

# **THE IMPLEMENTATION OF FULL DEPTH UHPC WAFFLE BRIDGE DECK PANELS**

## **PHASE 1 FINAL REPORT**

April 2, 2010

Sponsored by  
**Highways for LIFE**  
**Technology Partnerships Program**

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16. Abstract In the U.S. today, there are over 160,000 bridges that are structurally deficient or obsolete with more than 3,000 new bridges added each year. Federal, State and municipal bridge engineers are seeking new ways to build better bridges, reduce travel times, and improve repair techniques; thereby reducing maintenance. Additionally, owners are challenged with replacing critical bridge components, particularly bridge decks, during limited or overnight road closure periods.  The use of Full Depth UHPC Waffle Deck Panels is currently gaining significant interest amongst numerous State DOT's and the Federal Highway Administration as a possible solution to these challenges. The first implementation of this system on a U.S. roadway will be made possible by this grant's funding. The objective of this research is to confirm that this system is a viable solution to the problems encountered by design engineers. It is hoped that the Full Depth UHPC Bridge Deck System will revolutionize the way bridges are designed in North America.  The implementation of this system is broken up into two phases within the overall scope of this project. Phase 1 includes the design and testing of a mock-up bridge section for verification of design assumptions, as well as, evaluating the feasibility of manufacturing and installing the deck elements. Phase 2 will consist of a full scale two-lane bridge on a secondary road in Wapello County, Iowa constructed of prestressed concrete girders and 14 UHPC deck panels.			
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## **INTRODUCTION**

In the U.S. today, there are over 160,000 bridges that are structurally deficient or obsolete with more than 3,000 new bridges added each year.<sup>(1)</sup> Federal, State and municipal bridge engineers are seeking new ways to build better bridges, reduce travel times, and improve repair techniques; thereby reducing maintenance. Additionally, owners are challenged with replacing critical bridge components, particularly bridge decks, during limited or overnight road closure periods.

In response to these challenges, researchers at the FHWA Turner Fairbanks research facility began investigating potential solutions in the year 2000. Prototype designs of full depth Ultra High Performance Concrete (UHPC) waffle deck panel systems have been in development over the past three years in both Europe and the United States.

The use of UHPC provides superior durability against chlorides, freeze-thaw effects, salt scaling, abrasion, accidental impact, fatigue, and overload, thereby extending the useful life of the bridge deck. Combining these positive attributes of UHPC and the efficiency of the waffle panel design provides an extremely durable option that enables faster construction and longer girder spans through the efficient use of materials and reduced weight. In addition to these benefits, the UHPC Waffle bridge deck system is applicable to both new construction and the rehabilitation of existing deteriorated bridge decks. The use of this solution for existing bridge rehabilitation not only restores the deck, but also provides opportunities for upgrading the load capacity of existing bridges, through the improved strength and reduced deck dead load.

The use of Full Depth UHPC Waffle Deck Panels is currently gaining significant interest amongst numerous State DOT's and the Federal Highway Administration (FHWA). By demonstrating this system is a viable solution to the problems encountered by design engineers it is hoped that the Full Depth UHPC Bridge Deck System will revolutionize the way bridges are designed in North America.

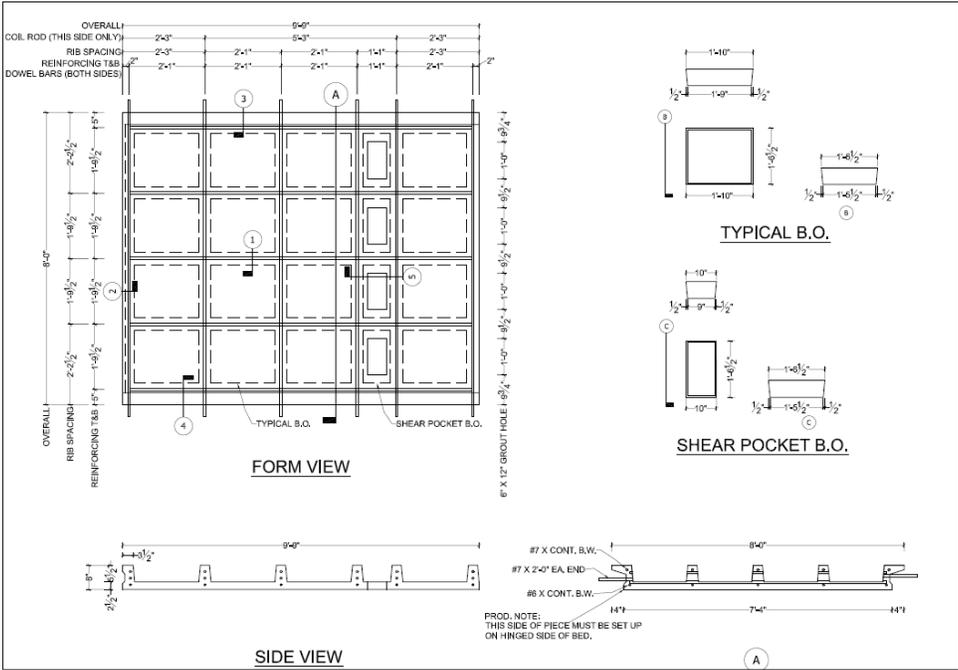
The implementation of the system in this project is broken up into two phases. Phase 1 includes the design and testing of a mock-up bridge section for verification of design assumptions, as well as, feasibility of manufacturing and installation of the deck elements. Phase 2 will consist of a full scale two-lane bridge on a secondary road in Wapello County, Iowa constructed of prestressed concrete girders and 14 UHPC deck panels. The following sections describe the results of Phase 1 and the upcoming events and schedule of Phase 2.

# 1.1 WORK COMPLETED IN PHASE ONE

The following sections summarize the progress that was made throughout Phase 1 of the project. As mentioned previously, the objectives of Phase 1 were to prototype and model the demonstration bridge planned for Phase 2. These objectives were met by producing two prototype UHPC waffle deck panels based on preliminary design work completed by the Iowa Department of Transportation (IDOT), modeling the section of the demonstration bridge that would undergo testing in Phase 1 using a Finite Element Analysis (FEA) to predict the response of the system and, load testing the prototype waffle slabs to confirm the design assumptions and FEA validity.

## 1.1.1 Waffle Panel Design and Fabrication

The design of the waffle slab was completed by the IDOT in late August of 2009, and shop drawings for use by Coreslab Structures (Omaha) Inc. production staff were complete by early September. Coreslab Structures and Lafarge North America consulted on the fabrication and aspects of the design relating to ease of production and requirements for UHPC joint fill respectively. The prototype panels were 8'-0" wide by 9'-9" long, with a number six bar in the top and a number 7 bar in the bottom of each rib (See Figure 1 below). One girder to girder span along the length of the bridge was modeled by the prototype panel. It was determined that two slabs would be necessary to test the transverse panel to panel joint.



**Figure 1**  
Prototype Panel Shop Drawing

The two prototype panels were produced in mid September 2009 at the Coreslab Structures plant in LaPlatte, NE, with the assistance of Lafarge North America's technical representative. The panels were cast using a displacement technique where the form was filled with fluid UHPC. The voids which make the ribs of the panel were forced downward into the UHPC to displace the material creating the shape of the panel. This technique was used to allow for the removal of the voids once the UHPC reached initial set, which is necessary to allow for unrestrained shrinkage and also to maintain the random orientation and consistent meshing of the fiber reinforcement. An additional benefit of casting the panels in an inverted orientation is that the final driving surface will be cast into the panels through the use of a form liner. By eliminating the application of a wearing surface placed in the field, the cost of the system is further reduced. Figures 2 through 6 depict the casting and production sequence.



**Figure 2**  
Final Form Setup

Figure 2 shows the reinforcing layout of the first casting. Uncoated reinforcing was used due to the fact the UHPC is effectively impermeable, eliminating the need for epoxy coated, galvanized, or stainless steel reinforcing to resist corrosion. The prototype panel formwork was made of wood in order to provide an inexpensive and temporary form. This strategy was employed due to the fact the design was preliminary and changes to the rib size, spacing, and joint profiles were possible between the prototype and demonstration panels. The final formwork will be made of steel and adjustable to fit different rib spacings for various panel configurations. In addition to the standard reinforcing, several monitoring instruments were also cast into the panels to monitor the internal responses of the UHPC and reinforcing.



**Figure 3**  
Casting

Figure 3 shows the placing of the UHPC into the form. The UHPC was placed with a specially designed bucket that is as wide as the form, which aligns the steel fibers in the UHPC in the longest direction of the panel. This alignment helps to increase the flexural strength of the panels. The bucket was moved along the length of the form and kept behind the leading edge of the flow to eliminate any discontinuity in the fiber orientation. The form was filled to a predetermined level and then the voids were set as an assembly (See Figure 4 page 3). Each of the prototype panels required approximately one cubic yard of UHPC.

By placing the voids as an assembly, the panels can be cast substantially faster and with less chance of error. The UHPC is displaced by the voids as they are lowered into position to create the final shape off the piece. Placing the voids prior to the casting would create a cold joint effect at the corners of the voids. This is due to the fact that as the UHPC flows into itself around the corners of the voids the fibers will not cross the two flows of UHPC.

As mentioned previously this casting method also allows the voids to be removed after the initial set off the UHPC, which allows the UHPC to shrink without inducing internal stresses. This is very important because the UHPC will reach initial set within 12-14 hours, but will not reach release strength for approximately 40 hours after casting. Damage from shrinkage can occur during this time if the appropriate precautions are not taken.



**Figure 4**  
Placing the Voids



**Figure 5**  
Formwork Removed

Figure 5 on page 5 shows the panel after the side forms and voids were removed. After the panels reached the required 14,000 psi release strength, they were moved from the casting area to the curing area. The panels were lifted into the vertical position, rotated 180 degrees and lowered back to the horizontal position in the proper orientation by the casting bed. This technique was used to reduce the handling stresses on the panels. Even though the concrete strength is very high at release, the sections where the panels are rotated from are very thin, limiting the amount of stress that can be taken without cracking. The critical section is at the very thin area where the toe of the transverse joint edge and the longitudinal rib intersect. By rotating the piece with the casting bed the issue of handling stresses is eliminated.



**Figure 6**  
Completed Panels

After the pieces were cast, they were cured for 48 hours at 195 °F as required by Lafarge for maximum strength and durability. Before the pieces were steam cured they measured approximately 15,000 psi compressive strength. A test cylinder was broken before the steam was turned off to verify that the panels had cured correctly, and the compressive strength of the UHPC at that time was 29,800 psi, exceeding the required design strength of 24,000 psi. After curing was complete, the panels were loaded on a truck and transported to ISU to be load tested.

### **1.1.2 Testing Equipment and Plant Modifications**

Special equipment is required to produce the waffle slabs and to cast, cure, and test the UHPC. Products were purchased with the use of grant funds including formwork, quality control testing equipment, and UHPC placing and curing equipment. The equipment necessary to produce the UHPC waffle slabs is described in the sections below.

#### ***1.1.2.1 Formwork***

No existing formwork matched the profiles of the waffle design or the edge conditions needed for the panel to panel joints. New formwork was designed to cast the slabs. At the time of the prototype casting, the final design was unknown because testing would be required to validate the design. By researching the typical bridge designs in the Iowa Highway system, it was determined that the maximum length would be 25 feet long. The slab design was far enough along at the time of prototype casting to determine that the width would be 8 feet. The casting bed was fabricated using this information and was utilized to rotate the prototype waffle slabs manufactured in Phase 1. The remaining form work for the demonstration project will be purchased once the panel design is completed.

#### ***1.1.2.2 Casting and Curing Equipment***

Special equipment is necessary to mix, place, and cure UHPC. A concrete placing bucket built to match the width of the form was purchased in Phase 1. As mentioned previously, the bucket was needed to create the alignment of the steel fibers. The pieces were cured using propane heaters for the initial curing and a manually monitored steam curing setup was used to reach the final cured state. An automated electric bed heat system and steam curing system will be purchased in Phase 2.

#### ***1.1.2.3 Quality Control Equipment***

Testing equipment is required to perform physical testing of fresh UHPC. This equipment was purchased before the prototype panel casting in September and included a flow table, vibrating table, molds for prisms and cylinders, and scales. Coreslab Structures personnel were trained on the equipment and related procedures.

**1.1.3 Laboratory Load Testing**

The prototype waffle slabs were shipped to the structures lab at ISU. A testing proposal was created by ISU, and it included testing the panels and the transverse joint in service, fatigue, and ultimate loading scenarios. Figure 7 below shows the test protocols. In order to more closely represent the field conditions of the Phase 2 bridge, it was determined that a prestressed concrete beam would need to be used as the support structure of the test setup. Fortunately one was available and donated by Coreslab Structures and shipped with the panels to ISU.

An UHPC joint casting was required and completed in the ISU structures lab with the help of Lafarge’s technical representative. The transverse joint between the panels and the longitudinal area along the length of the support beams were cast with Ductal UHPC in the ISU Structures lab in late November 2009.

An FEA model of the completed test setup was created by ISU and several runs were made to locate the worst case loading scenarios before the physical testing began. This helped limit the amount of physical testing required as well as decrease the amount of time required for the testing phase of the project. The model closely represented the results of the physical tests, and will enable future projects incorporating UHPC to be designed and investigated more efficiently and with less physical testing.

Test Number	Test Description	Location	Maximum Load
1	Service load test panel-2	Center of the panel	1.33 <sup>a</sup> x 16kips = 21.3 kips
2	Service load test on transverse joint	Center of the joint	1.75 <sup>b</sup> x 16 kips = 28 kips
3	Fatigue test on the transverse Joint	Center of the joint	28 kips ( 1 million cycles)
4	Ultimate load test of transverse joint	Center of the joint	48 kips
5	Fatigue test on the panel-1	Center of the panel	21.3 kips ( 1 million cycles)
6	Ultimate load test of the panel	Center of the panel	40 kips

a, b –dynamic allowance factors from AASHTO Table 3.6.2.1-1

**Figure 7**  
Test Protocols and Sequence

The results of the testing were promising. In summary, the 21.3 kip load placed on the panel caused two hairline cracks in the rib below the loading location, and the 28 kip load applied to the joint, caused a barely visible crack to form on the bottom of the joint as predicted by the FEA model. Fatigue loading applied for the specified 1,000,000 cycles, did not have any noticeable effect on the strength or durability of the panels.

Skid resistance of the driving surface was also tested. Seven commercially available form liners were ordered and samples of UHPC textures were cast and tested according to ASTM E303 standard method of testing skid resistance.

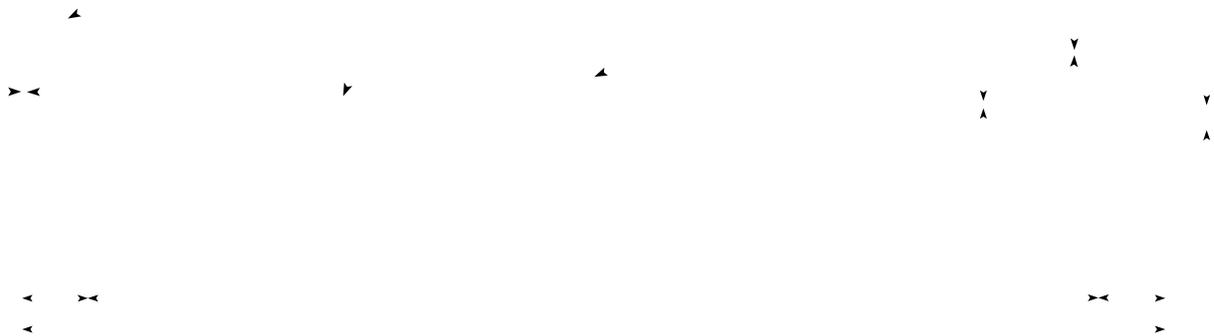
Additional information regarding the technical aspects of the testing and results is included in Appendix A. In general the panels performed very well and appear to be more than capable of holding up to the rigors of use on a public highway. Phase 1 was viewed as a success by all team members involved.

## 2.1 WORK SCHEDULED FOR PHASE TWO

The work scheduled for Phase 2 of the project includes construction of the full scale demonstration bridge in Wapello County, a life cycle cost analysis, and a final detailed report concluding all the findings from both Phase 1 and Phase 2.

### 2.1.1 Demonstration Project

The demonstration bridge in Wapello County will be 33'-2" wide by 60'-0" long consisting of 14 UHPC slabs on conventional prestressed concrete girders. The slabs will be supported on 5 "B" Beam girders spaced at 7'-4" with 1'-11" overhangs. The panels will be jointed at the crown longitudinally with UHPC. UHPC will be also be used to fill the transverse joints. A preliminary bridge cross section is shown in Figure 8 below.



**Figure 8**  
Cross Section of Demonstration Bridge

Wapello County will finalize the design of the demonstration bridge approaches and layout, and Iowa DOT will finalize the design of the demonstration bridge and deck panels in early April 2010. The letting is tentatively scheduled for early July 2010, with construction starting shortly after. Panel production should begin in June and will be completed in time to meet the proposed construction schedule.

### 2.1.2 Plant Modifications

Plant modifications to accommodate the casting of the demonstration project waffle slabs will take place during the months of April through June 2010. These modifications include the installation of a fiber addition system to the mixer, the purchase of the

remaining formwork necessary to cast the panels, installation of a bed heat system to efficiently cure the fresh UHPC, construction and installation of a steam curing system and chamber, and the purchase of equipment that eliminates the need for handling devices in the wearing surface of the bridge.

### **2.1.3 In-Situ Testing and Evaluation**

A specific instrumentation plan will be developed to evaluate the structural performance of the bridge and specifically the waffle slab component, using strain, deflection and acceleration sensors. It is anticipated that the instrumentation plan will enable the evaluation of at least the following structural characteristics:

- Deck strain / stress levels and load distribution
- Overall superstructure live load transverse distribution and strain / stress levels
- Overall bridge superstructure deflections
- Bridge end member restraint
- Edge stiffening
- Dynamic amplification factor

After the bridge construction is completed, the instrumentation plan noted above will be used during the performance of a live load test to evaluate the structural performance of the bridge. This testing will provide insight into the validity of the bridge design assumptions and identify any potential undesirable serviceability behavior. A second similar load test will be performed after the bridge has been in service for approximately three months to determine any potential changes in service behavior. A visual inspection will also be performed prior to each load test.

The testing will be conducted with controlled live loads at service level using a typically loaded standard truck. A series of different transverse static load positions will be used to create worst case loading conditions in the waffle slab and the bridge. A series of dynamic tests will also be conducted to provide some quantification of bridge dynamic performance.

A final report will be completed detailing the Phase 2 production processes and the findings from the in-situ testing. In addition to the Phase 2 report, a detailed life cycle cost analysis (LCCA) will be created to display the cost savings and feasibility of the full depth UHPC waffle deck system for use in future projects. Included in the LCCA will be a discussion of the costs of first deployments of the technology where fabrication facility construction costs will be included and repeat project costs where the fabrication facility costs will be removed. Also, design recommendations and production standards will be outlined in a separate document.

The Phase 2 report, design standards, and LCCA will help make the full depth UHPC bridge deck system a marketable and widely used construction system. This will provide the FHWA and departments of transportation a solution that will help extend the useful life of North America's bridges and highway system.

