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Laboratory testing of Ultra High Performance Concrete deck joints for use in accelerated bridge construction

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Laboratory testing of Ultra High Performance Concrete deck joints for use in accelerated bridge construction

by

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ABSTRACT

Accelerated bridge construction is one rapid renewal technique being investigated to address the needs of the United States' aging infrastructure under the Second Strategic Highway Research Program (SHRP 2). SHRP 2 Project R04 aimed to develop standards and codes for accelerated bridge construction through the construction of a demonstration bridge. Several design details were important the rapid renewal aspect of the demonstration bridge, but the Ultra High Performance Concrete (UHPC) transverse full-depth deck joint over the pier was the focus of the laboratory testing due to its significance in the negative moment region tensile zone. Three suites of laboratory tests were conducted to evaluate the UHPC deck joints used in the demonstration bridge. Abrasion testing was completed to assess the abrasion resistance of the cast-in-place deck joints with respect to anticipated grinding operations, a constructability test was carried out to assess the placement procedure and feasibility of the longitudinal and transverse UHPC joint intersection detail, and strength and serviceability testing was completed to quantify the cracking moment and ultimate moment capacity of the transverse module-to-module connection detail over the bridge pier. Through this testing regiment, recommendations were developed for the demonstration bridge regarding construction and performance of the UHPC deck joints.

CHAPTER 1. INTRODUCTION

Problem Statement

An aging infrastructure has many of the United States' highway bridges currently in need of repair or replacement. Increases in traffic are placing excess stress on highway systems and economic constraints are limiting their renewal. Technologies and solutions must be developed that rapidly and systematically produce long-lasting highway bridges in a way that presents minimal disruption to the public (Transportation Research Board 2011). Currently, work is being done developing accelerated bridge construction (ABC) standards and codes to implement as a method of rapid renewal and address these problems as a part of the Second Strategic Highway Research Program (SHRP 2). To establish these new standards and code provisions for ABC's use across the country, a demonstration bridge is being designed, constructed, and tested in Pottawattamie County Iowa. The demonstration bridge for the SHRP 2 rapid renewal project is to be located on U.S. Highway 6 near Council Bluffs. The design of the three-span precast modular demonstration bridge has been developed using various details from multiple other ABC projects across the country. The goal of this complete ABC design for rapid renewal is to reduce the estimated 6-month road closure for typical construction to a 14-day road closure.

The evaluation of promising technologies selected for this demonstration project includes the simultaneous laboratory testing of specific elements or details that are deemed critical to the speed of construction and service life of the demonstration bridge. The unique design of the transverse deck joint over the bridge piers is one such instance in this demonstration bridge. Because the transverse deck joint utilizes an emerging innovative material in Ultra High Performance Concrete (UHPC), and the UHPC joint is located in the negative moment region tensile zone, this detail is critical to bridge performance. The UHPC deck joints have been the focus of the laboratory testing.

Research Goals and Objectives

The main goal of this research is to evaluate the implementation of UHPC deck joints in a typical ABC demonstration bridge project. There are several different testing needs for the UHPC joints that have been identified during the design of the demonstration bridge. While a few of these needs are being addressed by researchers around the country, some aspects of the demonstration bridge were looked at through further lab testing to support the design. The lab tests conducted and their objectives follow:

- Test 1: Grinding of the UHPC closure joint material for the longitudinal and transverse joint closures between the precast deck modules
    - Evaluate the grindability of the cast-in-place UHPC in relation to the accelerated construction schedule
- Test 2: Placement, handling, and quality of the UHPC material at the intersecting closure joints
    - Evaluate the constructability of intersecting, cast-in-place UHPC joints with respect to the flow characteristics and properties of the material
    - Qualitatively assess the feasibility of the UHPC joint placement procedure for the demonstration bridge
    - Provide an opportunity to educate and showcase an emerging material to bridge designers and contractors
- Test 3: Serviceability and strength of the transverse bridge deck joint at the pier
    - Evaluate the negative bending performance of the module-to-module transverse connection detail at the piers
    - Determine the cracking moment at this location
    - Determine the ultimate moment capacity at this location

Additional investigations for the demonstration bridge conducted but not included in this thesis include live load testing of the demonstration bridge and direct tensile bond testing of the UHPC.

Arrangement of the Thesis

Chapter 2 summarizes the background for this project and presents a review of relevant literature and past work regarding UHPC in bridge design. Chapter 3 discusses the different test methods and materials used in this research. Chapter 4 presents the qualitative and quantitative results and discussion for the testing. Chapter 5, the final chapter, outlines the conclusions and recommendations from the testing program.

CHAPTER 2. BACKGROUND

This chapter presents a summary of the broader SHRP 2 Project R04 initiative and a review of relevant literature in the area of UHPC. The purpose of this chapter is to provide an overview that shows how this research fits theory and practice.

