>
<td>4.0</td>
<td>8.0</td>
<td>1.1</td>
<td>6.5</td>
</tr>
</tbody>
</table>

#### 6.1.8.1 Specimen “S1-UHPC”

The specimen reached the maximum allowable crack assuming a class (2) exposure at approximately 60 kips of vertical load which is almost 1.25 times the specified ultimate load and 2.25 times the service load. This is a good conclusion that the interface bond between the UHPC and the precast segments was perfectly satisfying the design requirements and the code allowable limits.

#### 6.1.8.2 Specimen “S2-PC”

Due to the inaccuracy of readings of “LVDT W” because it was significantly affected by the flexural cracks at the bottom of the specimen as it was mounted over one of the wide flexural cracks that ran parallel to the joint in the transverse direction. The interface bonding was investigated through “LVDT NW”. It was observed that the interface crack opening reached the maximum allowable crack assuming a class (2) exposure at approximately 48 kips of vertical load which is slightly higher than the ultimate load of the slab. This is a good conclusion that the interface bond between the PMMA-PC and the precast segments was perfectly satisfying the design requirements and code allowable limits.
6.2 Comparison of the Longitudinal Specimens

Specimen “S4-L-UHPC” is considered the control specimen for the other longitudinal PMMA-PC specimen. Hence, this section presents the comparison between specimen “S4-L-UHPC” and specimen “S5-L-PC”.

6.2.1 Summary of Results

The summary of the experimental results for the two longitudinal specimens is reported in Table 6.4.

Table 6-4: Summary of the experimental test results of the longitudinal specimens.

<table>
<thead>
<tr>
<th>Specimen Name</th>
<th>Peak Load (kips)</th>
<th>Middle Deflection at Peak Load (in)</th>
<th>Middle Deflection at Service Load (in)</th>
<th>Middle Deflection at Ultimate Load (in)</th>
<th>Initial Stiffness (kips/in)</th>
</tr>
</thead>
<tbody>
<tr>
<td>S4-L-UHPC</td>
<td>115.8</td>
<td>1.510</td>
<td>0.193</td>
<td>0.387</td>
<td>240</td>
</tr>
<tr>
<td>S5-L-PC</td>
<td>98.2</td>
<td>1.410</td>
<td>0.231</td>
<td>0.424</td>
<td>210</td>
</tr>
</tbody>
</table>

6.2.2 Load-Deflection Relationship

This section illustrates the load and deflection behavior of the two tested specimens as evaluated at different loading levels. The load and deflection relationship of both specimens are presented in a comparative format to show the difference in behavior between both specimens and to establish a comparison in terms of flexure stiffness, peak load, peak vertical displacement, and uniformity of the load distribution. The applied load versus the measured mid-span vertical displacements at three different locations along the longitudinal joint for both specimens are presented in Figure 6-7.
Three main observations can be drawn from Figure 6-7: (1) the overall flexure response of the two specimens is very comparable, however, the flexure capacity of the UHPC specimen is comparably higher than that of the PC specimen, (2) the initial stiffness of the UHPC specimen is slightly higher than that of the PC specimen due to the higher mechanical properties of the UHPC compared to the PC, and (3) the flexure capacity of both specimens surpassed the two specified AASHTO code limits, i.e. service and ultimate loads. Consequently, both UHPC and PC field joints were capable of providing a precast bridge deck system that can satisfy the targeted behavior of the conventional CIP deck systems in terms of strength and load distribution requirements. The flexural behavior of the two specimens was very comparable till reaching approximately 90 kips where the PC specimen started to soften and lose stiffness due to the interface crack that happened between the PC joint and the south loaded side of the slab as mentioned before.

As previously mentioned, the west and east side deflections that were measured along the joints of both specimens are almost identical due to the symmetry of loading in the longitudinal direction and the capability of both joints in distributing the applied load in the longitudinal direction of the bridge. The difference between the middle deflections and the east or west side deflections of the PC joint is higher than that of the UHPC joint due to the difference in stiffness and rigidity between both joints. The UHPC joint has higher stiffness and acts as a rigid beam in the longitudinal direction and consequently less difference in deflections between the middle and east or west sides and better load distribution in the longitudinal direction.