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1

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# **APPENDIX A**

**Technical report on the test protocols, results, and performance of the  
Full Depth UHPC Waffle Deck Panel System prepared by  
Iowa State University InTrans**

# **THE IMPLEMENTATION OF FULL DEPTH UHPC WAFFLE BRIDGE DECK PANELS**

## **PHASE 1 FINAL REPORT**

### **Structural Characterization of UHPC Waffle Bridge Deck Panels and Connections**

April 24, 2010

Sponsored by  
**Highways for LIFE**  
**Technology Partnerships Program**

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<b>16. Abstract</b> <p>The AASHTO strategic plan in 2005 for bridge engineering identified extending the service life of bridges and accelerating bridge construction as two of the grand challenges in bridge engineering, with the objective of producing safer and economical bridges at a faster rate that have a minimum service life of 75 years and reduced maintenance cost to cater the country's infrastructure needs. Previous studies has shown that a prefabricated full-depth precast concrete deck system is an innovative technique that accelerates the rehabilitation process of a bridge deck extending its service life with reduced user delays, and community disruptions and lowering its life-cycle costs. Previous use of Ultra high performance concrete (UHPC) for bridge applications in the United States have been proven to be efficient and economical due to its superior structural characteristics and durability.</p> <p>The design of full depth UHPC waffle deck panel systems have been developed over the past three years in Europe and the U.S. A full-scale, single span 60 ft long and 33 ft wide prototype bridge with full depth prefabricated UHPC waffle deck panels has been planned for a replacement bridge in Wapello County, Iowa. The structural performance characteristics and the constructability of the UHPC waffle deck system and its critical connections were studied through an experimental program at the structural laboratory of Iowa State University (ISU). Two prefabricated, full-depth, UHPC waffle deck (8ft x 9ft 9 inches x 8 inches) panels were connected to 24-ft long precast girders and the system was tested under service, fatigue and ultimate loads. Based on the test results, test observations and the experience gained from the sequence of construction events such as fabrication, casting of transverse and longitudinal joints, a prefabricated UHPC Waffle deck system is found to be a viable option to achieve the goals of AASHTO strategic plan.</p>			
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# Structural Characterization of UHPC Waffle Bridge Deck Panels and Connections

**Phase 1 Final Report**

**April 24, 2010**

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# ABSTRACT

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The AASHTO strategic plan in 2005 for bridge engineering identified extending the service life of bridges and accelerating bridge construction as two of the grand challenges in bridge engineering, with the objective of producing safer and economical bridges at a faster rate that have a minimum service life of 75 years and reduced maintenance cost to cater the country's infrastructure needs. Previous studies has shown that a prefabricated full-depth precast concrete deck system is an innovative technique that accelerates the rehabilitation process of a bridge deck extending its service life with reduced user delays, and community disruptions and lowering its life-cycle costs. Previous use of Ultra high performance concrete (UHPC) for bridge applications in the United States have been proven to be efficient and economical due to its superior structural characteristics and durability.

The design of full depth UHPC waffle deck panel systems have been developed over the past three years in Europe and the U.S. A full-scale, single span 60 ft long and 33 ft wide prototype bridge with full depth prefabricated UHPC waffle deck panels has been planned for a replacement bridge in Wapello County, Iowa. The structural performance characteristics and the constructability of the UHPC waffle deck system and its critical connections were studied through an experimental program at the structural laboratory of Iowa State University (ISU). Two prefabricated, full-depth, UHPC waffle deck (8ft x 9ft 9 inches x 8 inches) panels were connected to 24-ft long precast girders and the system was tested under service, fatigue and ultimate loads. Based on the test results and test observations, the experience gained from the sequence of construction events such as fabrication, casting of transverse and longitudinal joints, a prefabricated UHPC Waffle deck system is found to be a viable option to achieve the goals of AASHTO strategic plan.

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# Introduction

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Today there are over 160,000 bridges in the nation that are structurally deficient or obsolete with more than 3,000 new bridges added to this list each year (Bhide 2001). Many bridges are subjected to weights, loads, and traffic volumes exceeding limits of their original design while current bridge inspection methods do not detect all structural problems encountered in the field. Deterioration of the bridge deck is a leading cause for the obsolete and/or deficient inspection rating of the bridges ([http://www.zellcomp.com/infrastructure\\_crisis.html](http://www.zellcomp.com/infrastructure_crisis.html), Stantill-McMcillan and Hatfield 1994). Federal, State and municipal bridge engineers are seeking alternative ways to build better bridges, reduce travel times, and improve repair techniques, thereby reducing maintenance costs of bridge infrastructure. Additionally, owners are challenged with replacing critical bridge components, particularly rapidly deteriorating bridge decks, during limited or overnight road closure periods. Therefore, there is an impending need to develop and use longer-lasting materials and innovative technologies to accomplish safe and fast construction of high quality bridges and highways.

To overcome the nation's aging bridge infrastructure requires development of cost efficient, widely applicable, and long-lasting bridge elements and systems and accelerated bridge construction techniques. To increase longevity and reduce maintenance costs, the potential use of ultra-high performance concrete (UHPC) in bridges is gaining significant interest amongst several State Departments of Transportations (DOTs) and the Federal Highway Administration (FHWA). The use of full depth precast deck panels in bridges is not new, nor is the use of UHPC as deck panel joint fill. Several U.S. State and Canadian Provincial DOT's have explored the use of full depth precast deck panels in bridges. UHPC has also been used as joint fill material by the Ontario Ministry of Transportation on full depth solid deck panels made from High Performance Concrete (Perry et al. 2007).

In support of reducing the aging bridge infrastructure stock in the U.S., innovative use of UHPC in bridge applications has been underway for the past several years. The State of Iowa has been in the forefront of this mission with implementations of the first UHPC bulb-tee and Pi girders in bridges and development of an H-shaped UHPC precast pile for foundation application (Vande Voort et al. 2007; Keierleber et al. 2007, Sritharan 2009). The interest in using UHPC for highway bridge decks has been ongoing in the U.S. since the year 2000. Research and Development (R&D) at the FHWA Turner Fairbanks facility commenced in 2000 and prototype bridge decks utilizing UHPC have been under development since that time. Various types of UHPC precast deck systems have been prototyped during this period. However, to date, there are no UHPC precast deck panels in service in our highway system.

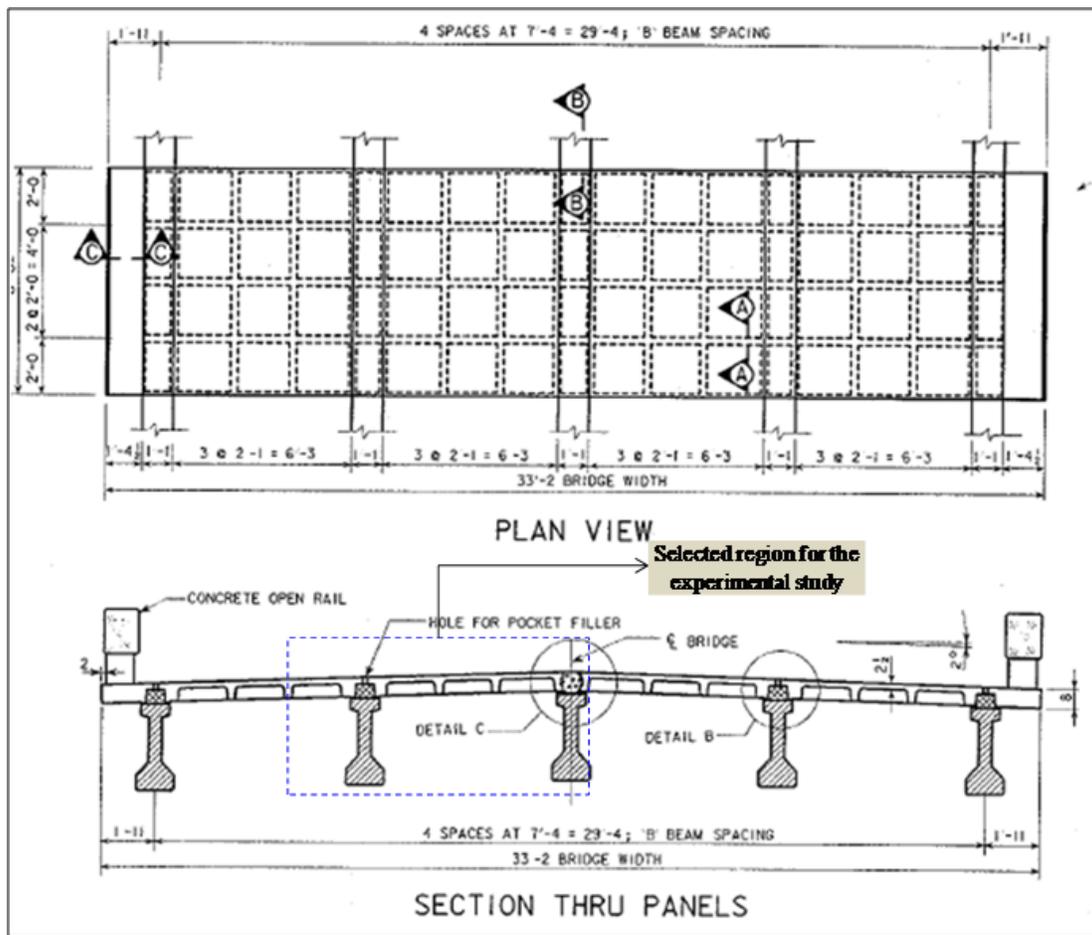
The design of full depth UHPC waffle deck panel systems have been developed over the past three years in Europe and the U.S. The FHWA explored this system and published a Techbrief on this topic (FHWA 2007). Significant research and development, analysis, design, and prototyping of separate components of this innovation have also been explored (i.e., joint, shear, key, skid resistance, durability, etc.) (Perry et al. 2007). Nevertheless, these innovations have not been installed in the U.S. highway system. State DOTs from Virginia, Florida, Iowa and New York have expressed interest in utilizing UHPC waffle deck panel system if the performance of the system is proven satisfactory through experimental testing. The main reason for the broad interest in the UHPC waffle deck panel is that this concept is applicable for both new bridges as well as for rehabilitation of existing deteriorated bridge decks.

The first application of the full depth UHPC waffle deck panel has been planned for a replacement bridge in Wapello County, Iowa. With the deck panels designed specifically for this project, the validation of the assumed structural performance characteristics of the UHPC waffle deck, critical connections, system performance, and rideability of the panel surface were performed through an experimental program at the structural laboratory of Iowa State University (ISU). For this project, two prefabricated, full-depth, UHPC waffle deck (8ft x 9ft 9 inches x 8 inches) panels were connected to 24-ft long precast girders and the system was tested under service, fatigue and ultimate loads. In addition, the response of the system was evaluated using a detailed 3D finite element model. The results from this investigation and recommendations for using these panels in the Wapello County bridge project are presented in this report.

# Waffle Deck Panel Fabrication

## Prototype Bridge

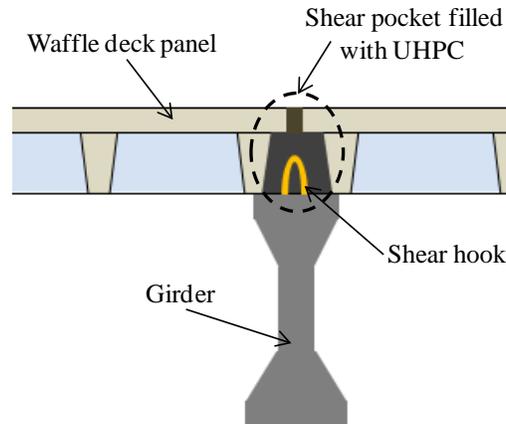
A full-scale single-span, two-lane, prototype, full-depth bridge system was designed as a replacement of an existing bridge in Wapello County, Iowa, and the details of this structure are shown in Figure 1. The prototype bridge is 33 ft wide and 60 ft long and it consisted of prefabricated, full-depth, precast concrete panels installed on five standard Iowa “B” girders placed at a center-to-center distance of 7ft 4 inches.



**Figure 1** Plan and cross section details of the proposed UHPC Waffle Deck Bridge in Wapello County.

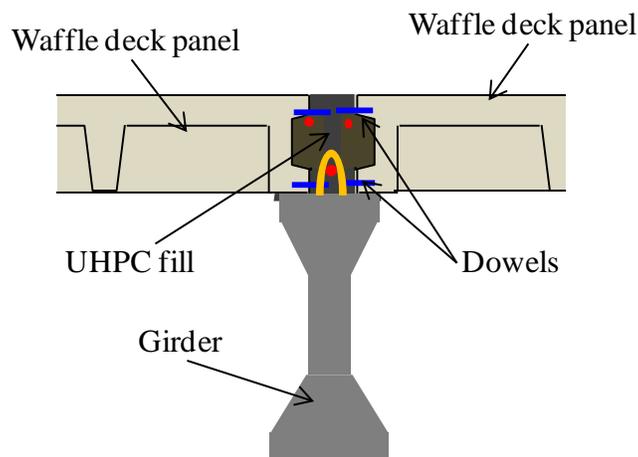
The waffle deck is expected to be made fully composite with girders using the following connections:

1. Shear pocket connection: This connection is formed between the girder and the waffle deck using shear pockets. In this connection, the shear pockets in the waffle deck are filled with UHPC and a shear hook extended from the girder is embedded into the UHPC fill (see Figure 2). This will cause the girders and the waffle deck to act in a composite manner.



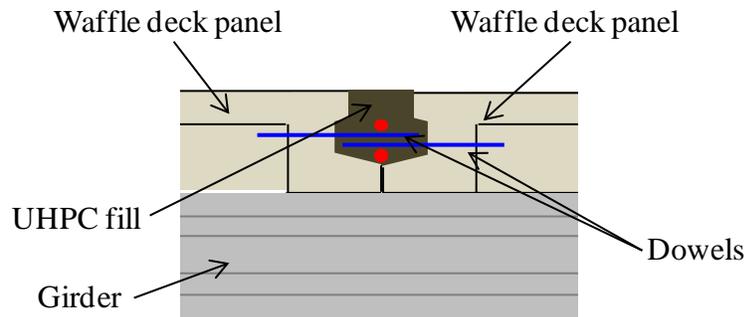
**Figure 2** Shear pocket connection details between girder and the waffle deck

2. Waffle panel to girder longitudinal connection: This connection is formed between the central girder and the waffle deck. In this connection, the dowel bars from the panels and the shear hook from the girder are tied together with additional reinforcement along the girder length and the gap between the panels is filled with UHPC (see Figure 3). This connection will provide a positive moment connection between the girder and the panels.



**Figure 3** Connection details between the central girder and the waffle deck

The bridge consisted of waffle deck panels which were 16 ft wide (full width) and 8 in. thick. They are connected across the length of the bridge using a transverse joint connection shown in Figure 4. In this connection, the dowel bars from the panels are tied together with additional transverse reinforcement and the gap between the panels is filled with UHPC (see Figure 4). This connection will provide continuity between the panels and facilitate load transfer between the panels.



**Figure 4** Connection details between the waffle deck panels

## Panel Details and Prefabrication

The UHPC waffle panels were designed with conventional mild steel reinforcement primarily to resist the transverse flexural moments (i.e., for moments induced about the bridge longitudinal axis) in accordance with the current AASHTO slab deck design provisions (AASHTO, 2007). This resulted in Grade 60 No. 7 ( $d_b = 0.875$  in, where  $d_b$  is diameter of the bar) and No.6 ( $d_b = 0.75$  in.) mild steel reinforcement located at  $1\frac{1}{4}$  inches from the bottom surface and at  $1\frac{5}{8}$  inches from the top surface of the panel, respectively. In the longitudinal direction, the panels were detailed with Grade 60, No. 7 and No.6 mild steel reinforcement at  $2\frac{1}{8}$  inches from the bottom surface and at  $2\frac{3}{8}$  inches from the top surface, respectively. All the reinforcement was provided along panel ribs in both directions. Figure 5 shows the cross-section and reinforcement details of a typical waffle deck panel designed for the prototype bridge.

For the experimental investigation, a waffle deck region between two adjacent girders as identified in Figure 1 was chosen. Accordingly, two waffle deck panels with dimensions of 8 ft (length) by 9 ft 9 in. (width) were fabricated by Coreslab Structures (Omaha) Inc. in September 2009. Commercially available standard Ductal® mix was used as the UHPC mix design.

Figure 6 illustrates the sequence of steps used for casting the waffle-shaped deck panels. The formwork for the panels was designed and constructed by Coreslab Structures (Omaha) to cast

them in an upside position, to facilitate a flat finish for the driving surface and easy placement and removal of the voids (see Figure 6e). A trough system with nearly the same width as the panel was used to pour the UHPC in place. Standard compression test cylinders (3 in. x 6 in.) and modulus beams were cast for every pour to establish the strength gain of the panel with time. A standard flow table was used to measure the flowability of UHPC for each pour. After the pour, the panels were covered with plastic tarp and subjected to cure at 110° F for two days using torpedo style propane heater. After 7 days, the slabs were heat treated at 190°F+/-5°F for a period of 48 hours using steam to maintain 100% relative humidity. The test cylinders and modulus beams were also subjected to the same curing conditions as the panels and were tested by Coreslab Structures (Omaha) Inc. These tests were conducted through a subcontract by the precaster at regular intervals to monitor the strength gain with time. Table 1 shows the details of the UHPC strength gain with time. The average 28-day compressive strength of the concrete was found to be 21,981 psi, which is below the expected value of about 26,000 psi. This noticeable discrepancy is attributed to inadequate quality control performed during the compression testing of the cylinders. While more improved compression tests should be conducted during construction of the prototype waffle panels, it is worth noting that the expected compression strength was achieved for similar compression cylinders produced by the same precaster in Omaha as part of a Iowa DOT funded project on UHPC piles (Vande Voort et al. 2008).

**Table 1** Strength gain of UHPC in the waffle deck panels

Panel-1 (UWP1)		Panel-2 (UWP2)	
Time (hours)	Strength (psi)	Time (hours)	Strength (psi)
22	1800	20	850
24	4500	26	5000
26	6250	44	10650
44	11650	52	13800
52	13400	-	-
28 days (post-steam cure strength)	21843	28 days	22120

After curing, the panels were transported to the Iowa State University's (ISU) Structures Laboratory. Both the deck panels exhibited a very smooth surface on all sides that were in contact with the formwork. Other surfaces, especially the underside of the panels appeared somewhat rough.