PROJECT BACKGROUND

Second Strategic Highway Research Program

As part of a multi-phase study on accelerated bridge construction which aims to establish new standards and code provisions for the use of typical ABC across the country, a demonstration bridge is being designed, constructed, and tested in Pottawattamie County Iowa. The Second Strategic Highway Research Program, under which this project is funded, is focused on safety, renewal, reliability, and capacity. This ABC demonstration bridge project, SHRP 2 Project R04, is focused on "developing technologies and institutional solutions to support systematic rehabilitation of highway infrastructure in a way that is rapid, presents minimal disruption to users, and results in long-lasting facilities" (Transportation Research Board 2011). Further, the objective of SHRP 2 Project R04 is to "develop standardized approaches to designing, constructing, and reusing (including future widening) complete bridge systems that address rapid renewal needs and efficiently integrate modern construction equipment" (Transportation Research Board 2011). The project includes four phases which provides for research and development/design of the rapid renewal demonstration bridge up through construction and field demonstration. Typical bridges, characterized as bridges with up to 3 spans and a maximum span length of 200 feet, are the focus because of the opportunity for widespread application. This research is the result of SHRP 2 Project R04 Phase III Task 10C, which calls for laboratory testing of the handling and constructability of the UHPC joint material. While Task 10C was the primary research conducted by the team at Iowa State University, additional investigations for the demonstration bridge were performed, including live load testing of the demonstration bridge and direct tensile bond testing of the UHPC.

Bridge Description

The design of the three-span precast modular demonstration bridge has been developed by the Iowa Department of Transportation (IDOT) and the design engineer, HNTB

Corporation, using various details from several other ABC bridges across the country and around the world. The new bridge will replace a concrete haunched girder bridge built over Keg Creek in 1953.

Figure 2.1. Existing US Highway 6 Bridge over Keg Creek

The new demonstration bridge is a precast modular bridge system which includes precast approach pavement slabs. The precast column and capbeam construction for the piers will be connected using grouted couplers, while the precast superstructure deck modules will create a semi-integral abutment allowing rapid construction. On the deck, durable, moment-resisting UHPC joints will connect the deck modules and create no open deck joints across the span. The SHRP 2 Project R04 demonstration bridge will be the first bridge in the United States with moment resisting UHPC joints at the piers.

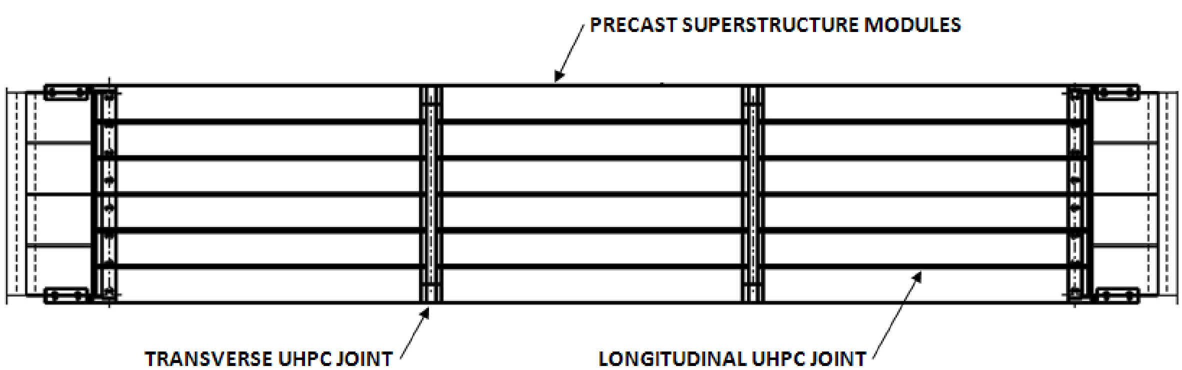


Figure 2.2. Bridge Plan View

(Iowa Department of Transportation 2011)

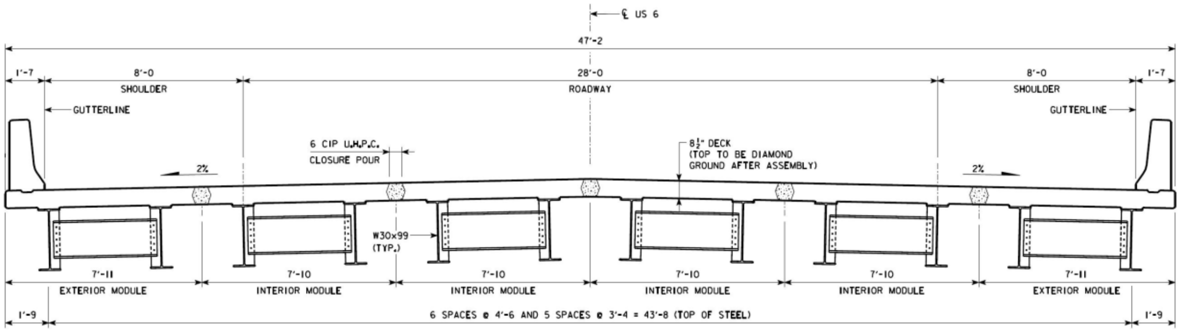


Figure 2.3. Bridge Deck Cross Section

(Iowa Department of Transportation 2011)

At 204 ft 6 in long and 47 ft 2 in wide, the new bridge will consist of a pair of 67 ft 3 in end spans and a 70 ft 0 in center span (Figure 2.2 & Figure 2.3). Six precast deck modules connected by the longitudinal UHPC closure joints make up the bridge cross section.