One other observation from Figure 6-7 is that the PC specimen has more ductile failure than the UHPC specimen due to the difference between the reasons for failure for both specimens. The
UHPC specimen has failed mainly because of the flexure cracks at the bottom due to yielding of the main reinforcement then crushing at the top surface and crack localization at the UHPC joint. However, the PC specimen has failed due to the extensive cracking that happened under the loading patch and then these cracks extended to the PC joint at approximately 75 kips that lead to redistribution of the applied loads to the west and east sides of the slab till finally interface debonding was observed at the middle of the specimen. This load redistribution allowed more ductile failure for the PC specimen.

The load-displacement curves from the string potentiometers located at quarter span in addition to the middle string potentiometer for both specimens are presented in Figure 6-8.

**Figure 6-8:** Load-deflection relationship at quarter-span for specimens “S4-L-UHPC” and “S5-L-PC”.

### 6.2.3 Difference in Deflections

As discussed earlier in Chapter 5, the difference between the middle deflection and the north or south deflections could be a useful tool to determine the efficiency of the field joints in transferring the applied loads across the precast panels. For both longitudinal specimens, it was observed that the measured deflection of the south side of the specimen is slightly lower than the deflection at the middle of the field joint as can be seen in Figure 6-8. However, it was also observed that the unloaded side of the specimen (north side) attained lower deflections which were expected because of the asymmetry of the specimen loading in the transverse direction, i.e. north-south direction and the capability of the field joint to transfer the full applied loads between the two sides of the slab. Figure 6-9 shows the difference between the mid-span vertical displacement and the north and
south side displacements among the applied vertical load of the slab for both longitudinal specimens.

![Figure 6-9: Difference between mid-span deflections and the deflections of the north and south sides of the longitudinal specimens.](image)

It can be seen that both specimens are very comparable in behavior as the difference between the middle deflections and the south deflections are minimal till reaching the peak loads for both specimens. While the difference between the middle deflections and the north deflections are comparably higher and are increasing with the applied loads. The difference between the middle and north deflections are almost identical for both specimens till reaching approximately 75 kips. After this load limit, the PC joint started to crack and lose stiffness that resulted in the redistribution of the loads towards the east and west sides of the specimen. This change in behavior resulted in a significant increase in the difference in deflections between the middle and the north side of specimen “S5-L-PC”. This difference was also subjected to more increase because of the interface de-bonding that happened at 88.25 kips at the middle of the specimen between the south precast panel and the PC field joint.

### 6.2.4 Early Flexural Behavior

It is important to compare the early performances of the two proposed longitudinal field joints based on the early flexural behavior of the specimens. This will reflect on the behavior of the actual bridge decks under the specified AASHTO loads. The main observations that can be concluded from the early assessment of the longitudinal specimens under the AASHTO ultimate load are as follows:
- No flexural cracks were observed on the field joints before reaching the AASHTO ultimate load
- No interface cracks were observed between the field joints and the precast panels
- The 6 in width joints were capable of transferring the loads uniformly across the precast panels without any signs of a bar bond-slip
- The UHPC specimen failed in flexure rather than failure because of the joints
- The PC specimen failed because of the flexural cracking of the joint at 75 kips in addition to the interface crack that happened at 88.25 kips
- None of the bottom transverse reinforcement reached the yield strain before the AASHTO ultimate load, hence all the specimens remained elastic up to this limit.
- The flexural cracks at the bottom of the precast panels were limited and narrow

The crack pattern at the bottom face of the two transverse specimens at the AASHTO ultimate load level is shown in Figure 6-10.