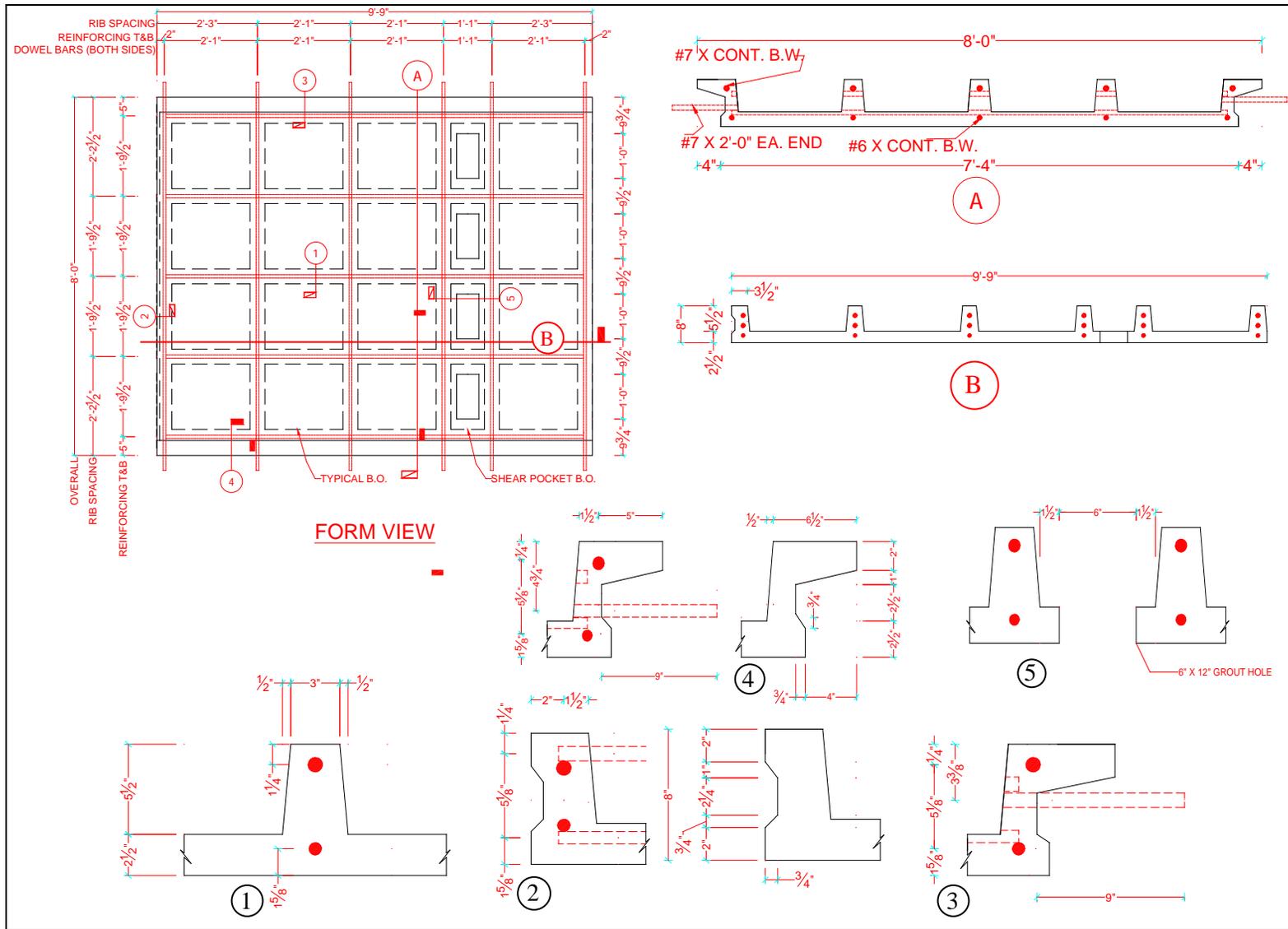
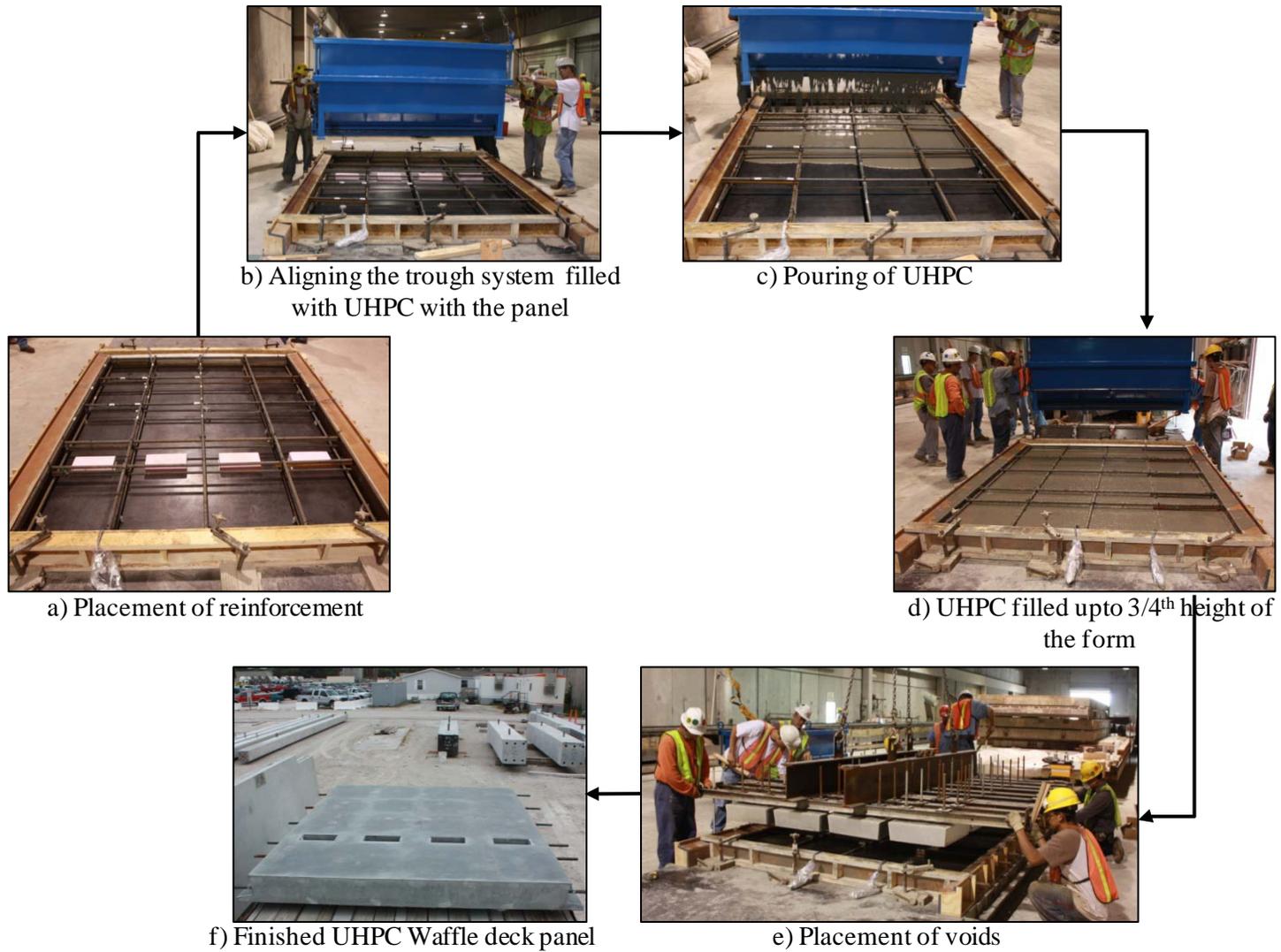


Figure 5 Reinforcement details of the UHPC waffle deck test panels.



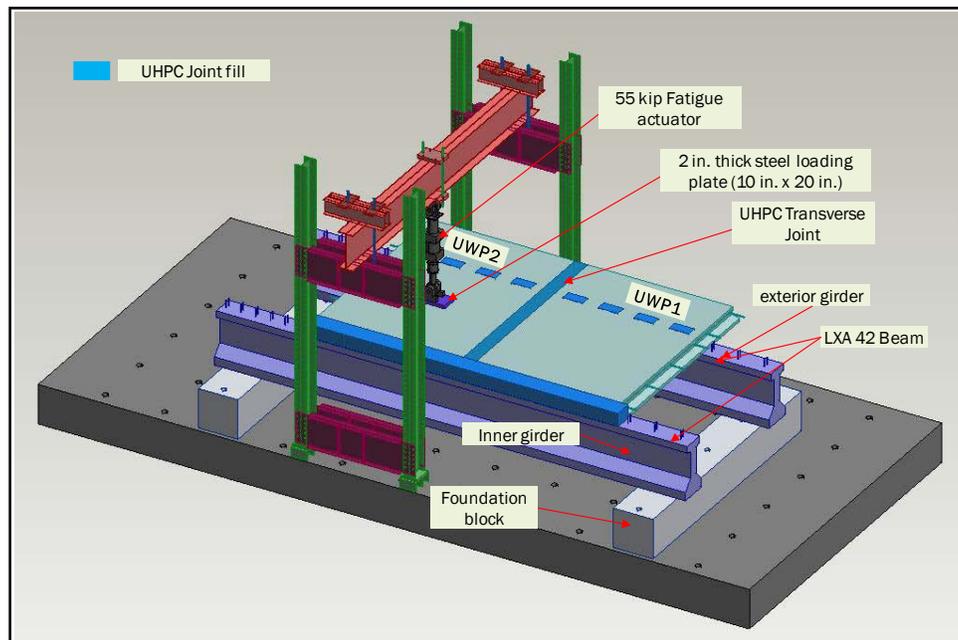
**Figure 6** Construction sequence of the UHPC Waffle deck panel at the precast plant.

# Experimental Investigation

This section discusses the test setup, instrumentation and loading protocols used for the experimental investigation of the UHPC waffle deck system as well as the test observations and results.

## Test Setup

A schematic of the test setup used for the UHPC waffle deck system is shown in Figure 7, which was established to closely replicate the critical regions of the field structure in the laboratory. As noted earlier, the setup represented an end section of the prototype bridge encompassing two adjacent girders including a portion of an exterior girder. The UHPC deck panels were supported on two 24-ft long prestressed concrete girders having cross sections of girder type LXA 42, which were simply supported at the ends on concrete foundation blocks as shown in Figure 7.

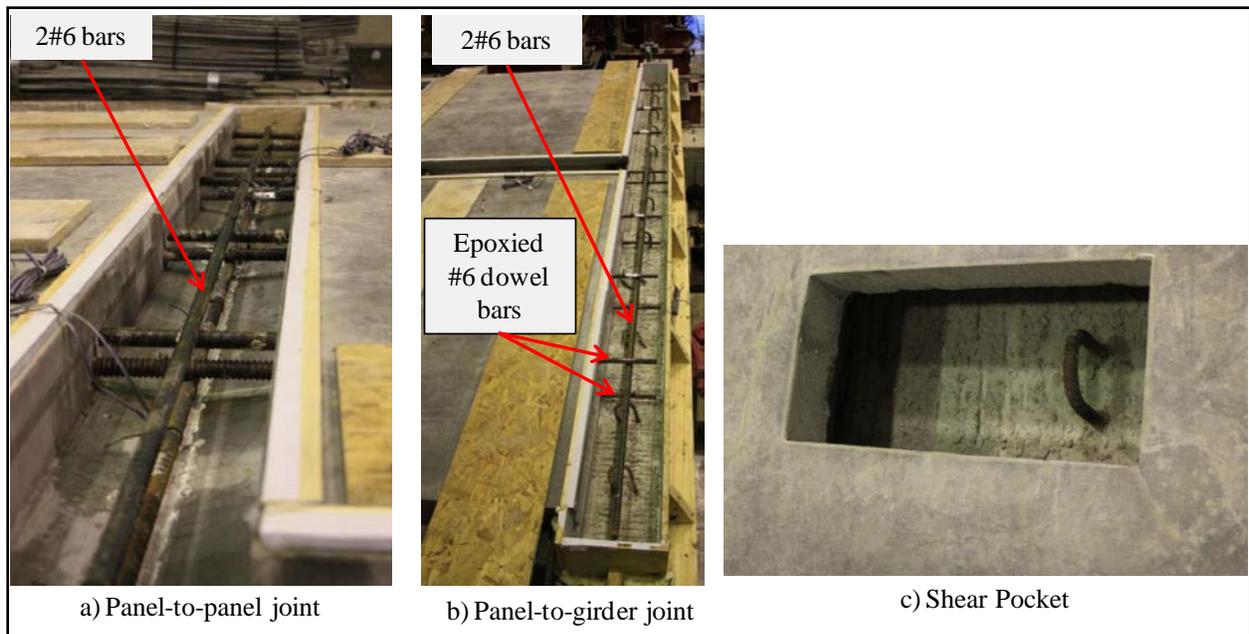


**Figure 7** Schematic of the test setup used for testing of the UHPC Waffle deck panel system.

The foundation blocks were post-tensioned to the strong floor of the laboratory using a total of four one-inch diameter high strength threaded rod to prevent them from experiencing any lateral movement during testing. The girders were established by cutting a 48-ft long LXA 42 prestressed concrete girder, which was used by Iowa DOT as a standard girder in the past. The girders were placed on the foundation blocks at a center-to-center distance of 7ft and 4 inches

between them. They were supported on rollers at one end and pinned at the other end. After the girders were set in place, the waffle deck panels were placed on the girders with specified bearing length (1.25 inches) on each girder.

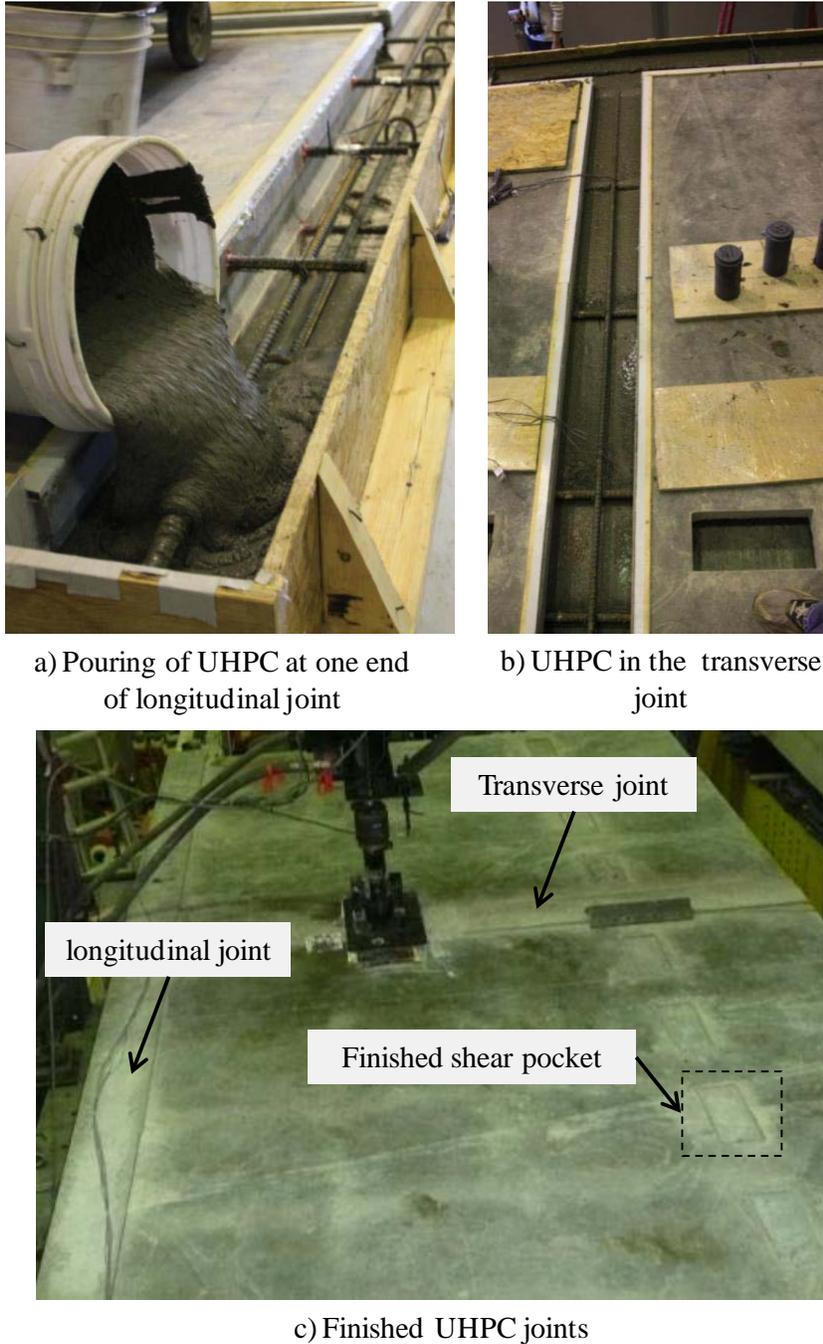
To establish a positive moment connection between the waffle deck and the interior girder, 12 in. long Grade 60, No.6 ( $d_b = 0.75$  in, where  $d_b$  is diameter of the bar) mild steel dowel bars were embedded at the left end face of the panels using high strength epoxy (see Figure 8b). In addition, two No.6 bars were placed and tied to the dowel bars along the girder length to represent the effect of the continuous slab over the inner girder, which is expected in the prototype bridge. Two No.6 bars were provided in the panel-to-panel joint (transverse joint) as the main reinforcement to resist the bending moment about the longitudinal axis (see Figure 8a). The connection between the exterior girder and the waffle deck was established using a shear pocket (see Figure 8c). Every shear pocket contained at least one shear hook extending from the girder.



**Figure 8** Details of the reinforcement provided in various joints.

The joint between the two deck panels as well as those between the panels and the girders were cast using UHPC mixed in the laboratory at ISU. UHPC required for the joint fill was prepared in total of five batches using two Imer Mortarman 750 mixers. Every batch used nine bags of Ductal® premix and produced 5.3 ft<sup>3</sup> of UHPC mix. Standard cylinders (3in. x 6 in.) were cast for every batch to establish the strength gain of the joint fill with time. A standard flow table was used to measure the flowability of every batch of UHPC and measured values are presented in Table 2. UHPC was poured from one end of the longitudinal joint (panel-to-girder

joint) and it travelled along the entire length of the joints (see Figure 9). After casting, all UHPC joints were covered with form plywood to minimize any moisture loss. The test cylinders were also subjected to the same curing conditions as the joints. They were tested at ISU in regular intervals to monitor the strength gain with time. Table 3 shows the details of the UHPC strength gain with time.



**Figure 9** UHPC joint pour

**Table 2** Measured flow values for the UHPC joint fill

Batch number	Mix temperature (C <sup>0</sup> )	Flow		Comments
		Static (in.)	Dynamic (in.)	
1	30	8.5	9.75	longitudinal joint
2	29	9	10.125	longitudinal joint
3	30	8.75	9.75	transverse joint
4	26	9.5	Off table	Shear pockets
5	27	8.5	9.75	Shear pockets

**Table 3** Strength gain of UHPC in the joints

Time (days)	Strength (psi)
3	11591
14	15201
28	18831

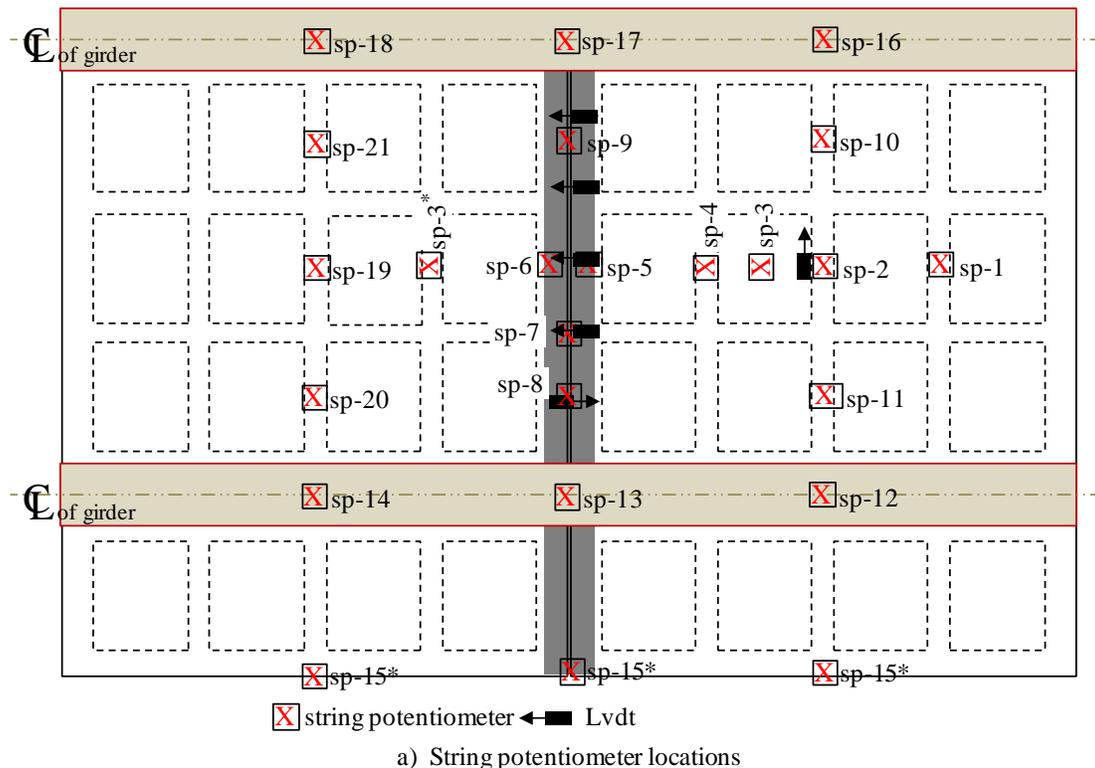
The test preparation work began immediately upon completion of the joints. The plywood was removed after 3 days and the testing got underway 34 days after casting of the UHPC joints. A  $\pm$  55kip capacity fatigue hydraulic actuator, mounted to a steel reaction frame as shown in Figure 7, was used to apply the load to the test unit. The frame was post-tensioned to the strong floor of the laboratory using four 1 1/4 in.-diameter high strength bars. A 10 in. by 20 in. steel plate was used at the loading end of the actuator to simulate a truck wheel load on the panel for all testing described in this report.

## Instrumentation

This section presents the details of instrumentation used to monitor the performance of the waffle deck system during testing. Several different types of instruments were used for this study, including linear variable differential transducers (LVDTs), string potentiometers and strain gauges. String potentiometers were used to measure the vertical displacements of the deck panels as well as the bridge girders. The locations and identifications used for these string potentiometers are shown in Figure 10a and Figure 10b.