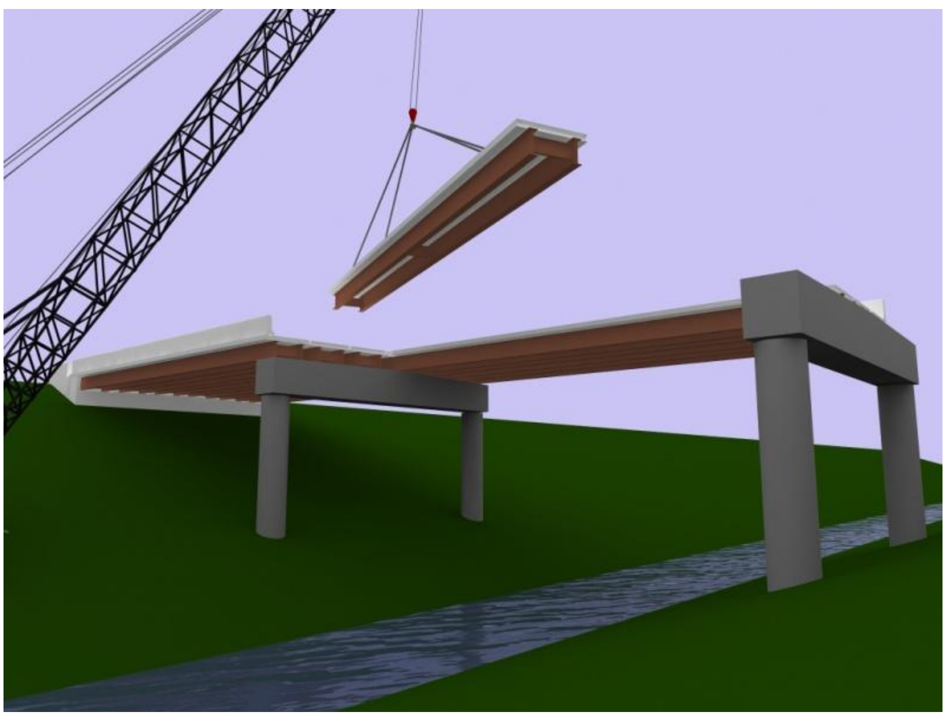


Figure 2.4. Precast Deck Modules

(HNTB Corporation 2011)

Figure 2.5. Completed SHRP 2 Project R04 Demonstration Bridge

(Iowa DOT Office of Bridges and Structures 2011)

By creating a complete ABC design for rapid renewal, the design engineer has been able to reduce the estimated 6-month road closure required for the typical construction to 14 days. Since U.S. 6 in Pottawattamie County is a primary highway route through the county and western Iowa, it provides an excellent platform to showcase the ABC rapid renewal concept for typical bridges.

The evaluation of promising technologies for this demonstration project includes the simultaneous laboratory testing of specific elements that are found to be critical to the speed of construction and service life of the bridge design. The unique design of the transverse deck joint over the bridge piers is one such example in this demonstration bridge. Because the transverse deck joint employs an emerging innovative material in UHPC, and the UHPC joint is the tension reinforcement through the transverse connection over the piers, it is critical to bridge performance. It is, therefore, the focus SHRP 2 Project R04's Phase III laboratory testing.

RELEVANT UHPC MATERIAL BACKGROUND

As the United States faces the challenge of renewing its aging highway infrastructure, the longevity and durability of new structures is of particular interest in addition to accelerated construction. Research into many different materials and techniques to achieve durability has taken place. UHPC, a new class of cementitious material, is one technology that is increasingly being considered to provide durability and longevity in highway infrastructures due to its advanced material behaviors (Graybeal 2009). While widespread implementation of UHPC has not yet taken place in the United States, multiple state departments of transportations have employed UHPC in recent demonstration projects. The SHRP2 Project R04 uses the patented mix, Ductal®, developed by Lafarge Canada. Material background, characteristics, and present applications of UHPC relating to the SHRP2 Project R04 are discussed below.