![Figure 6-10: Crack pattern at the AASHTO ultimate load level at (a) bottom side of “S4-L-UHPC”; and (b) bottom side of “S5-L-PC”.](image)

Both specimens had almost the same crack pattern as they were mainly cracked under the loading pad however, the PC specimen has more cracks closer to the field joint. Consequently, the early crack propagation of the UHPC specimen was better than the PC specimen due to the rigidity of the UHPC specimen. The crack patterns in Figure 6-10 were highlighted to clearly identify them as the cracks at the AASHTO ultimate load level were still narrow and small that makes them hard to be seen. Again, no flexural or interface cracks were observed in the field joints for the longitudinal specimens till reaching the AASHTO ultimate load level as can be seen from the previous figure.

6.2.5 Transverse Reinforcement Strains

This section presents selected results from the measured strain data. The load versus strain readings of selected strain gages mounted at the bottom transverse reinforcement at the mid-span location
are shown in Figure 6-11 and discussed here. The selected strain gages were located inside the precast panels and just outside the field joints.

None of the transverse bottom reinforcement yielded before reaching the ultimate and service loads specified in the AASHTO, i.e., the deck system remained elastic up to the code limits as required. The onset of yielding was observed in one of the bars at the middle of the south precast panel at 64 kips for specimen “S4-L-UHPC”. Similarly, the onset of yielding for specimen “S5-L-PC” was not captured through the installed strain gages however, it was predicted to happen approximately at 75 kips in one of the bars at the middle of the south precast panel and adjacent to the middle instrumented bars.

6.2.6 Peak Strain Values

The strain gage readings of the top and bottom transverse reinforcement at peak loads for the two longitudinal specimens are reported in Table 6.5. These strain values were reported herein to be as a reference or a verification tool for any further finite element or numerical studies that could be conducted in the future.
Table 6-5: Summary of the measured reinforcement strain values at peak load.

<table>
<thead>
<tr>
<th>Reinforcement</th>
<th>Strain Gage Location</th>
<th>SG #</th>
<th>S1-UHPC Strain (%)</th>
<th>S2-PC Strain (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Bottom Transverse Reinforcement</td>
<td>W</td>
<td>ST 1</td>
<td>0.25</td>
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6.2.7 Longitudinal Reinforcement Strains

The load versus strain readings of selected strain gages located at the middle of the longitudinal reinforcement at the mid-span location are shown in Figure 6-12.

![Figure 6-12](image_url)

**Figure 6-12:** Strain gage readings of the longitudinal bars at the middle of the longitudinal specimens.

It can be seen that none of the joint or slab longitudinal reinforcement yielded before reaching the ultimate and service loads specified in the AASHTO, i.e., the deck system remained elastic up to
the code limits as required. The strains of the longitudinal bars that are located inside the field joints and the south precast panel were comparably higher than the strains of the bar on the north precast panel due to the asymmetry of loading in the transverse direction.

6.2.8  Interface Opening

The interface opening width at the bottom face of the slab between the field joint and the precast parts was monitored through the test using six LVDTs. The comparison between the interface opening width at service and ultimate loads for specimens “S4-L-UHPC” and “S5-L-PC” are reported in Table 6.6. It can be seen that both specimens have very comparable values for the interface crack opening at the early behavior of the specimen till the AASHTO ultimate load.

**Table 6-6: Interface opening width of specimens “S4-L-UHPC” and “S5-L-PC” at service and ultimate loads.**

<table>
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<tr>
<th>LVDT Name</th>
<th>S4-L-UHPC Service Load, x 10^-3 in</th>
<th>S4-L-UHPC Ultimate Load, x 10^-3 in</th>
<th>S5-L-PC Service Load, x 10^-3 in</th>
<th>S5-L-PC Ultimate Load, x 10^-3 in</th>
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7. Concluding Remarks and Future Work