LVDTs were placed along the panel-to-panel joint region to capture any possible gap opening along the transverse joint during testing. LVDTs were also used down the depth of the

panels to measure average strains and neutral axis depth during loading (see Figure 10c). Also, the width of the flexural cracks along the transverse ribs was monitored during testing using LVDTs. A number of embedded strain gauges were used to measure the strain demands in the reinforcement along the transverse and longitudinal ribs of the panels and in the reinforcement placed within the joints. The No.6 ( $d_b = 0.75$  in., where  $d_b$  is diameter of the bar) dowel bars epoxied into the side face of the deck panels were also gauged to monitor the strain demands in these bars during testing. Figure 11 and Figure 12 show the locations of the strain gauges on the bottom and top deck reinforcement, respectively. During the test, the data from all gauges and displacement devices were recorded using a computer based data acquisition system.

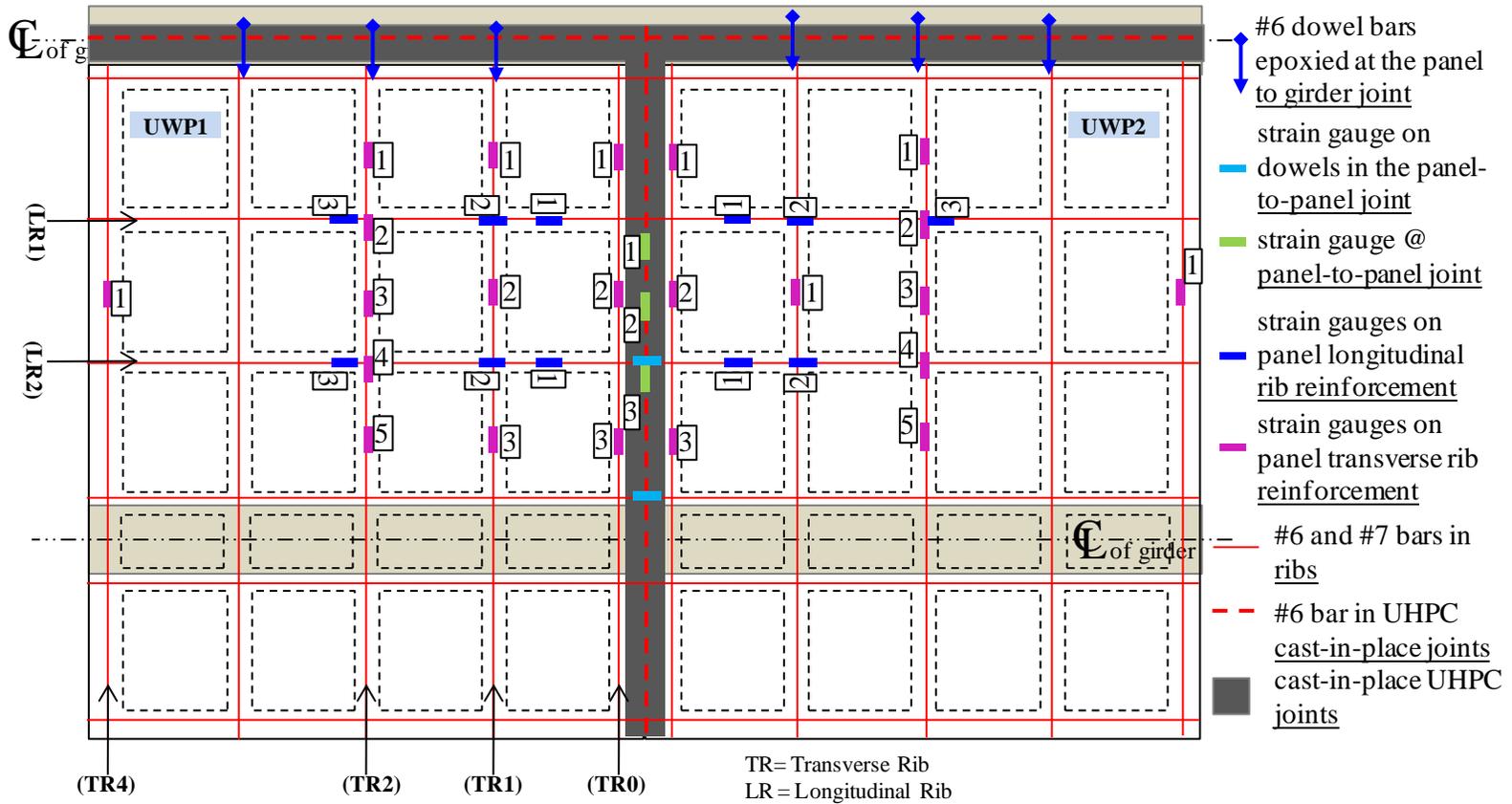


b) String potentiometer configuration

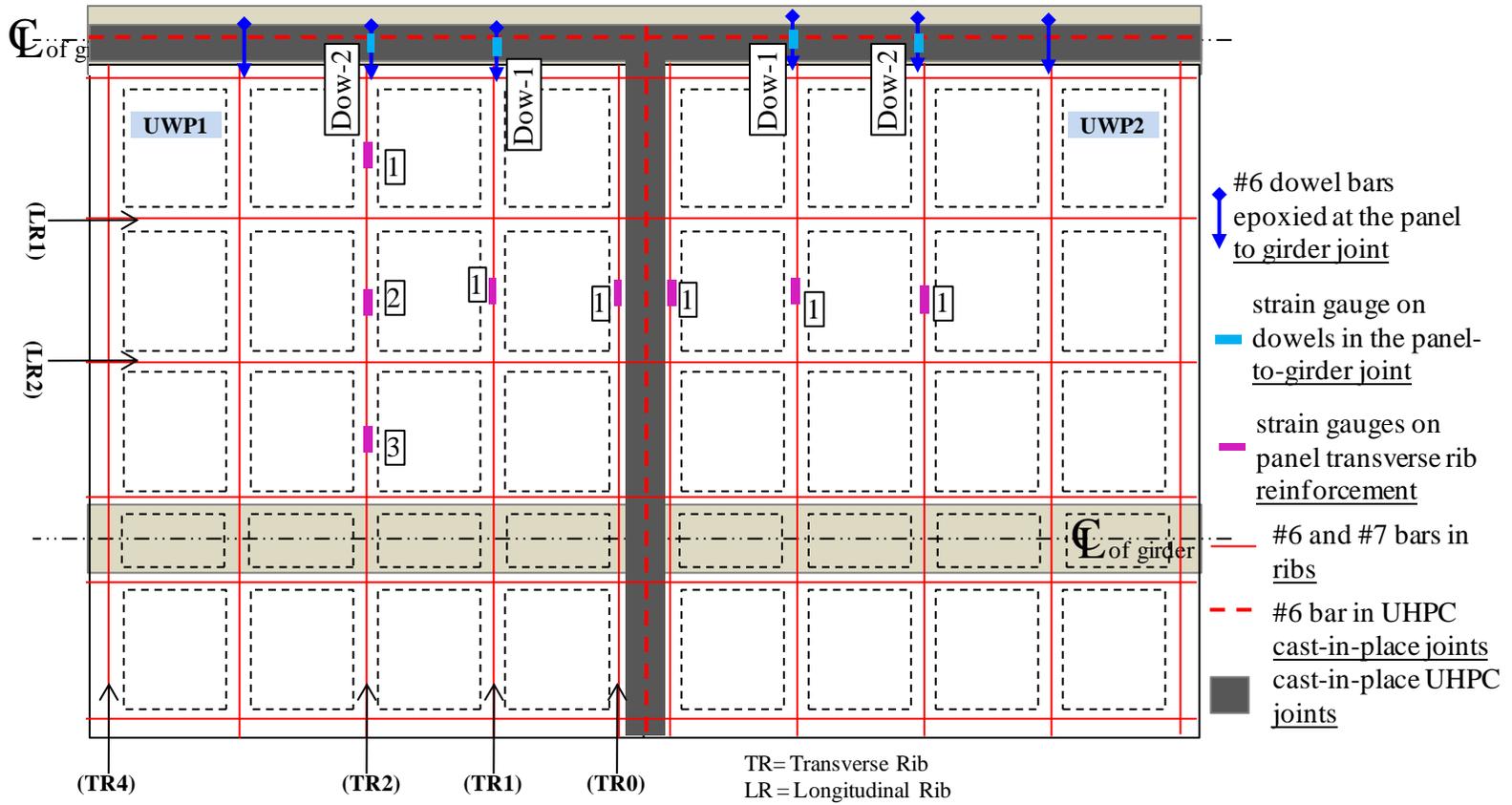


c) LVDT attached to a transverse rib

**Figure 10** Schematic of the displacement transducers mounted to the test unit.



**Figure 11** Location of strain gauges used on the bottom deck reinforcing bars.



**Figure 12** Location of strain gauges on the top deck reinforcing bars and dowel bars.

## Load Protocols

The performance of UHPC waffle deck system, including the UHPC joints, was examined using six different tests and a single wheel truck load. Two different locations were chosen to apply the load along the centerline between the two girders: one was at the center of the deck panel and the other was at the center of the transverse joint between the deck panels. The critical locations for the tests were determined using a 3D finite element analysis model of the test specimen in ABAQUS software (ABAQUS, 2009). For each test location, a service load test, a fatigue test and an ultimate load test were conducted. The service and ultimate load tests were performed using monotonic increments of loads and these tests were paused during loading to the target values for visual inspection of any damage to the test system, including formation of cracks. For the fatigue test, the system was subjected to one million cycles at a constant frequency of 2 Hz. This test was paused twice during the tests and the same maximum load was applied in a quasi-static manner to examine any progressive damage to the system.

The specimen was load tested in the following order: 1) service load test of deck panel UWP2, 2) service load test of the panel-to-panel joint (transverse joint), 3) fatigue test of the panel-to-panel joint, 4) ultimate load test of the panel-to-panel joint, 5) fatigue test of panel UWP1, and 6) ultimate load test of panel UWP1. More details of each test and expected damage established from the finite element analysis are summarized in Table 4.

**Table 4** Sequence and details of the tests conducted on the Waffle deck system.

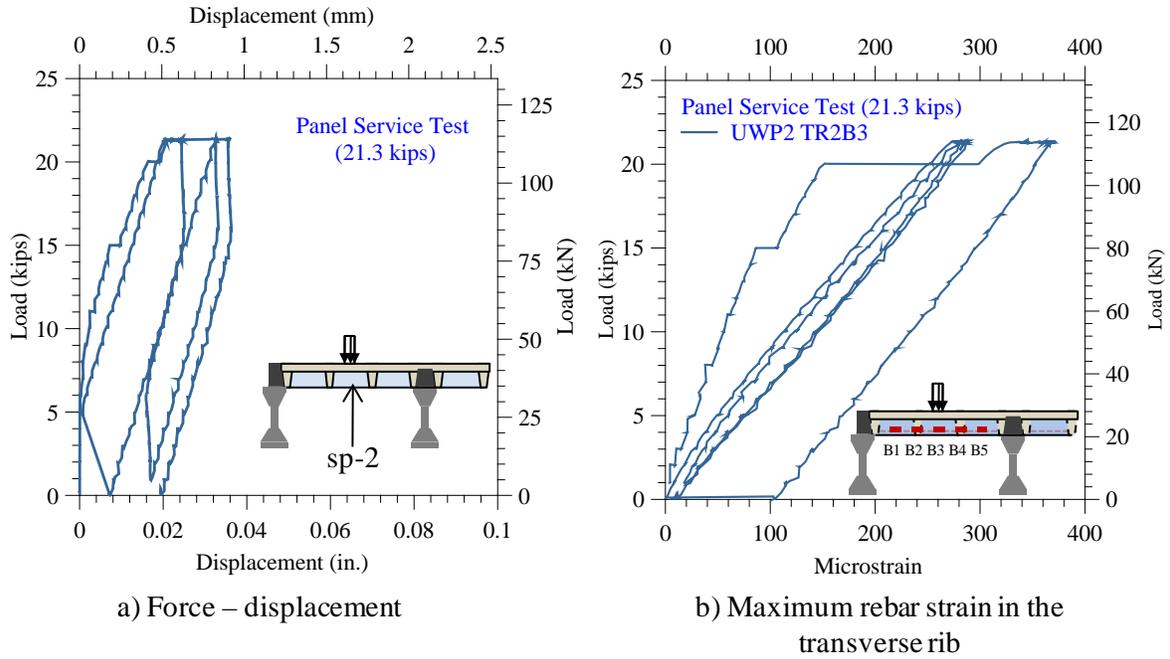
Test Number	Test Description	Location	Maximum Load	Expected Damage
1	Service load test panel-2	Center of the panel	$1.33^a \times 16\text{kips}$ = 21.3 kips	Micro cracking in ribs
2	Service load test on transverse joint	Center of the joint	$1.75^b \times 16\text{kips}$ = 28 kips	Micro cracking in joint
3	Fatigue test on the transverse Joint	Center of the joint	28 kips ( 1 million cycles)	No prediction was made
4	Ultimate load test of transverse joint	Center of the joint	48 kips	Visible flexural cracks (more than one) along the joint and transverse ribs
5	Fatigue test on the panel-1	Center of the panel	21.3 kips ( 1 million cycles)	No prediction was made
6	Ultimate load test of the panel	Center of the panel	40 kips	Several visible flexural cracks along transverse ribs

a, b –dynamic load allowance factors from AASHTO Table 3.6.2.1-1

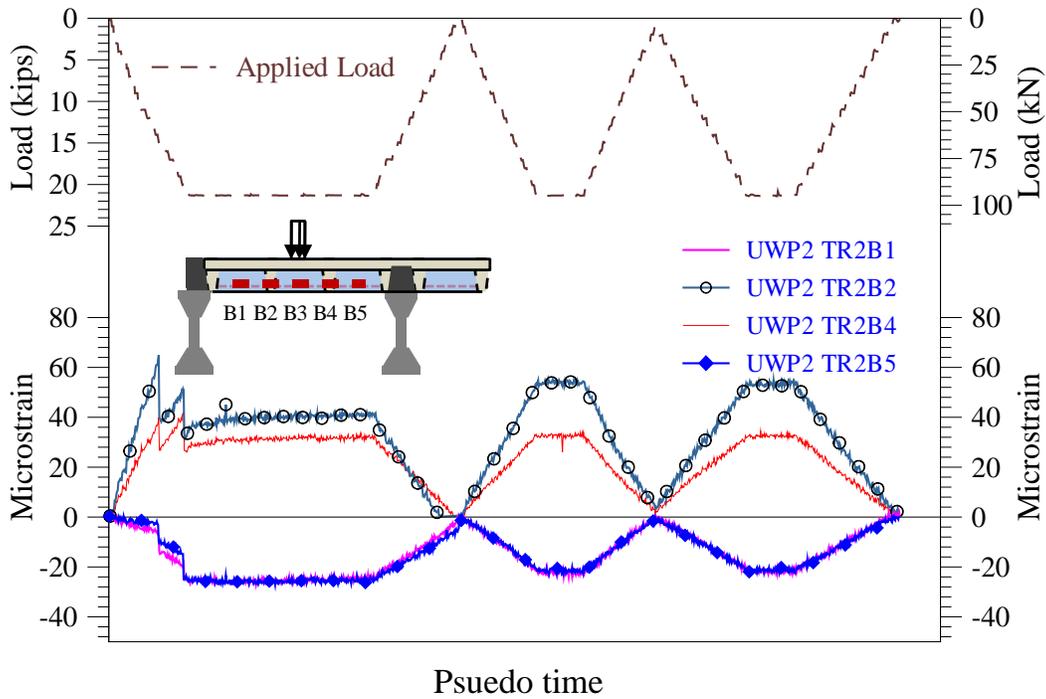
## **Panel Service Load Test (Test 1)**

As noted earlier, a 10 in. x 20 in. plate represented the dimensions of a wheel, and a maximum load of 21.3 kips was applied at the center of panel UWP2 to simulate the service load condition. This load was established using the AASHTO service truck wheel load of 16 kips with a 1.33 factor to account for the 33% load increase suggested to account for the wheel load impact from the moving loads. The 3D finite element analysis of the test setup using ABAQUS confirmed the most critical location being one wheel at the center of the panel than placing two wheels at off-centered positions. To ensure no strength or stiffness degradation would take place due to repeated loading, the panel was subjected to three load cycles at this load level. The load-deflection curve established at the center of the panel for this test is shown in Figure 13a. For this test, a nearly linear relationship was observed between the load and deflection, with the maximum recorded deflection during the first cycle being 0.02 inches. This deflection corresponds to  $L/4400$  ( $L$  = the span length between the girder), which is significantly less than the specified AASHTO limit of  $L/800$  recommended for the serviceability condition (Section 9.5.2) of continuous span bridges with pedestrian traffic. The AASHTO allowable serviceability displacement of  $L/800$  would lead to 0.11 inches for the tested system.

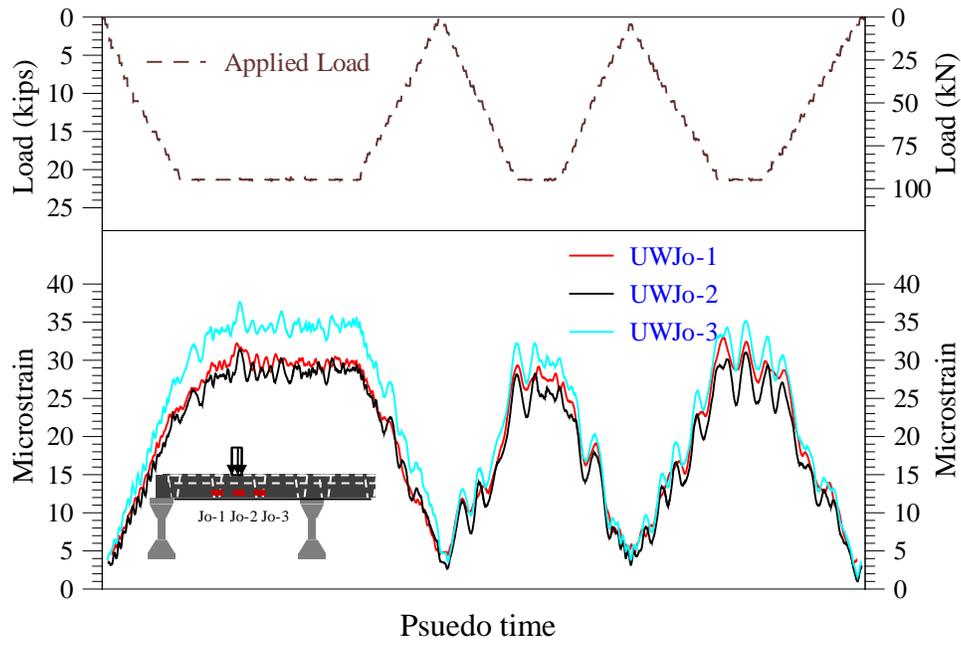
The peak recorded strain in the bottom reinforcement of the center rib running in the transverse direction is shown in Figure 13b, which reached a maximum strain of only  $375\mu\epsilon$  or 18% of the yield strain. The strain variations along the length of the bottom reinforcement in the transverse rib TR2 of panel UWP2 and the panel-to-panel joint are shown in Figure 14 and Figure 15, respectively. Strain in the joint reinforcement was  $40\mu\epsilon$ , indicating no damage to the joint region. A single crack having a width less than 0.002 in. was observed on the transverse rib under the load and is identified in Figure 16. In comparison to traditional normal concrete, it is important to realize that the material behavior of UHPC is quite dependent on the crack width. Hence, it may not be appropriate to use the AASHTO crack width provisions to qualify the serviceability and durability considerations for the behavior of UHPC structural members. Due to the lack of any specific UHPC bridge design serviceability criteria available in the literature, the crack width limits suggested for UHPC to control the fiber pullout criteria are used to comment on the implication of the crack developed in this test. Based on the AFGC 2002 recommendations, the fiber pull out and strength degradation in UHPC initiate when a crack width reaches a width of 0.0118 in. (0.3 mm) (see Figure 17). This limit is nearly more than 10 times the observed crack width during the service test, confirming that the overall behavior of the precast waffle deck system was outstanding. In addition, it is noted that the test results also confirmed that the system performance satisfied the deflection and crack width requirements recommended for the serviceability condition by AASHTO (AASHTO, 2007).