Material Background and Characteristics

The development of concrete materials known as UHPC is one of the newest advances in current concrete products. Developed in Europe beginning in the early 1990's, the first commercial UHPC product became available in the United States in 2000. Vande Voort (2008) discusses the material characteristics that define the range of cementitious products known as UHPC. Generally, UHPC has compressive strength that is greater than 22 ksi, contains fiber reinforcement, and uses powder components which helps to eliminate some of the typical limitations of normal concrete. Additionally, Graybeal (2011) states that "UHPC has a discontinuous pore structure that reduces liquid ingress, significantly enhancing durability…" when it is compared with normal strength and high performance concretes (HPC). The low permeability is attributed to the fine powders and chemical reactivity which create an extremely compact matrix and small, discontinuous pore structure (Perry and Royce 2010). A typical mix of UHPC contains silica fume, ground quartz, sand, cement, fibers, superplasticizer, and water. The increased material strength of UHPC when compared with HPC, along with its low permeability are both reasons it is being considered as a potential solution for the durability and longevity issues in the country's highway infrastructure.

Researchers in Europe have been at the forefront of the UHPC material testing and literature. However, increasingly, studies into this emerging material have commenced in the U.S., and Graybeal (2006) has completed wide ranging, in-depth testing on the advanced material characteristics of UHPC.

Graybeal (2006) reports that when compared with normal strength concrete, UHPC displays significantly enhanced material properties. Notably, compressive strength, tensile strength, rate of strength gain, and several durability properties significantly exceed those of normal concrete.

While steam treatment of UHPC does considerably initially improve the material's properties, increasing compressive strength by 53 percent, modulus of elasticity by 23 percent, and essentially eliminating long term shrinkage, UHPC still shows very high compressive strengths no matter what type of curing is employed. It was found that the time needed to reach initial set for UHPC was between 12 and 24 hours, which is longer than normal concrete. However, once the initial set takes place, UHPC gains compressive strength very rapidly. The speed of the setting time and strength gain can be controlled using different mix additives. The tensile strength was found to be higher than normal concrete both pre and post tensile cracking. With and without steam treatment, the tensile strength was found to be 1.3 ksi and 0.9 ksi, respectively. Cyclic testing for durability characteristics showed that the UHPC performs very well across the range of tested conditions, and even cracked UHPC cylinders exhibited extremely limited permeability and deterioration. Graybeal's broad material testing regimen provided a base of information critical to the development of current demonstration details using UHPC in highway structures. Vande Voort (2008) extensively discusses UHPC material characteristics, and explains how the low porosity of the UHPC microstructure is the significant factor behind the material's superior durability properties.

Horszczaruk (2004) and Graybeal and Tanesi (2007) conducted abrasion resistance testing on high-strength fiber reinforced concrete and UHPC using the ASTM C944 standard procedure (ASTM). Horszczaruk focused on 12 to 14.5 ksi compressive strength concretes that included larger aggregates than are present in any UHPC mix design. Graybeal and Tanesi, conducted the abrasion resistance testing on UHPC of different curing types and surfaces: cast, blasted, and ground. They found that steam-based curing significantly impacts the abrasion resistance of the material. Untreated specimens lost nearly ten times the amount of mass compared to specimens undergoing steam curing treatment. Additionally, smoother textures tended to be more resistant to abrasion than did the blasted finish.

Ductal®, the patented UHPC mix from Lafarge Canada commercially available in the U.S. was developed by three companies, Lafarge, Bouygues, and Rhodia, nearly two decades ago. Ductal® is composed of silica fume, ground quartz, sand, cement, high tensile strength steel fibers, high range water reducer, and water. The high tensile strength steel fibers included in the mix are 0.008 in in diameter and 0.5 in long. When Ductal® is used; the silica fume, ground quartz, sand, and cement are combined into a premix which arrives in bags along with the steel fibers and high range water reducer from Lafarge Canada. Ductal® JS1000 is the specific mix recommended when UHPC is being used as a joint closure material. The product data sheet from Lafarge Canada, contains further material information, batching, and placement guidelines for the Ductal® JS1000 product (Lafarge Canada).

Current Bridge Applications

Presently there have been multiple applications of UHPC in different components of bridge systems. Applications range from UHPC I-Girders and complete re-decking systems to UHPC joint closures and precast concrete piles. The most prominent examples of using UHPC in bridge superstructure components are discussed below.

I-Girder

In the United States, two simple-span prestressed concrete girder bridges have been constructed using UHPC I-girder shapes. The Mars Hill Bridge in Wapello County, Iowa, was the first to be constructed in 2005 by the Iowa Department of Transportation (Iowa DOT). The bridge was a 111 ft long single span bridge with a 3 girder cross section at 9 ft 7 in spacing and a 4 ft overhangs. The UHPC I-girders are modified Iowa 45-in bulb-tee sections. To save material in the section, the web width was reduced by two inches, the bottom flange by two inches, and the top flange by one in (Bierwagen and Abu-Hawash 2005).