7.1 Summary and Conclusions

This study presented results from extensive experimental testing that aimed at providing alternative closure joint material for the full-depth transverse and longitudinal field joints in precast bridge decks. This report presented the static structural performance of the transversely and longitudinally connected precast bridge deck panels using UHPC and PMMA-PC for closure joints. A total number of five specimens have been tested in this study. Three simply-supported full-scale transverse specimens were tested under vertical static loading at mid-span. The first two specimens considered transverse connection with diamond shear key shape and a non-contact straight lap splice of 5 in; a detail that has been demonstrated to properly transfer the flexure and shear stresses imparted through field joints. The third specimen considered transverse connection with a modified diamond shear key shape to eliminate the need for formwork erection. Moreover, the third specimen used a loop splice of 4.5 in and PMMA-PC as a closure joint material. Two other simply-supported full-scale longitudinal specimens were tested under vertical static loading at mid-span. Both specimens considered longitudinal connection with diamond shear key shape and a non-contact straight lap splice of 5 in. This report summarized the test results in the form of a detailed structural behavior for each specimen with a structural performance assessment under the specified AASHTO loads in addition to establishing comparisons between different specimens to investigate different test parameters and to verify the adequacy of using the PMMA-PC inside the field joints instead of using the UHPC. The following observations and concluding remarks can be drawn from this experimental study:

- The structural performance of the deck system with PMMA-PC closure joints used for full depth field connections was found to be comparable in terms of load and deflection capacities to the deck systems with UHPC connections.

- Either UHPC and PMMA-PC field connections adequately satisfy the service and ultimate load requirements specified in the AASHTO LRFD, where deck systems remain elastic without any major flexural or interface cracking in the joint or any rebar slippage.

- No flexural cracks were observed in the UHPC or the PMMA-PC joints until reaching the AASHTO LRFD ultimate load level, which is attributed to the higher tensile properties of the closure joint materials compared to the NSC.

- The initial stiffness of the UHPC deck system was found to be higher than that of the PMMA-PC, which is attributed to the higher mechanical properties of UHPC.

- Owing to the flexural capacity, the PMMA-PC transverse specimen with shorter loop splice length provided more strength and displacement capacity in addition to better load distribution.
across the precast panels than the UHPC or PMMA-PC transverse specimens with straight splice. This is due to the denser reinforcement distribution inside the joint.

- A new shear key type that provides elimination of formwork is proposed in this study and demonstrated a comparable performance with the existing shear keys.

- Both UHPC and PMMA-PC transverse specimens showed adequate load distribution capabilities all the way through failure without any shear failure or significant interface debonding. However, the UHPC deck system exhibited two localized main cracks at failure as opposed to several and multiple cracks in the PMMA-PC deck system at failure.

- The flexural capacity of the UHPC longitudinal specimen appeared to be higher than the PMMA-PC specimen. However, both specimens have a comparable behavior in terms of loads, deflections, and field joint performance at the ultimate AASHTO load level.

- PMMA-PC can be effectively used inside the full-depth precast bridge deck field joints using simplified reinforcement splice details without the need for post-tensioning or mechanical splicing. However, a 6-inch field joint width appeared to be sufficient to provide monolithic bridge deck systems in terms of load distribution.

### 7.2 Future Work

The results of this study demonstrate that the PMMA-PC can be potentially used inside the transverse and longitudinal field joints in precast bridge deck systems. While this research provides important research findings for the applicability of using a newly identified material (PMMA-PC) to the field joints of the bridge decks based on the static testing of five full-scale specimens, more research is needed to identify the performance of this material under a large number of cyclic loading and to determine the long-term performance of this material. The following research topics are recommended for future studies:

- Large-scale experimental testing of transverse and longitudinal full-scale specimens under a large number of loading cycles.

- Analytical and finite element studies to investigate different parameters such as the variation of the joint width, lap splice length, and reinforcement distribution inside the joints to determine their effect on the structural performance of the bridge decks and to provide new design recommendations for the reinforcement of the bridge decks based on the splicing requirements inside the field joints.
8. References


ACI (American Concrete Institute). Guide for the Use of Polymers in Concrete; ACI 548.1R-09; ACI: Farmington Hills, MI, USA, 2009.

ACI Committee. (2008). Building code requirements for structural concrete (ACI 318-08) and commentary. American Concrete Institute.

American Road and Transportation Builders Association (ARTBA) 2020 bridge report.


PCI (Precast/Prestressed Concrete Institute) PCI Bridge Design Manual, 3rd ed.; First Release, 2011.