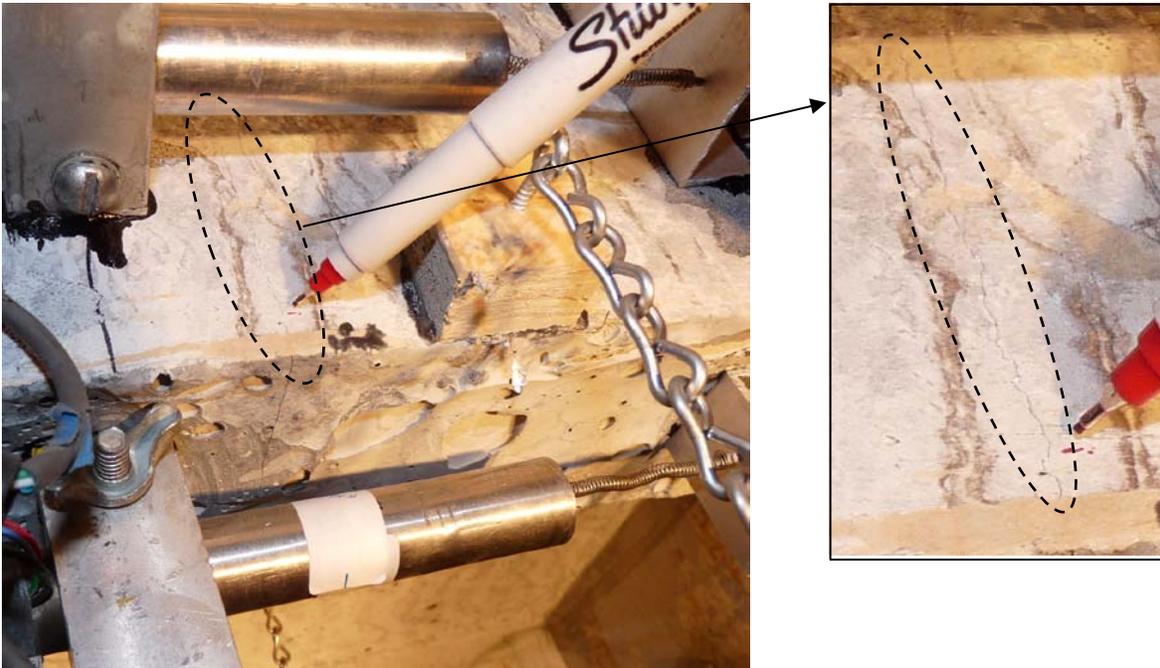
**Figure 13** Measured force-displacement response and peak rebar strain from gauge B3 at the center of the transverse rib TR2 of panel UWP2.



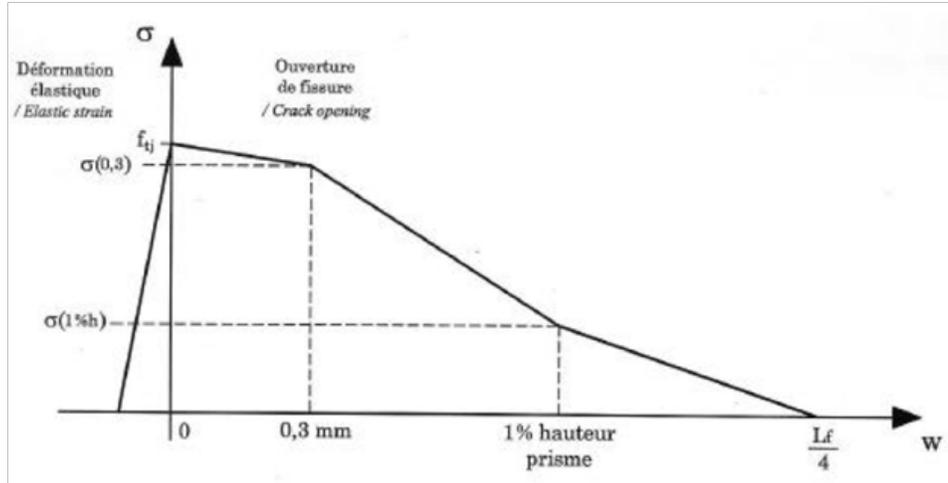
**Figure 14** Measured strains along the bottom reinforcement of the transverse rib TR2 of panel UWP2.



**Figure 15** Measured strains along a bottom reinforcement of the panel-to-panel joint.



**Figure 16** A hairline crack in the UWP2 panel transverse rib TR2 at 21.3 kips.



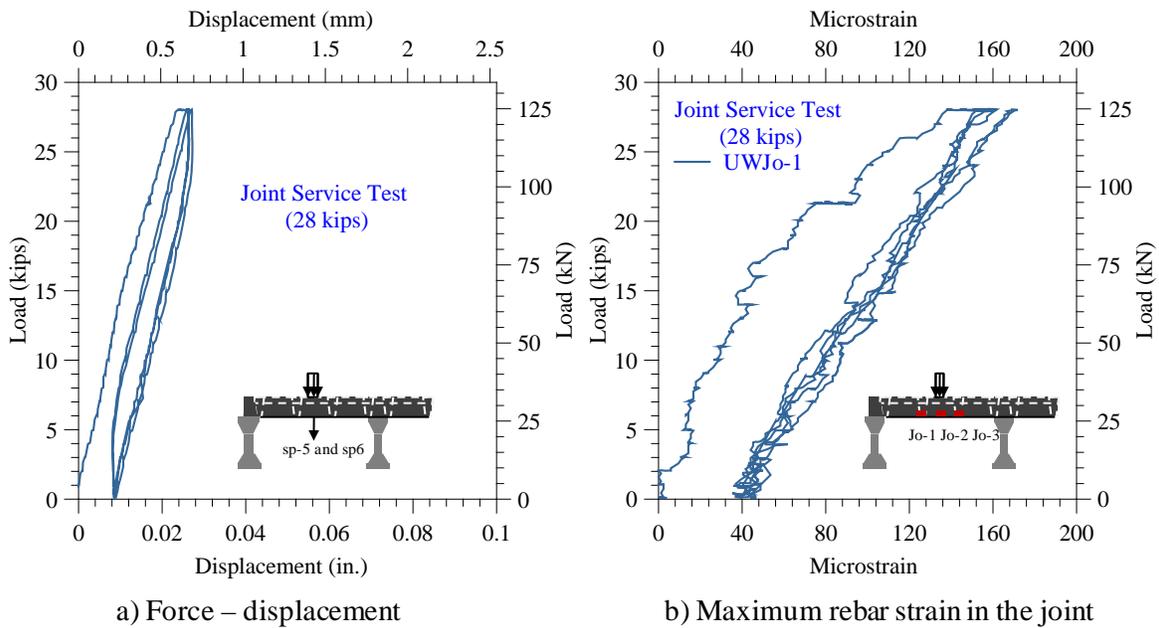
**Figure 17** A relationship proposed for the UHPC tensile strength variation as a function of crack width (AFGC 2002).

### **Joint Service Load Test (Test 2)**

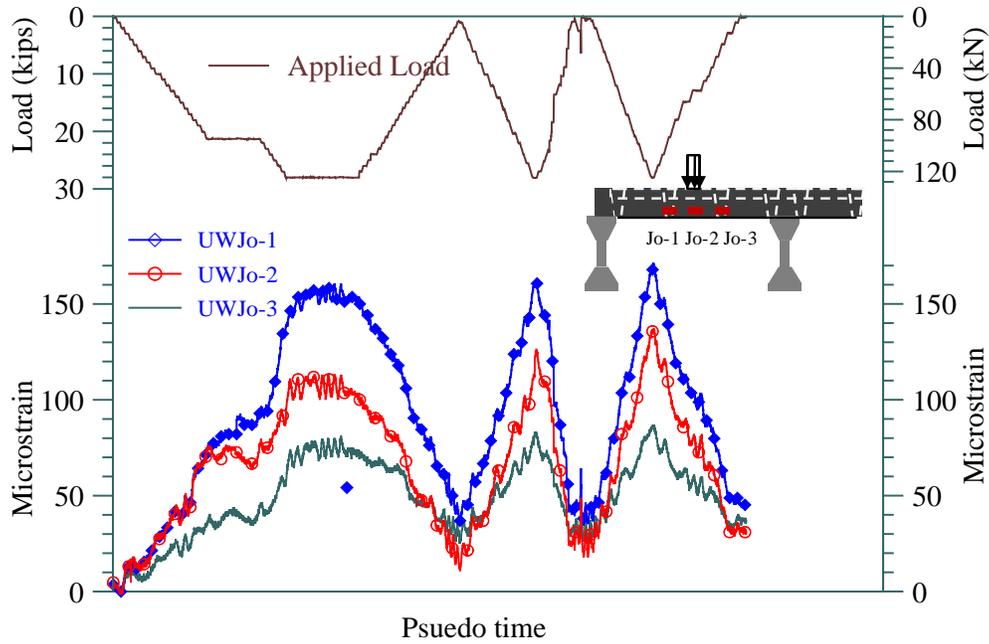
Similar to the panel service load test, the transverse panel-to-panel joint test was then conducted under the service loading condition. In this case, the maximum load of 28 kips was used, which represented the AASHTO service load of 16 kips for one wheel times 1.75 factor, which accounted for 75% increase in load to account for the wheel load impact on joints due to moving loads. Similar to the previous service load test, the critical location of the load was determined from the finite element analysis and the load was repeated three times to ensure the stability of the force-displacement response of the system. The applied load vs. the measured deflection at the center of the joint is shown in Figure 18a. Again, a linear response was obtained with a maximum deflection reaching only 0.022 in. during the first load cycle. This deflection corresponds to  $L/4000$ , which is 20% of the specified AASHTO limit of  $L/800$  (Section 9.5.2 in AASHTO (2007) for continuous spans with pedestrian traffic under the serviceability condition.

The load vs. strain plot for the gauge recorded the maximum strains and the strain variation along a bottom reinforcement in the joint are shown in Figure 18b and Figure 19, respectively. The peak recorded strain in the joint bottom reinforcement was  $170 \mu\epsilon$ , indicating significant reserve capacity of the joint. The strain variations obtained for the bottom reinforcement in the transverse rib TR2 of panels UWP2 and UWP1 are shown in Figure 20 and Figure 21, respectively. Both figures show comparable strain demands, indicating that the applied joint load was evenly distributed to both panels. Figure 22 shows the variation of strains at the center of the rib across the transverse ribs of panel-1, indicating their relative contribution. No cracking was observed at 21.3 kips load. However at the peak load of 28 kips, a single hairline crack having width less than 0.002 in. was observed on the transverse ribs forming the

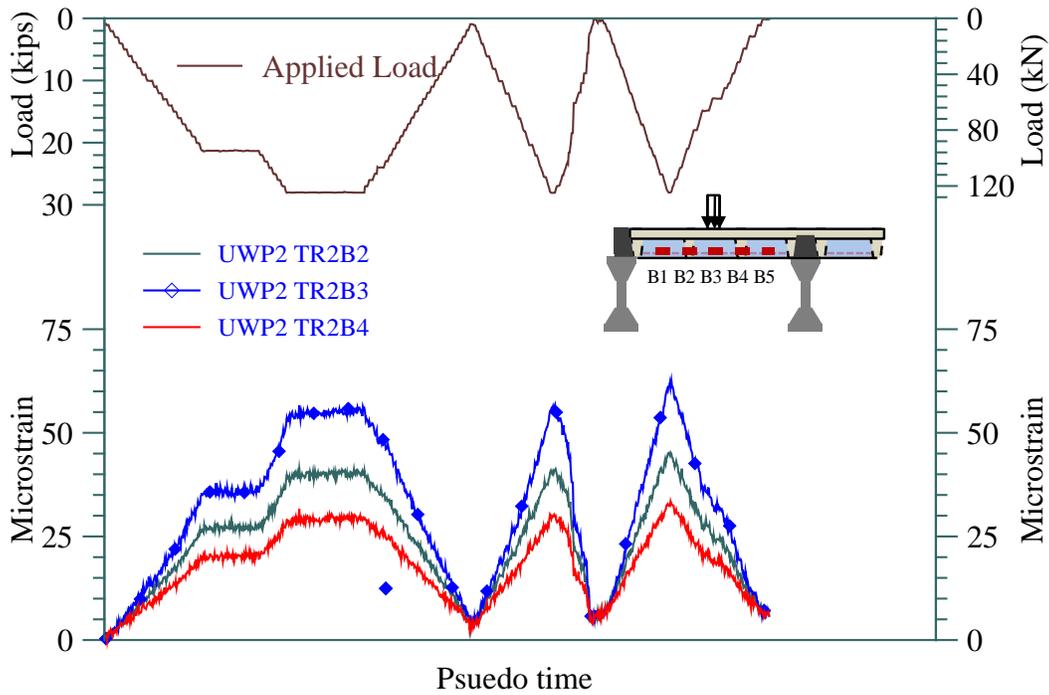
joint (see Figure 23). Given that this crack is significantly smaller than 0.0118 in. (0.3 mm) (see Figure 17) corresponding to initiation of fiber pull out and strength degradation of UHPC in tension, it is concluded that the overall behavior of the transverse joint subjected to service load was outstanding. The test results also indicated that the system performance satisfied the deflection and crack width requirements recommended for the serviceability condition by AASHTO (AASHTO, 2007).



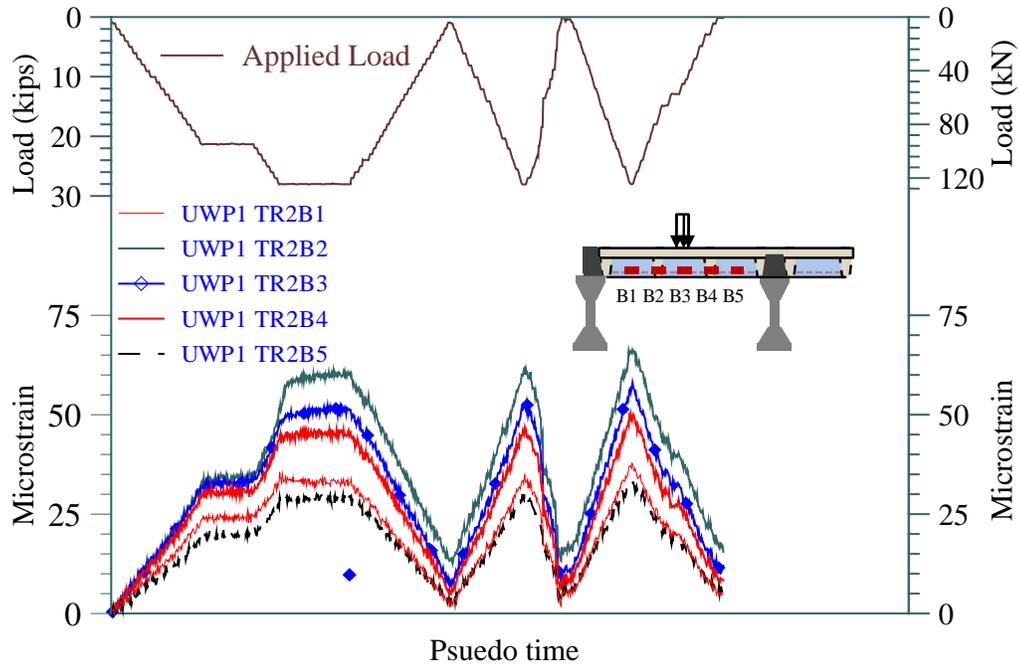
**Figure 18** Measured force-displacement response and peak rebar strain at the center of the joint at the service load.



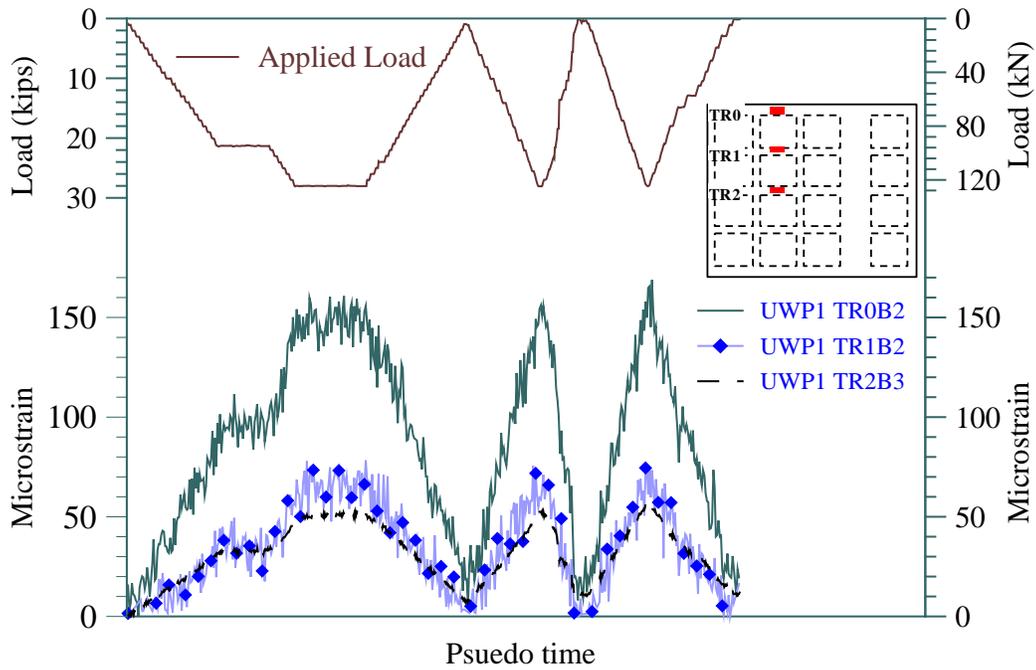
**Figure 19** Measured strains along the bottom reinforcement of the joint during the service load test.



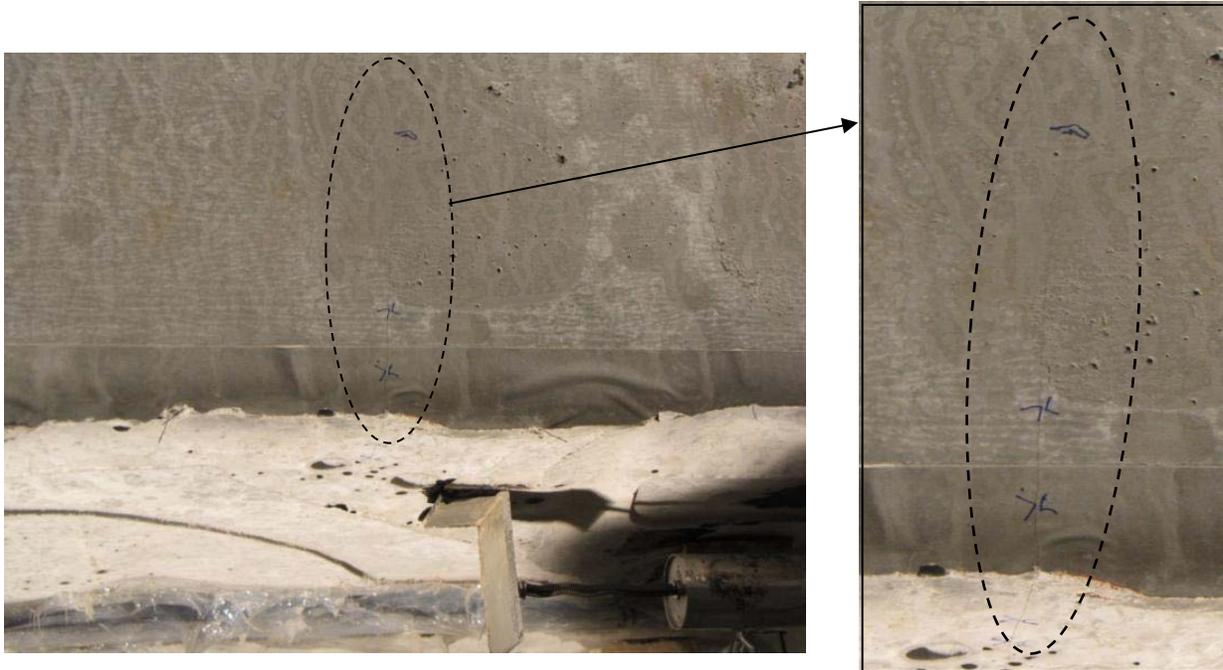
**Figure 20** Measured strains in the bottom reinforcement of the transverse rib (TR2) along the length of panel UWP2 at service load.