Figure 2.6. UHPC I-Girder Bridge - Wapello County, Iowa

(Bierwagen and Abu-Hawash 2005)

The Virginia Department of Transportation (VDOT) constructed the second bridge using UHPC I-girder shapes. One 81.5 ft long span of the 10 span Route 624 Bridge over Cat Point Creek was built with 5 UHPC I-girders. The girders were 45 inch tall bulb-tee beams and contained no conventional steel stirrups for shear reinforcement because the steel fibers present in the UHPC provided adequate shear resistance (Ozyildirim 2011).

Bulb-Double-Tee Girder

Another deployment of UHPC in bridge construction in the United States involves the design and implementation of a UHPC bulb-double-tee girder or PI section in Buchanan County, Iowa. The 112 ft 4 in long, 24 ft 3 in wide three span bridge utilized 3 PI sections in the 51 ft 2 in center span. IDOT worked in collaboration with the Federal Highway Administration's (FHWA) Turner-Fairbank Laboratory, Iowa State University's Bridge Engineering Center (ISU BEC), and Massachusetts Institute of Technology to develop the UHPC PI section (Keierleber, Bierwagen, et. al 2007).

Figure 2.7. PI Girder Bridge - Buchanan County, Iowa

(Berg 2010)

Decking System

A two-way ribbed precast slab system, or waffle slab, was developed to capitalize on the strength and durability characteristics of UHPC. The longevity of bridge decks could be increased by UHPC’s low permeability characteristics and the strength of the material helps to reduce the mass of material required. This waffle slab system has undergone testing at the ISU BEC and construction of the bridge in Wapello County, Iowa by IDOT took place in fall of 2011.

Field Cast Joint Closures

There have been two instances where UHPC was used as a deck joint closure material in the United States. Two bridges in New York implemented UHPC as joint closure material between precast deck panels. The New York State Department of Transportation (NYSDOT) used the UHPC joint fill material in transverse deck joints of Route 23 Bridge in Oneonta, New York and longitudinal deck joints of Route 31 Bridge in Lyons, New York. However, unlike in the current demonstration project, the transverse deck joints in the Oneonta bridge were located in the positive bending moment region, placing the UHPC joint in compression.

Figure 2.8. UHPC Deck Closure Joints

(Perry and Royce 2010)

Graybeal (2010) conducted testing on the UHPC bridge deck connections used in the NYSDOT's bridges under static and cyclic structural loading at the Turner-Fairbank Highway Research Center's Structural Testing Laboratory. Wheel patch loads were applied adjacent to the UHPC joints for four transverse joint specimens. The test specimens were arranged and tested so that flexural stresses which would be caused by traffic were oriented parallel to the joint. It is important to note that the test setup focused on local flexural behaviors of the deck only. The test did not account for global flexural behaviors of the deck and girder system. Neglecting overall flexural effects of the deck and girder system, Graybeal found no significant UHPC or interface cracks parallel to the transverse joint.

Two specimens representing the longitudinal joint connection were also tested. Wheel patch loads were applied adjacent to the joint and the specimen was oriented so that flexural stresses occurred perpendicular to the joint. Graybeal suggested that the field-cast UHPC joint wouldn't necessarily debond at the connection interface under the low loads and small direct flexure in the longitudinal direction.

CHAPTER 3. METHODS

The evaluation of promising technologies for the SHRP 2 accelerated bridge construction demonstration project includes the simultaneous laboratory testing of specific elements or details that are found to be critical to the speed of construction and durability of the bridge design. The unique design of the transverse deck joint over the bridge piers is one such instance in the demonstration bridge. Because the transverse deck joint utilizes an emerging innovative material in Ultra-High Performance Concrete (UHPC) and the UHPC joint experiences high levels of flexural tension, it is critical to the bridge's performance and thus the focus of the lab testing.

Several areas of testing for the UHPC joints were identified during the design of the demonstration bridge. While a few of these areas have been addressed by researchers around the country, some aspects of the demonstration bridge were examined through further lab testing to support the bridge's design.

The lab tests conducted for this study include abrasion testing of the UHPC closure joint material, constructability testing of the intersecting deck joints, and strength and serviceability testing of the transverse deck joint at the pier. The methods used in this testing regimen were selected to address these objectives: to determine the grindability of cast-in-place UHPC; to assess the feasibility of intersecting deck joint placement, and to evaluate the bending performance of the module-to-module transverse connection detail at the piers.

RESEARCH DESIGN

To address the objectives of this study, abrasion testing was conducted in the Portland Cement Concrete Pavement and Materials Research Laboratory and constructability and strength and serviceability testing was conducted in the structures laboratory at Iowa State University.

UHPC ABRASION TESTING

The abrasion testing of the cast-in-place UHPC was done in order to determine the early age grindability of the material when used in the demonstration bridge. Identifying a period of time in which the contractor is able to grind the joint material without causing damage to the joints or equipment had to be identified. The ability of the contractor to grind the material in a timely manner is extremely important to the schedule of the project. In the demonstration bridge it was specified that the UHPC closure joint attain 10,000psi of compressive strength before it should be ground. This test helped to determine the relative ease of grinding for this material after the 10,000psi threshold has been reached. Experimental variables for this test included the maturity of the UHPC and the specimen surface finish.

Mixing and Casting UHPC Cylinders

In order to complete the abrasion testing, UHPC cylinders were mixed and cast. From one cylinder, four test specimens were produced. The batching and casting was completed in one day utilizing three batches of the same UHPC mix design proportions.

In total, 24-four in by eight in UHPC cylinders were cast from three batches of UHPC to create the test specimens. Twelve of the cylinders were cut in half to create four surfaces per cylinder for testing. Each surface was then treated as a separate specimen. The other twelve cylinders were used in compressive strength tests which were used to correlate maturity and compressive strength of the UHPC to abrasion test results. Cylinders were cured at 40°F, 70°F, and 100°F and tested 2, 4, 7, and 28 days after casting.

Table 3.1. Specimen Curing and Testing Matrix

Days After Placement	40°F Cure	70°F Cure	100°F Cure
2	4 Specimens	4 Specimens	4 Specimens
4	4 Specimens	4 Specimens	4 Specimens
7	4 Specimens	4 Specimens	4 Specimens
28	4 Specimens	4 Specimens	4 Specimens

Ductal® JS1000, from Lafarge Canada, was the UHPC premix material used for the abrasion testing specimens. The UHPC mix designed by Lafarge Canada included Ductal® JS1000 premix, water, Chryso Premia 150 super plasticizer, and steel fiber. UHPC mix design proportions are included in Table 3.2 and Table 3.3.

Table 3.2. Abrasion Testing Specimen UHPC Mix Design 1 (0.58 ft³ Batch)

Material	Weight, lbs	Mix Proportion
Ductal JS1000 Premix	79.9337	*87.42%*
Water	4.7312	*5.17%*
Chryso Premia 150	1.0931	*1.20%*
Steel Fiber	5.6818	*6.21%*
	91.4398	*100.00%*

Table 3.3. Abrasion Test Specimen UHPC Mix Design 1 (0.40 ft³ Batch)

Material	Weight, lbs	Mix Proportion
Ductal JS1000 Premix	54.8148	*87.42%*
Water	3.2444	*5.17%*
Chryso Premia 150	0.7496	*1.20%*
Steel Fiber	3.8963	*6.21%*
	62.7052	*100.00%*

The batching procedure was adapted from the procedure recommended by Lafarge Canada for the Ductal® JS1000 premix in a Imer Mortarman 750 mixer to batch the UHPC mix design (Lafarge Batching Procedure). A Lancaster Products 1.5 ft³ mixer was used to mix the two 0.58 ft³ batches and one 0.40 ft³ batch of UHPC for the test specimens.

Figure 3.1. Lancaster Products Mixer

Abrasion and Compressive Strength Tests

Three surface finishes were tested for grindability during the abrasion testing; a rough top surface, a diamond cut surface, and a smooth formed surface. Upon curing the cylinders at the various temperatures, four specimens were produced by cutting each cylinder in half. For one cylinder at each cure temp and time, one rough top surface, one smooth form surface, and two diamond cut surfaces were tested.

Figure 3.2. Diamond Cut and Rough Top Surface Finishes

To evaluate the UHPC material for grindability, testing was completed using ASTM C944, the Standard Test Method for Abrasion Resistance of Concrete or Mortar Surfaces by the Rotating-Cutter Method (ASTM C944). For fabricated concrete or mortar based specimens, this test helps indicate the relative wearing resistance. ASTM C944 uses a drill press with an abrading cutter rotating at 200 rotations per minute. The normal force exerted on each specimen surface in this test is 22 ± 0.2 lbf. The rotating cutter head leaves a 3.25 in diameter abraded circular area.