**Figure 21** Measured strains in the bottom reinforcement of the transverse rib (TR2) along the length of panel UWP1 at service load.



**Figure 22** Measured strains at the center of the panel across the transverse ribs of UWP1 at service load.



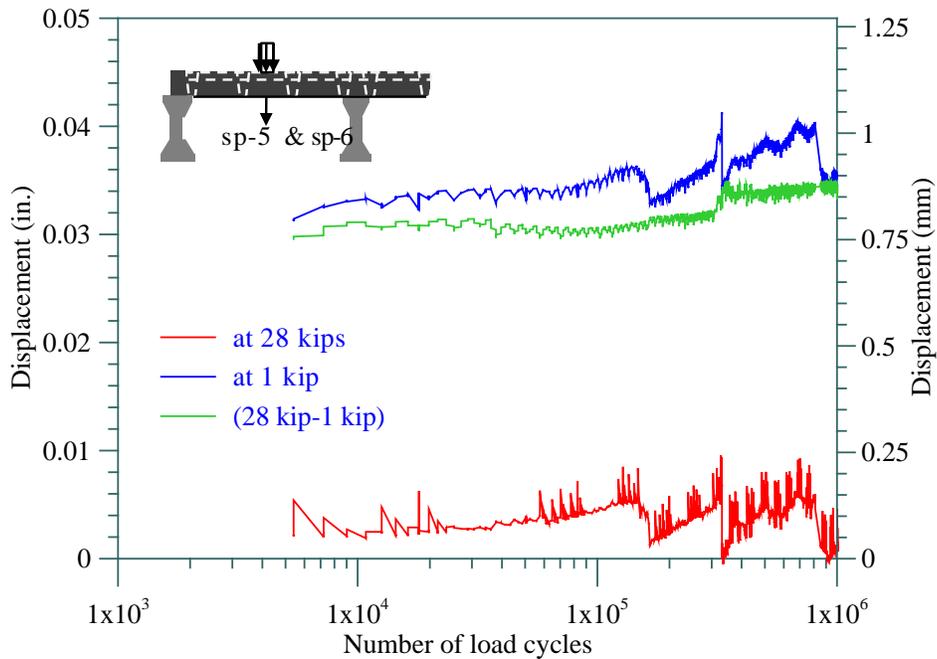
**Figure 23** A hairline crack formed at the center of underside of the transverse joint at 28 kips.

### **Joint Fatigue Load Test (Test3)**

The transverse joint between the waffle deck panels was subjected to one million cycles to test the joint for potential low amplitude fatigue damage. The load variation was computer controlled in a sinusoidal manner between 1 kip and 28 kips at a frequency of 2 Hz. In other words, the peak load of 28 kips was reached twice within a one-second interval. During the test that lasted for several days as well as at the end of fatigue test, the deck panels and the joint were monitored for formation of any new cracks. Except for those formed during the joint service load test, no further cracks developed during the joint fatigue test. The load, displacements and strain data obtained from selected gauges from the test were recorded continuously for 5 seconds at 20 Hz frequency at the end of every 1800 cycles (i.e., at every 15 min.). In addition, the fatigue test was paused and static joint load tests were conducted at the end of 168,000, 333,875 and 1 million cycles to determine the influence of fatigue damage on the joint and system behavior.

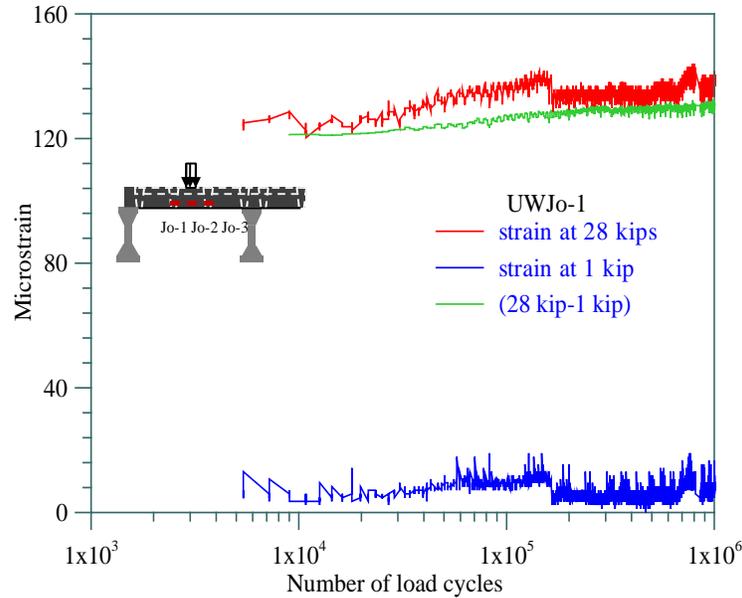
Based on the recorded data, the displacements recorded at the center of the joint at 28 kip and 1 kip are plotted as a function of the load cycle during the fatigue loading in Figure 24. It is apparent that that the gauge data experienced drift due to ambient condition and other reasons during the test. However, when the displacement corresponding to the load increment of 27 kips (i.e., 28 kips – 1 kip) was examined, it is clear that this displacement remained almost constant

throughout the test and the change in the displacement reading is largely due to noise observed at 1 kip. With the variation of the displacement being very small and limitations with the sensitivity of the string potentiometers, it is concluded that the UHPC did not experience any fatigue damage.

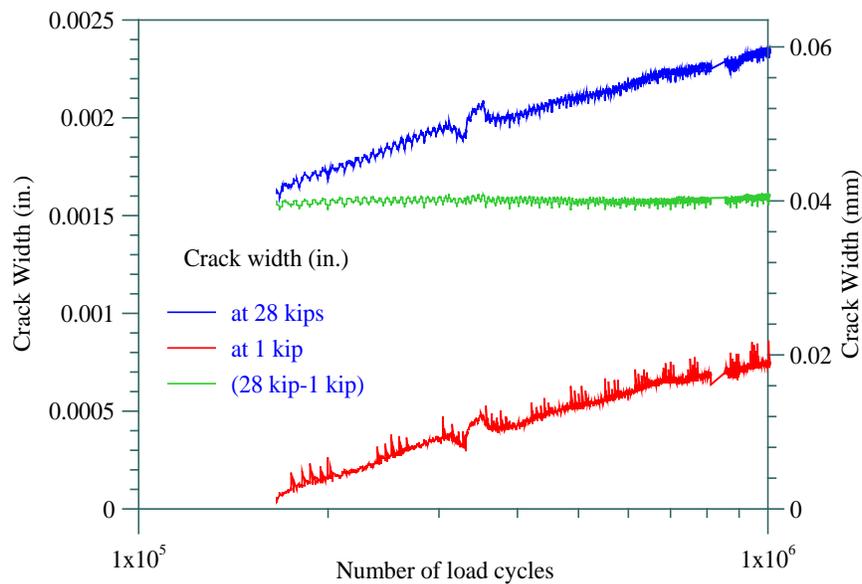


**Figure 24** The variation of the peak displacement at the center of the joint during the joint fatigue test.

Figure 25 shows the strains recorded by the gauge mounted to the joint transverse reinforcement located at the center of the joint as a function of the load cycle. Although the drifts in measured data are apparent, the change in strain remained almost constant at a value of  $135 \mu\epsilon$  as the load increased from 1 to 28 kips. This variation is comparable to the peak strain of  $170 \mu\epsilon$  recorded during the service load test. Except for the noise in the data, the crack width in the transverse joint was nearly constant over the entire fatigue test and is shown in Figure 26.



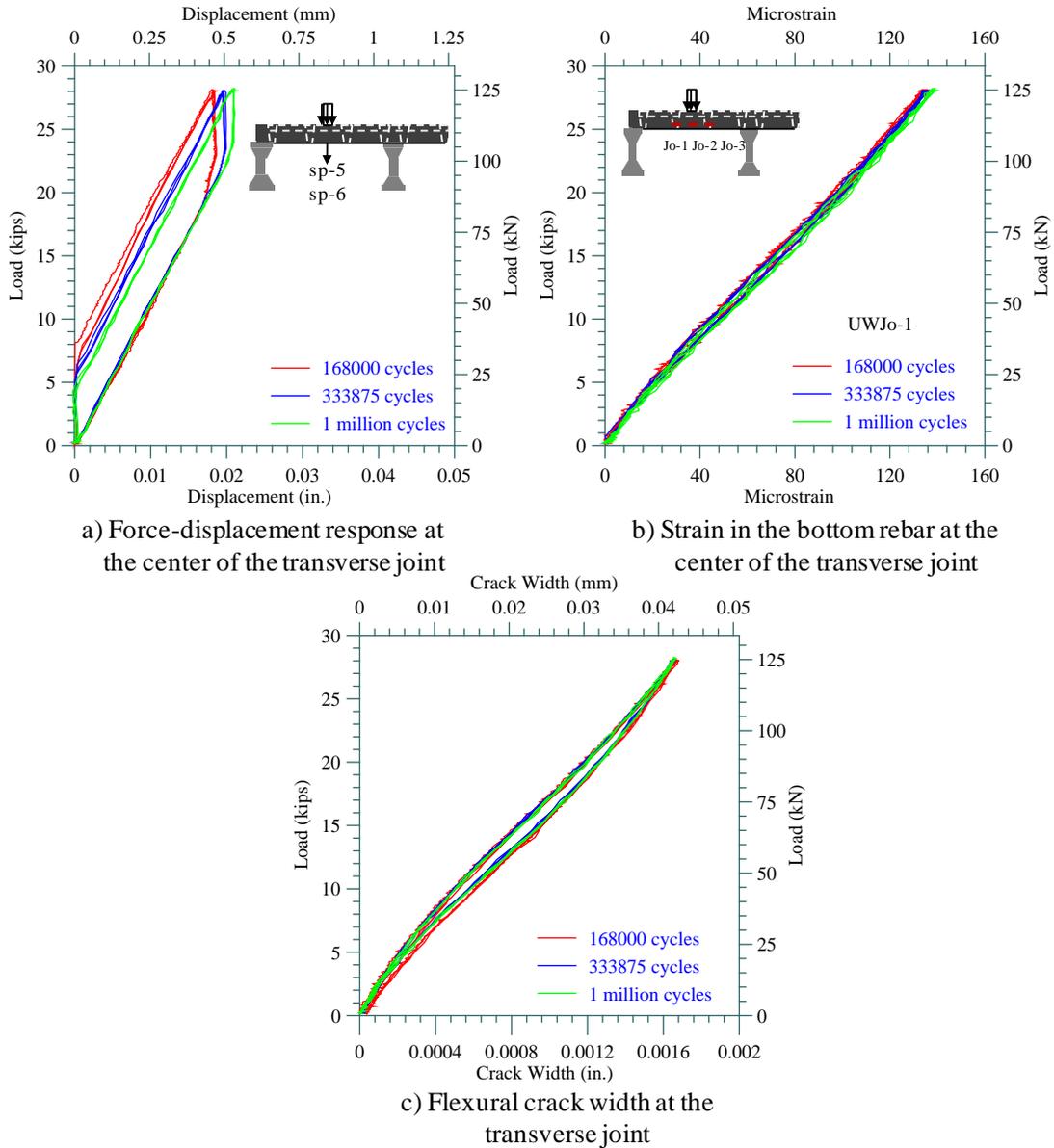
**Figure 25** The variation of the peak strain in the bottom joint transverse reinforcement during the joint fatigue test.



**Figure 26** The variation of the crack width in the transverse joint with number of load cycles.

For the static load tests performed at the end of 168,000, 333,875 and 1 million cycles, the load-displacement, peak strain in the bottom reinforcement in the joint, and the crack width in transverse ribs forming the joint during the intermediate static load tests are presented in Figure 27. The initial secant stiffness at the peak load of panel-to-panel joint at the end of 168,000, 333,875 and 1 million cycles of loading is 1166.7 kip/in, 1135.6 kip/in and 1139.2 kip/in respectively, which compares closely with each other and shows variations of less than

3%. It can be seen from these figures that the joint or the UHPC waffle deck system did not experience any significant fatigue damage after subjected to one million cycles of amplified serviced load.

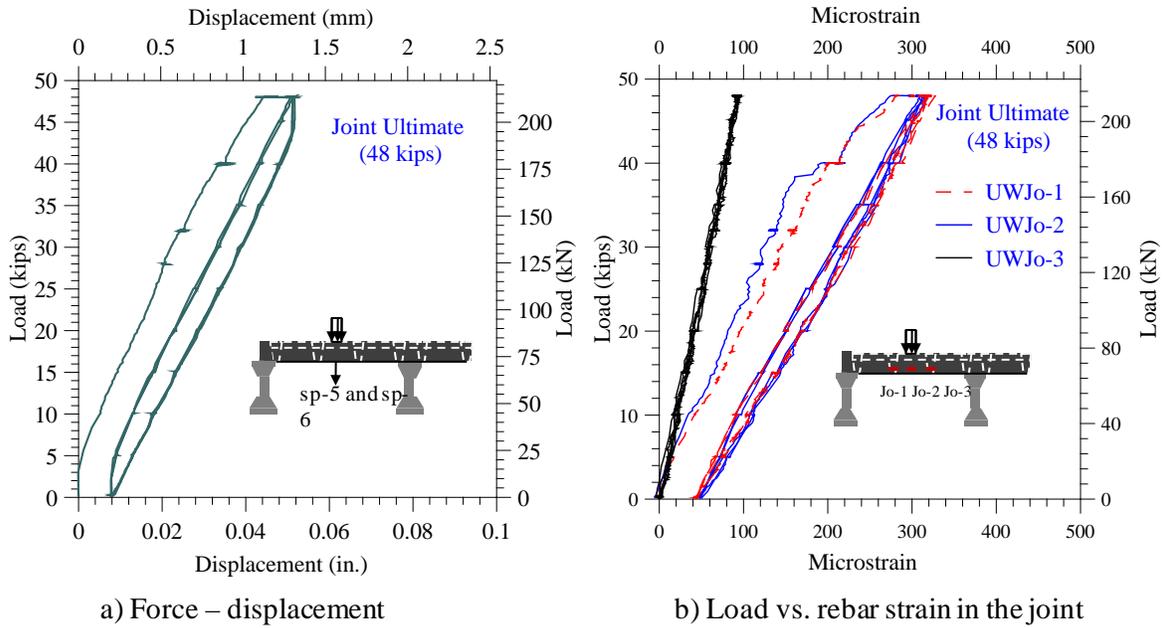


**Figure 27** Measured responses of the waffle deck system from the static service load tests conducted during the joint fatigue test.

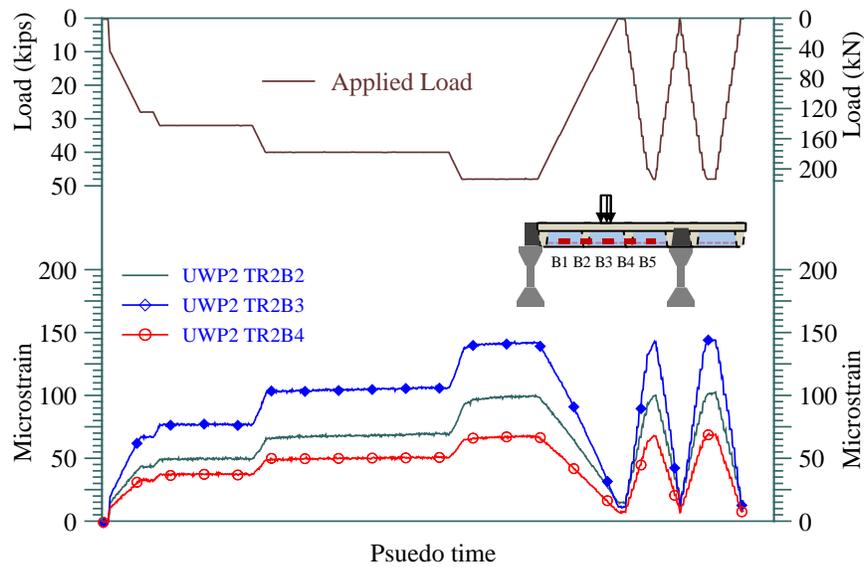
### **Joint Ultimate Load Test (Test4)**

The ultimate load test was carried out to investigate the adequacy of the transverse joint at the ultimate limit state. The load corresponding to this limit state was defined as a factor of the service wheel load of 16 kips without causing any significant damaging the joint so that the waffle deck system could be used to conduct the fatigue and ultimate load tests at the center of a panel. Using a load factor of three, the maximum load suitable for conducting the ultimate load test was defined as 48 kips. Similar to the service load test, the joint was subjected to three load cycles at this load level to ensure the stability of the force-displacement response of the system. The load-deflection curve established at the center of the joint for this test is shown in Figure 28a. The transverse joint exhibited a linear force-displacement response even for this test, with insignificant damage and a maximum deflection of 0.05 inches. This deflection corresponds to  $L/1760$ , which is 46% of the AASHTO serviceability limit of  $L/800$  (Section 9.5.2 in AASHTO 2007) for continuous span bridges with pedestrian traffic.

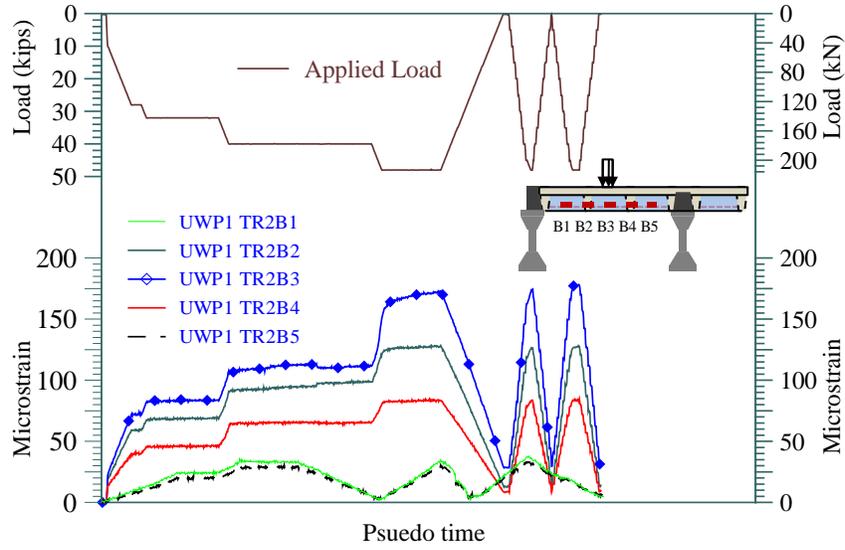
The strain variations along the bottom reinforcement in the joint as a function of the applied load are shown in Figure 28b. The peak strain in the joint region bottom reinforcement was  $330 \mu\epsilon$ , which is only about 15% of the yield strain of the reinforcement. The strain variations in the bottom reinforcement in the transverse rib TR2 of the panels UWP2 and UWP1 are shown in Figure 29 and Figure 30, respectively. Figure 31 shows the variation of the strain at the center of the rib across the transverse ribs of panel UWP1, indicating their relative participation in resisting the load. A series of hairline cracks were observed in the central region of the joint and are shown in Figure 32. The maximum crack width measured along the transverse ribs forming the joint was 0.003 inches, which can be seen in Figure 33.