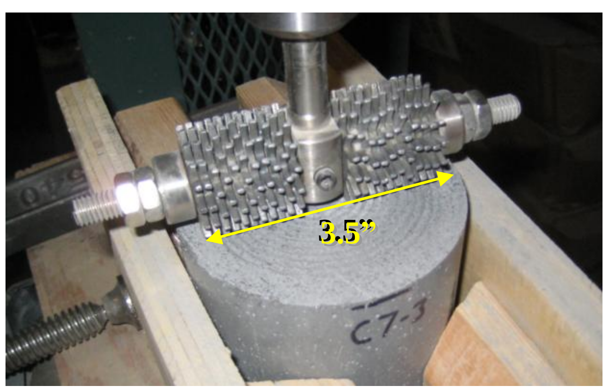


Figure 3.3. Rotating Cutter Head

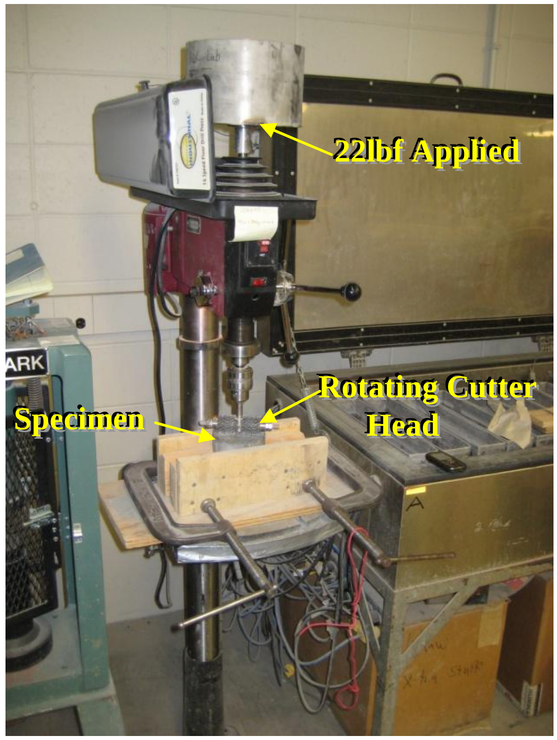


Figure 3.4. ASTM C944 Test Setup

Following the ASTM C944 standard, the mass of each specimen was determined to the nearest 0.1g. The specimen was then clamped into the testing device such that no rotation could take place. To ensure the quality of each test, special care was taken to ensure that each specimen surface was level and normal with the shaft of the rotating cutter head. Once properly secured in the device, the motor was started and the cutter was slowly lowered into contact with the specimen. Following continual abrasion of the specimen for two minutes, the specimen was removed from the testing device, cleared of dust and debris, and massed again to the nearest 0.1g. This process was repeated two additional times for each individual specimen. In total, twelve abrasion tests were done on each test day. Four tests were completed for each of the three curing temperatures at 2, 4, 7, and 28 days after batching.

Compressive strength tests were simultaneously conducted on corresponding cylinders of the same age and curing temperature using ASTM C39, Standard Test Method for Compressive Strength of Cylindrical Concrete Specimens (ASTM C39). To ensure the ends of the cylinders were smooth and parallel, the ends of each cylinder were cut prior to testing. Because of the high strength of the UHPC, capping compound was not used during the compressive strength testing. Description of the surface finish, size of the specimen, concrete maturity data, time of abrasion, normal load, loss in mass, and general notes were all recorded for each specimen as a result of these tests.

JOINT CONSTRUCTABILITY TESTING

The interesting deck joint mock-up was built in order to evaluate the constructability of intersecting cast-in-place UHPC joints, qualitatively asses the feasibility of the UHPC joint placement procedure, and provide an opportunity to demonstrate casting the material for bridge designers and contractors. Effectively casting the UHPC deck joints is essential to the construction schedule and performance of the SHRP 2 ABC demonstration bridge project. This mock-up, which replicated the conditions in the demonstration bridge, provided an opportunity to understand the flow characteristics and properties of the UHPC mix design with respect to the actual conditions. This assisted the bridge designer and contractor in planning material staging and placement.

Designing and Constructing Intersecting Joint Formwork

Formwork for a representative portion of the intersection region of transverse and longitudinal UHPC deck joint was designed and constructed in order to conduct the constructability testing (Figure 3.6). The finished intersecting joint specimen was 6 ft 6 in long by 7 ft 4 in long in the transverse and longitudinal joint directions, respectively. The transverse joint, which runs perpendicular to the bridge traffic, measured 16 in wide while the longitudinal joint, which runs parallel to the direction of traffic, measured 6 in wide (Figure 3.5). Each joint is 8 1/2 in thick, matching the precast module concrete deck thickness.

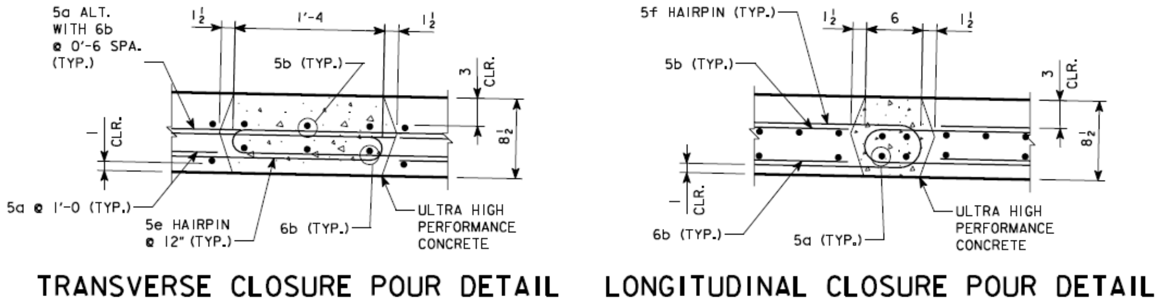


Figure 3.5. UHPC Deck Joint Details

(Iowa Department of Transportation 2011)

The specimen contained all of the steel reinforcement in the joint detail including those which protruded from the precast deck modules. In order to fully replicate the demonstration bridge conditions, the specimen was constructed with a two percent bridge deck cross slope.