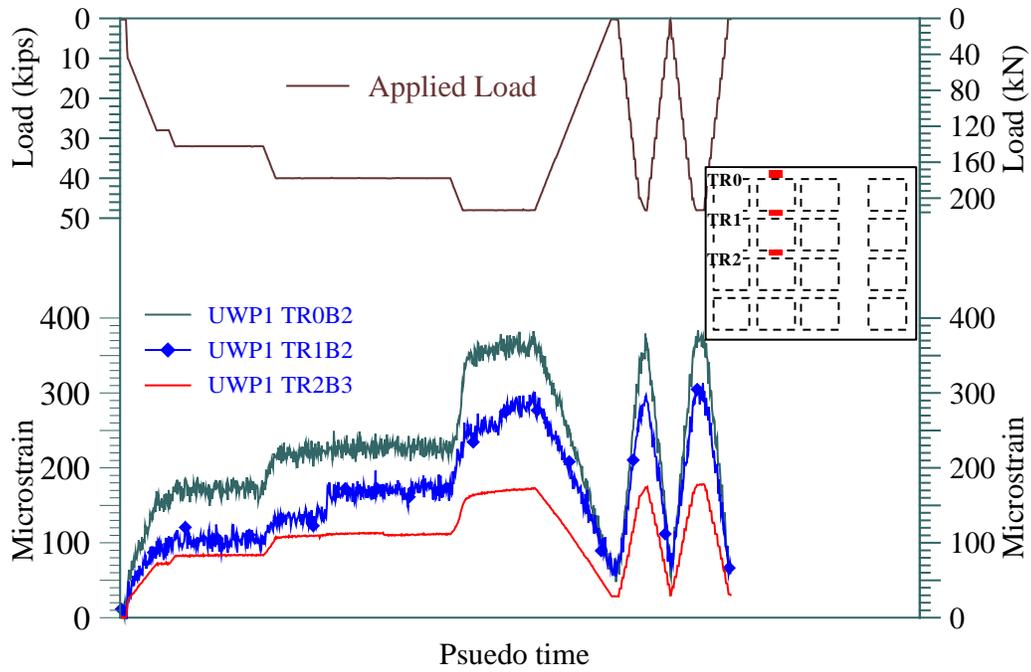
**Figure 28** Measured force-displacement response and peak rebar strain at the center of the joint at the ultimate load of 48 kips.



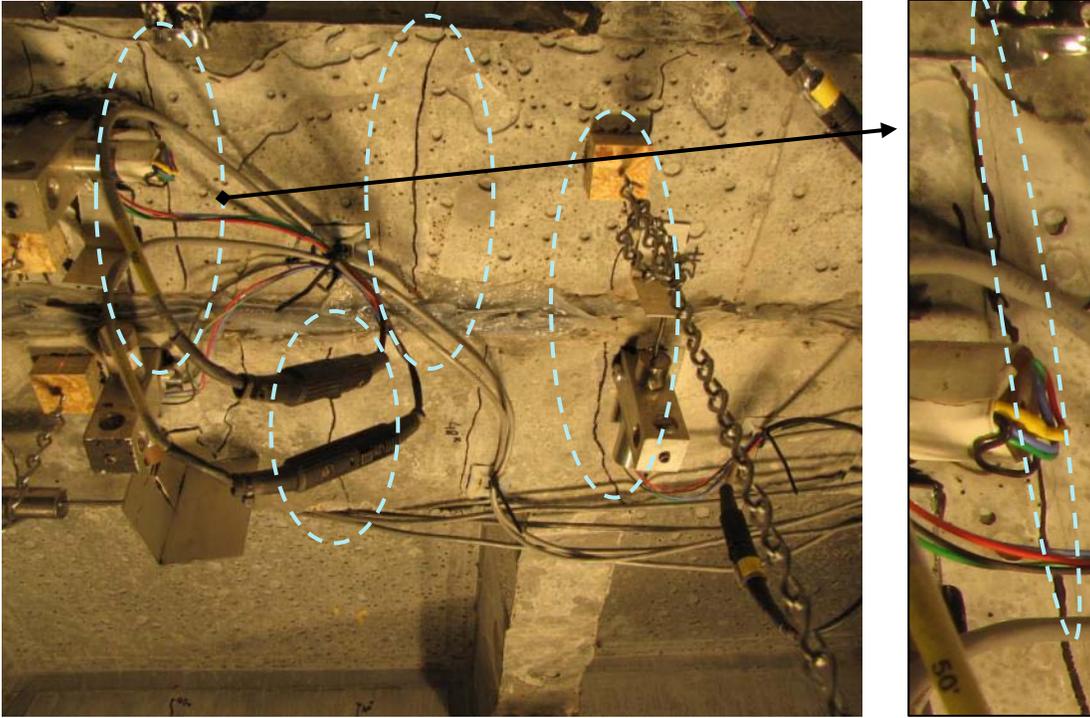
**Figure 29** Measured strains in the bottom reinforcement of transverse rib TR2 along the length of Panel UWP2 at the ultimate load.



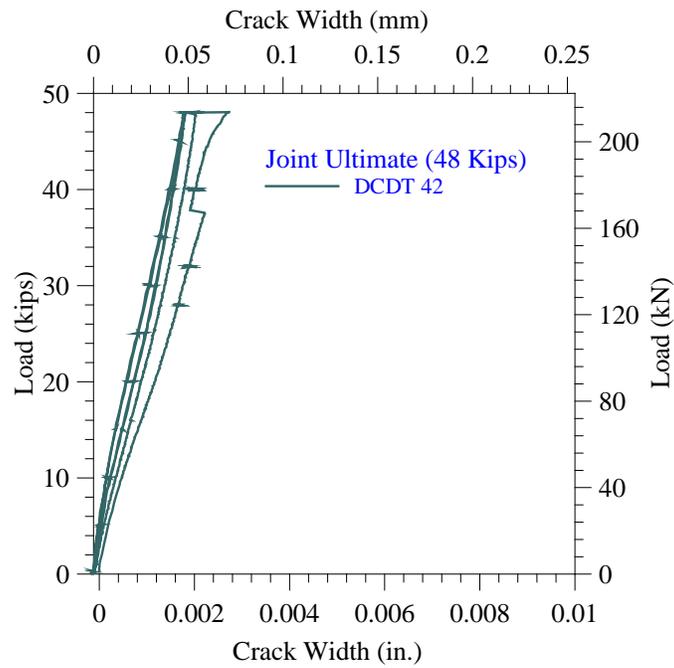
**Figure 30** Measured strains in the bottom reinforcement of transverse rib TR2 along the length of Panel UWP1 at the ultimate load.



**Figure 31** Measured strains at the center of the panel across the transverse ribs of panel UWP1 at joint ultimate load.



**Figure 32** Hairline cracks formed at the center of underside of the transverse joint at the ultimate load of 48 kips.

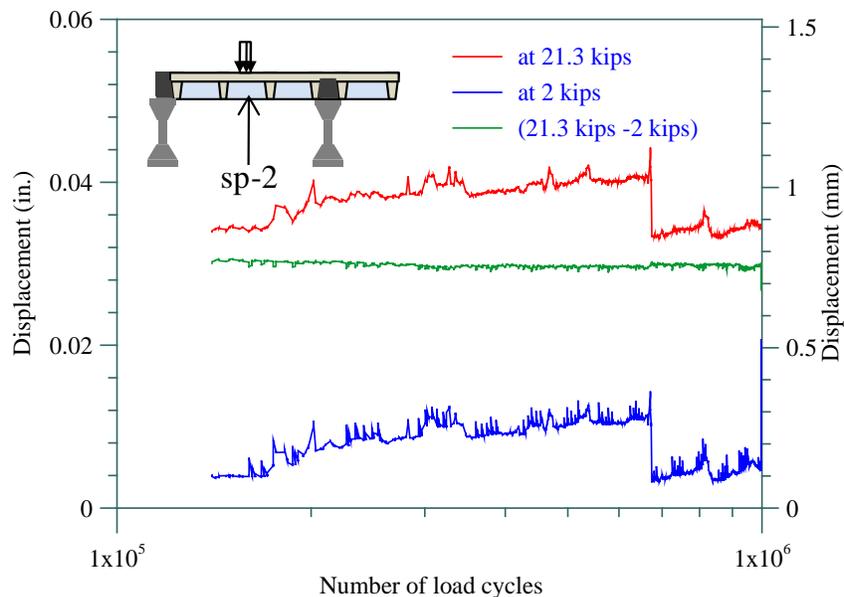


**Figure 33** The variation in the width of the most critical crack in the ribs forming the transverse joint.

## **Panel Fatigue Load Test (Test 5)**

As with the joint test, the waffle deck panel UWP1 was subjected to one million cycles to test this panel for potential low amplitude fatigue damage. The load variation was again computer controlled in a sinusoidal manner between 2 kip and 21.3 kips at a frequency of 2 Hz. During the test, the deck panels and the joint were examined periodically for formation of any new cracks. During the test, the load, displacements and strain data from selected gauges were recorded continuously for 5 seconds at 20 Hz frequency at the end of every 1800 cycles (i.e., at every 15 min.). In addition, the fatigue test was paused and static load tests were conducted at the end of 135000, 670000 and 1 million cycles with a maximum load of 21.3 kips to determine the influence of any fatigue damage on the panel and system behavior.

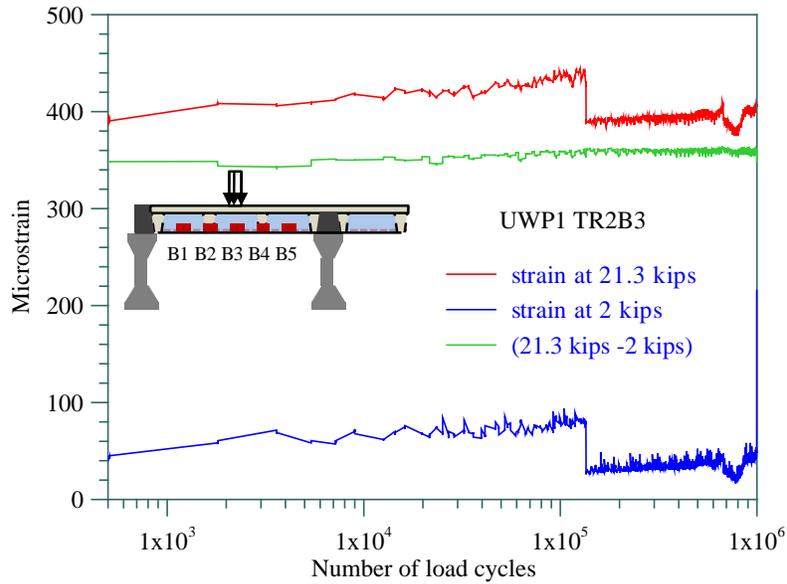
Based on the recorded data, the displacements recorded at the center of the panel UWP1 at 21.3 kips and 2 kips are plotted as a function of the load cycle during in Figure 34. It is apparent that the gauges experienced drifts due to ambient variations and that the data was influenced by high frequency noise. However, when the displacement corresponding to the load increment of 19.3 kips (i.e., 21.3 kips – 2 kips) was examined, it is clear that this displacement remained nearly constant throughout the test. Based on these observations, it is concluded that the UHPC panel did not experience any fatigue damage.



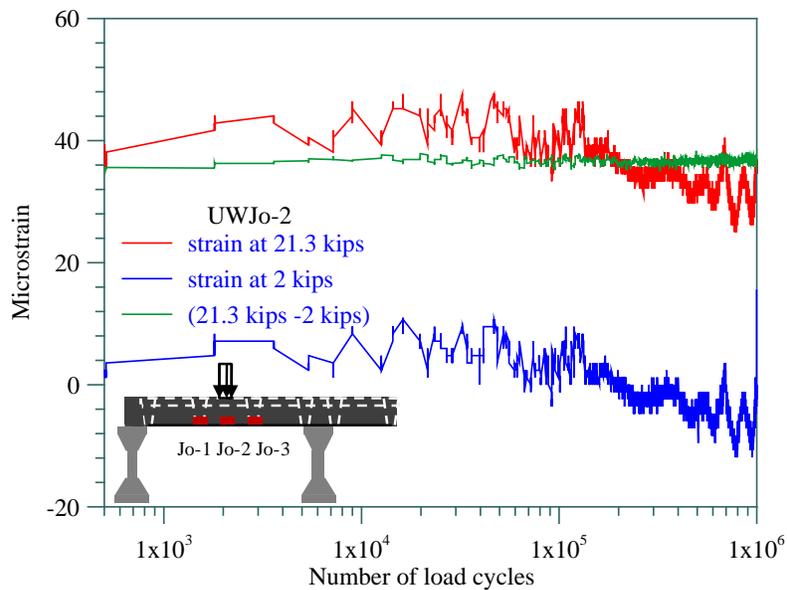
**Figure 34** The peak displacement variation at the center of Panel UWP1 during the joint fatigue test.

Figure 35a shows the strains recorded by the gauge mounted to the transverse rib reinforcement located at the center of the rib TR2 of panel UWP1 as a function of the load cycle. Although the drifts in measured data are apparent, the change in strain remained almost constant

at a value of  $360\mu\epsilon$  as the load increased from 2 to 21.3 kips. This variation is comparable to a strain of  $375\mu\epsilon$  recorded during the service load test of panel UWP2. Figure 35b shows the strains recorded by the gauge mounted to the joint transverse reinforcement located at the center of the joint as a function of the load cycle.



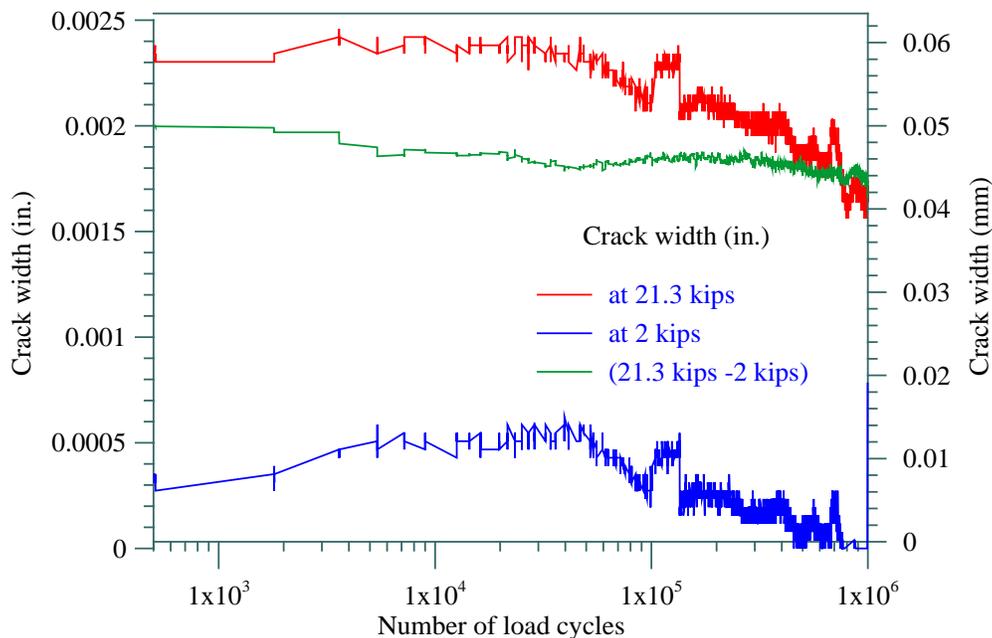
a) Peak rebar strain in the transverse rib



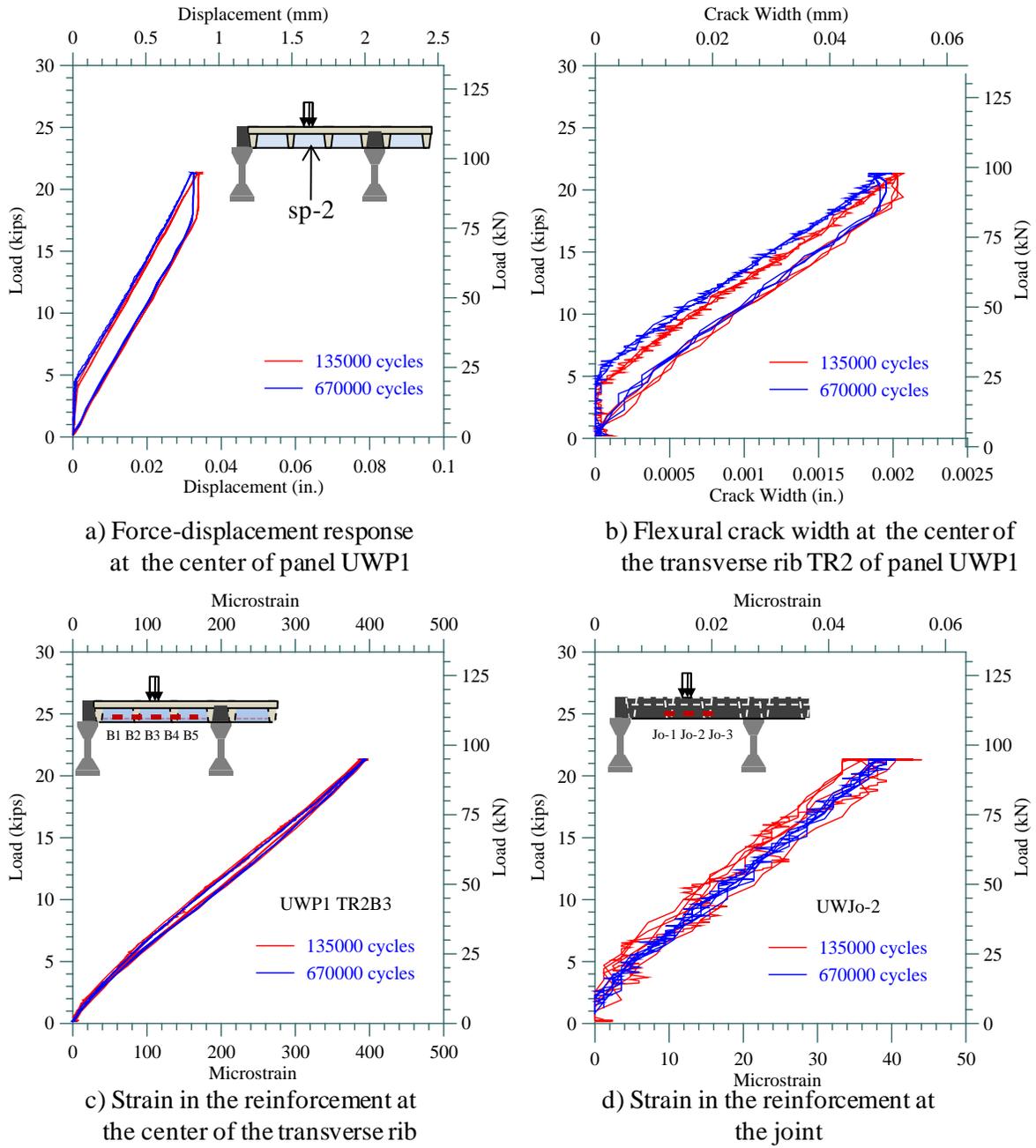
b) Peak rebar strain in the joint

**Figure 35** The peak strain variation in bottom deck reinforcement in the transverse rib of UWP1 and the joint during panel fatigue test.

During the fatigue test, no additional cracking in the panel was observed besides those formed during the service load test. The crack width at the bottom of the transverse rib was nearly constant over the entire fatigue test and is shown in Figure 36. This data varied between 0.0018 in. and 0.0023 in. or within a range of 0.0005 inches, which is close to the sensitivity of the LVDTs used to measure the crack width. For the static load tests performed at the end of 135000, 670000 and 1 million cycles, the load-displacement, peak strain in the bottom reinforcement in the transverse joint, and the crack width in transverse rib TR2 of panel UWP1 during the intermediate static load tests are presented in Figure 37. The initial stiffness of the panel at the end of 135000, 670000 and 1 million cycles of loading was 667.71 kip/in., 637.72 kip/in., and 653.34 kip/in., respectively. These values compare closely with each other and show variations within 5%. From these observations and Figure 37, it is clear that the joint or the UHPC waffle deck system did not experience any significant fatigue damage even after subjected to one million cycles at an amplified level of the service load.



**Figure 36** The crack width variation in transverse rib TR2 of panel UWP1 during panel fatigue test

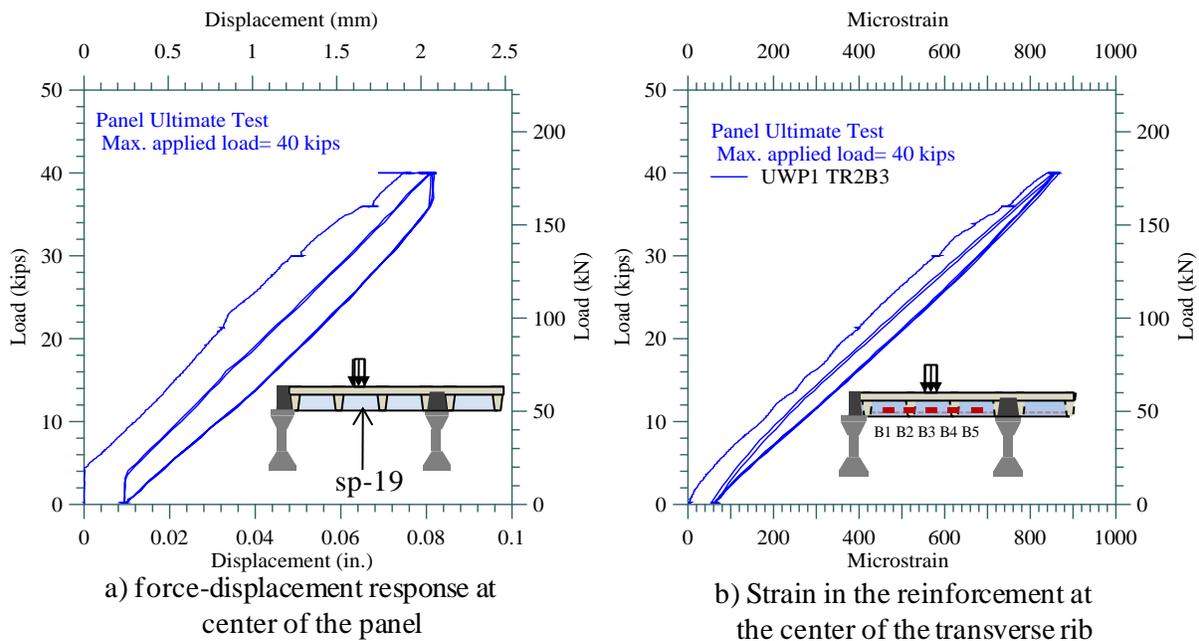


**Figure 37** Measured responses of the waffle deck system for static service load tests conducted during the panel fatigue test.

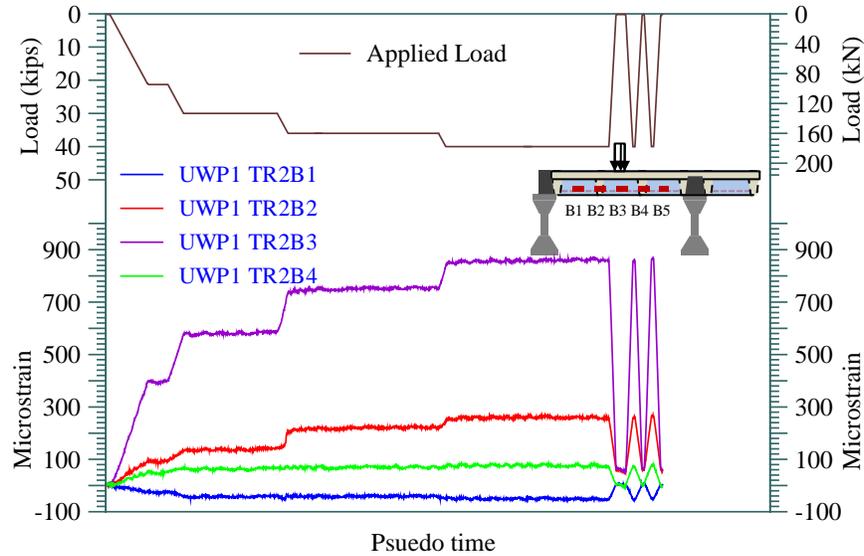
## Panel Ultimate Load Test (Test6)

The ultimate load test was carried out to investigate the adequacy of the waffle deck panel at the ultimate limit state. Similar to the joint ultimate load test, this limit state was defined as a factor of the service wheel load of 16 kips without causing any significant damage to the panel UWP1. A maximum load of 40 kips; equivalent to 2.5 times the service wheel load of 16 kips was applied at the center of panel UWP1.

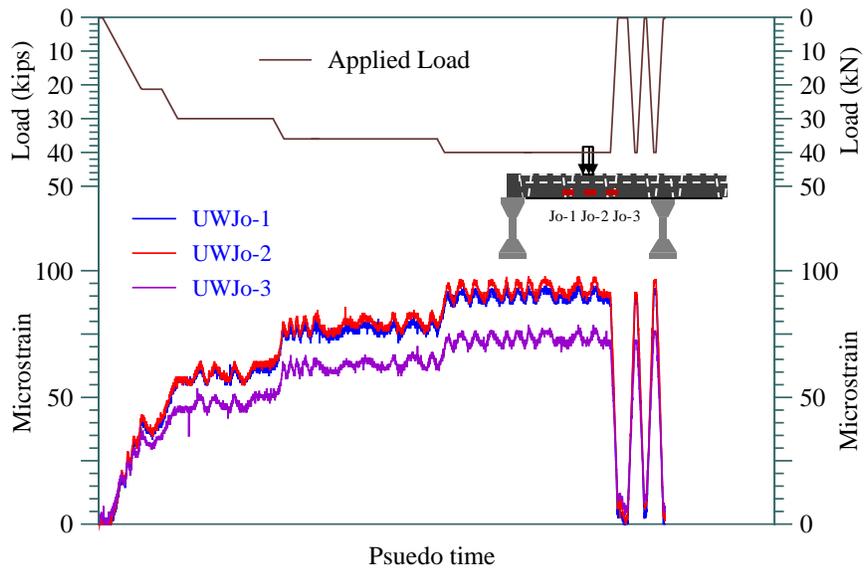
Similar to the previous service load tests, three load cycles at this load level were conducted to ensure the stability of the force-displacement response of the system. The load-deflection curve established at the center of panel UWP1 for this test is shown in Figure 38a. The panel exhibited a linear force-displacement behavior response with insignificant damage. A maximum deflection of 0.08 in. was measured at the center of panel UWP1. This deflection corresponds to  $L/1100$ , which is 73% of the AASHTO specified serviceability limit of  $L/800$  for continuous spans with pedestrian traffic.



**Figure 38** Measured force-displacement response and peak rebar strain at the center of the transverse rib of UWP1 at the ultimate load of 40 kips.



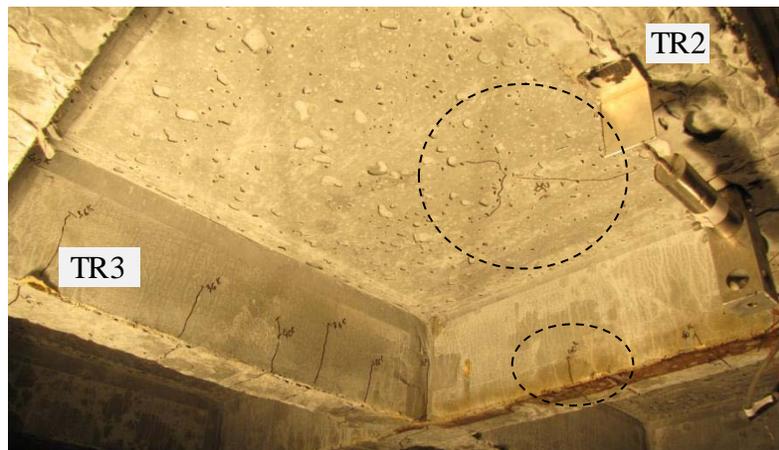
**Figure 39** Measured strains in the bottom reinforcement of the transverse rib along the length of UWP1 at during the ultimate load test.



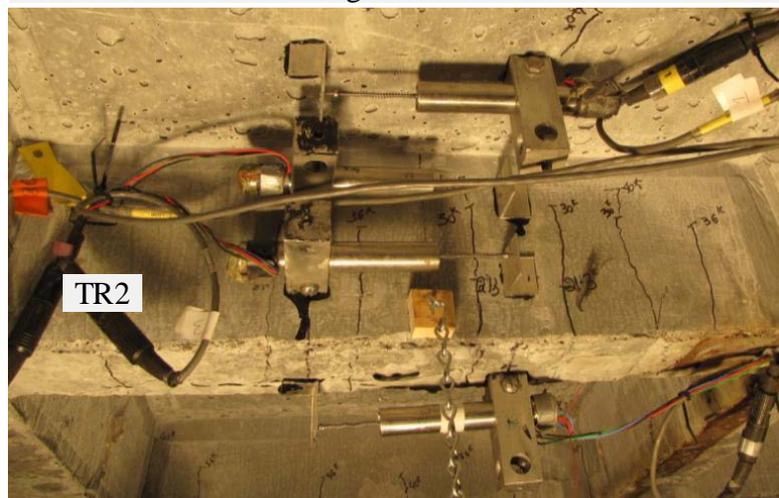
**Figure 40** Measured strains in the bottom reinforcement of the joint along the joint length during the ultimate load test.

The variation of the most critical strain in the bottom reinforcement of the transverse rib TR2 of panel UWP1 as a function of the applied load is shown in Figure 38b. The peak strain in the bottom reinforcement of transverse rib TR2 was only  $880 \mu\epsilon$ , which is only about 43% of the yield strain of the reinforcement. The strain variations in the bottom reinforcement placed in the transverse rib TR2 of panel UWP1 and the joint are shown in Figure 39 and Figure 40,

respectively. Three to four hairline cracks were observed on both transverse ribs (TR1, TR2 and TR3) and longitudinal ribs (LR1 and LR2) of panel UWP1 (see Figure 41). A hairline crack was seen on the bottom surface of UWP1 (between ribs TR2 and TR3) at the peak load (see Figure 41a). The maximum crack width measured along the transverse rib TR2 in UWP1 was 0.008 inches and its variation with the applied load is shown in Figure 42. Figure 43 shows the strain demand on the dowel bar in the panel-to-girder joint during the panel ultimate load test. It is clear that the dowel bars were engaged in load transfer when 35 kips load was applied at the center of the panel.

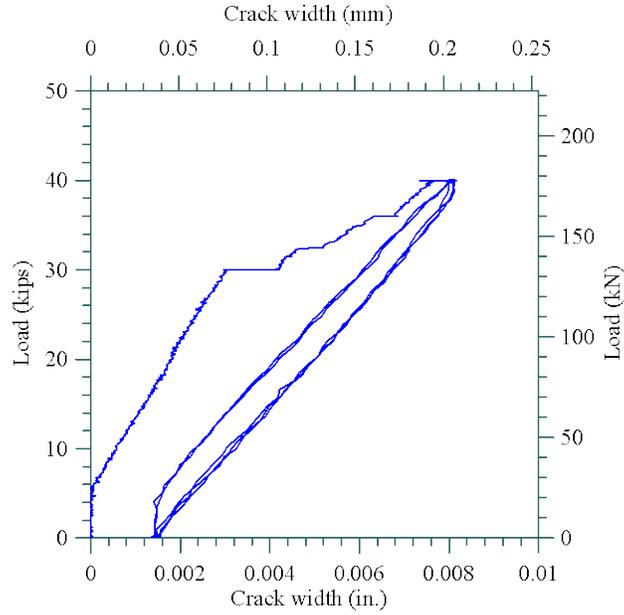


a) Cracking on the bottom surface of the slab and on a longitudinal rib

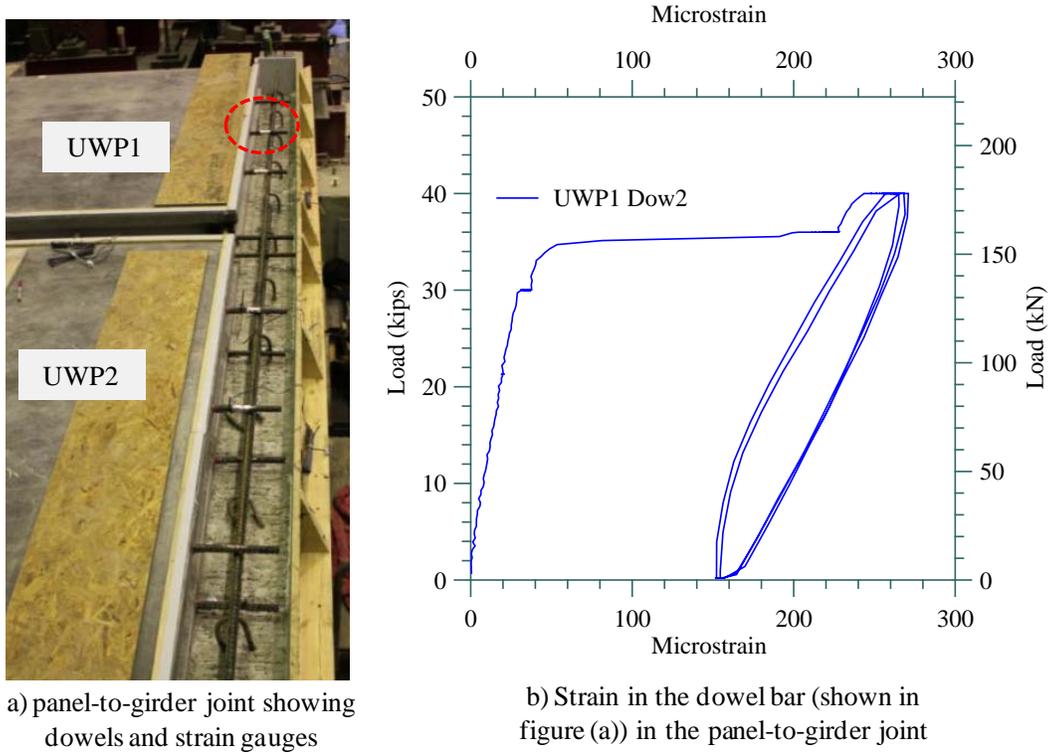


b) Cracking along transverse rib TR2 of UWP1

**Figure 41** Hairline cracks developed on panel UWP1 at an ultimate load of 40 kips



**Figure 42** Measured crack width in transverse rib TR2 of UWP1 at during the ultimate load test.



a) panel-to-girder joint showing dowels and strain gauges

b) Strain in the dowel bar (shown in figure (a)) in the panel-to-girder joint

**Figure 43** Strain variations in a dowel bar placed in the panel-to-girder joint during the panel ultimate load test.

# Summary and Conclusions

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As the first part of a study that is aimed for the first application of the full depth UHPC waffle deck panels in the field, an experimental investigation on a UHPC waffle deck panel system consisted of two panels was conducted at Iowa State University to examine the structural performance of the UHPC waffle deck, critical connections and system performance. A total of six tests were conducted and the key results obtained from the different tests, indicating that the overall performance of the system was satisfactory under the service, fatigue and ultimate load conditions are summarized below.

## **Panel Service Test (Test 1)**

- Load applied: 16 kips x 1.33 (33% IM factor) = 21.3 kips
- Maximum measured panel displacement: 0.03 inches (< 0.11 in. of allowable deck displacement at the service load specified by AASHTO, section 9.5.2)
- Maximum measured strain in panel bottom reinforcement =  $375 \times 10^{-6}$
- Maximum measured strain in joint bottom transverse reinforcement =  $40 \times 10^{-6}$
- Measured crack width in the transverse rib < 0.002 in. (• 0.017 inches, the allowable crack width by AASHTO, 2007; • 0.0118 inches of crack width expected for the fiber pullout (AFGC 2002))
- No cracks developed in the joint region.

## **Joint Service Test (Test 2)**

- Load applied: 16 kips x 1.75 (75% IM factor) = 28 kips
- Maximum measured panel displacement : 0.022 inches (< 0.11 in., allowable deck displacement at service load by AASHTO, section 9.5.2)
- Maximum measured strain in panel bottom reinforcement:  $160 \times 10^{-6}$
- Maximum measured strain in joint bottom transverse reinforcement:  $175 \times 10^{-6}$
- Measured crack width in the transverse ribs forming the joint < 0.002 in. (• 0.017 inches, the allowable crack width by AASHTO, 2007; • 0.0118 inches of crack width expected for the fiber pullout (AFGC 2002))

## **Joint Fatigue Test (Test 3)**

- Load applied = 16 kips x 1.75 (75% IM factor) = 28 kips
- Number of load cycles = 1 million cycles at 2 Hz frequency.

- Maximum measured panel displacement = 0.024 inches (< 0.11 in., allowable deck displacement at service load by AASHTO, Section 9.5.2)
- Maximum measured strain in panel bottom reinforcement: “not measured”
- Maximum measured strain in joint bottom transverse reinforcement:  $150 \times 10^{-6}$
- Measured crack width in the transverse ribs forming the joint = 0.0017 in. (• 0.017 inches, the allowable crack width by AASHTO, 2007; • 0.0118 inches of crack width expected for the fiber pullout (AFGC 2002))
- No fatigue damage occurred to the joint or the panels.

#### **Joint Ultimate Test (Test 4)**

- Load applied =  $3.0 \times 16$  kips = 48 kips
- Maximum measured panel displacement = 0.052 inches (< 0.11 in., allowable deck displacement at service load by AASHTO, section 9.5.2)
- Maximum measured strain in panel bottom reinforcement =  $360 \times 10^{-6}$
- Maximum measured strain in joint bottom transverse reinforcement:  $325 \times 10^{-6}$
- Crack width in the transverse ribs forming the joint = 0.003 in. (• 0.017 inches, the allowable crack width by AASHTO, 2007; • 0.0118 inches of crack width expected for the fiber pullout (AFGC 2002))
- Multiple cracks were observed in the transverse and longitudinal ribs adjacent the joint.

#### **Panel Fatigue Test (Test 5)**

- Load applied:  $16$  kips  $\times$  1.33 (33% IM factor) = 21.3 kips
- Number of load cycles: 1 million cycles at 2 Hz frequency.
- Maximum measured panel displacement : 0.039 inches (< 0.11 in., allowable deck displacement at service load by AASHTO, section 9.5.2)
- Maximum measured strain in panel bottom reinforcement:  $450 \times 10^{-6}$
- Maximum measured strain in joint bottom transverse reinforcement:  $150 \times 10^{-6}$
- Measured crack width in the transverse ribs of the panel < 0.0023 in. ( • 0.017 inches, the allowable crack width by AASHTO, 2007• 0.0118 inches of crack width expected for the fiber pullout (AFGC 2002))
- No fatigue loading damage observed to the panel and joint

#### **Panel Ultimate Test (Test 6)**

- Load applied =  $2.5 \times 16$  kips = 40 kips

- Maximum measured panel displacement = 0.08 inches (< 0.11 in., allowable deck displacement at service load by AASHTO, section 9.5.2)
- Maximum measured strain in panel bottom reinforcement:  $880 \times 10^{-6}$
- Maximum measured strain in joint bottom transverse reinforcement:  $100 \times 10^{-6}$
- Crack width in the transverse ribs forming the joint = 0.008 in. ( • 0.017 inches, the allowable crack width by AASHTO, 2007; • 0.0118 inches of crack width expected for the fiber pullout (AFGC 2002))
- Multiple cracks in the transverse and longitudinal ribs of the panel.

## **Conclusions**

Based on the experimental testing of the UHPC waffle deck system under service, ultimate and fatigue load conditions, the following conclusions are drawn for the prototype bridge system:

- Overall system behavior of the UHPC waffle deck bridge system would be satisfactory.
- The UHPC waffle panel or the joints are not expected to experience any fatigue damage under service loads.
- Displacements of the bridge deck under service conditions will be much smaller than the AASHTO specified allowable limits.
- The provided reinforcement and the use of wet UHPC infill for the joints will be satisfactory.
- Expect hairline cracks to form in the prototype bridge on the underside of the deck under service conditions.
- Crack widths will be negligibly small and are not expected to widen due to repeated loading under the most critical service conditions
- Larger cracks may form if the boundary conditions of the deck are altered from what was used for the test setup (e.g., by providing rigid connections between the deck and abutments).
- Dowel bars attached to the sides of the panels to form a positive connection with an interior girder experienced stresses in the order of 3 to 8 ksi and these bars should be included in the prototype bridge.

## **Recommendations**

In light of the conclusions established from the study, the proposed waffle deck panel can be used in the Wapello County prototype bridge, provided:

- Connection reinforcement matches or marginally exceeds those provided in the test unit;  
and
- Moment demands on the slab are kept below those induced during the tests for various limit states.

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