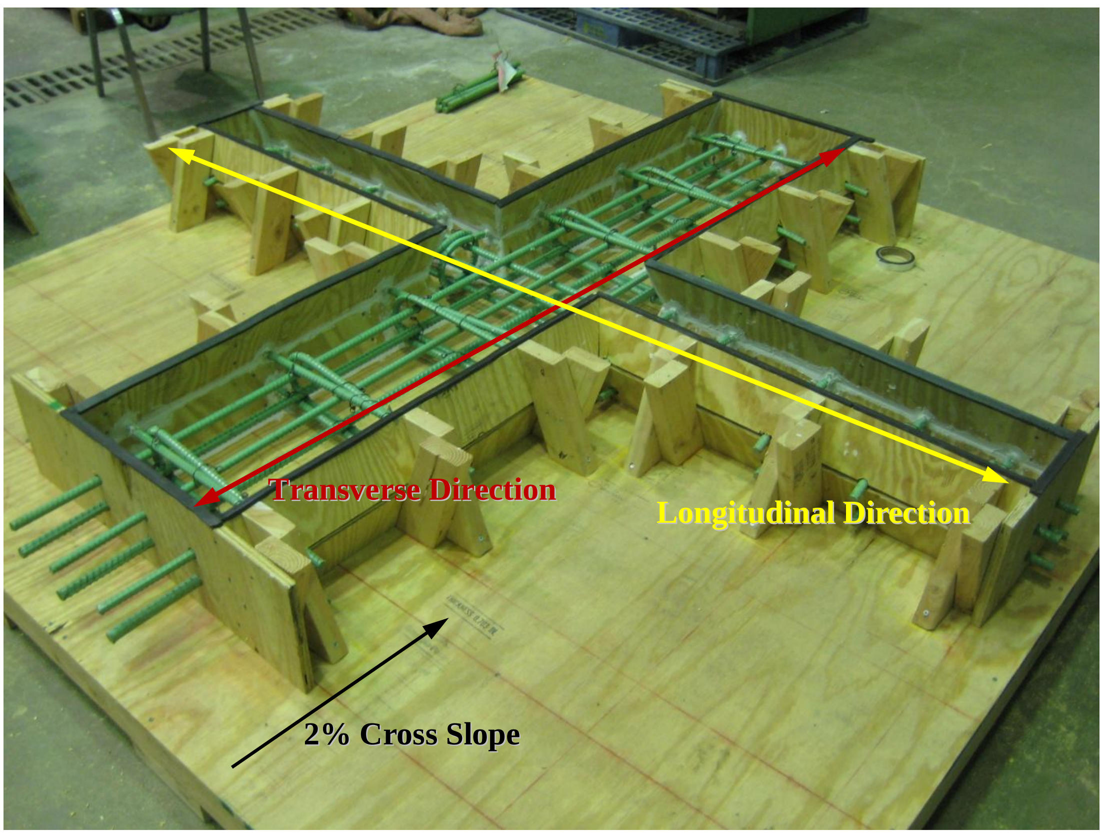


Figure 3.6. Intersecting Joint Specimen Formwork

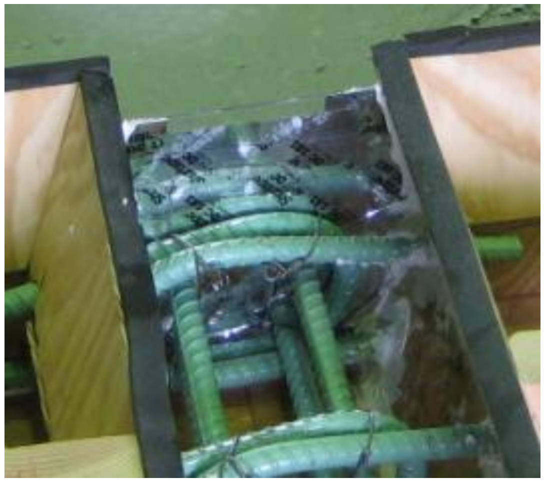


Figure 3.7. Acrylic Glass Vertical Bulkhead

To replicate the proposed demonstration bridge UHPC placement technique, the last element of the formwork for the specimen was the acrylic glass bulkheads which aimed to prevent the formation of cold joints. In the test specimen, the bulkheads were located in the longitudinal joint approximately two in from the transverse joint. The placement of the vertical bulkheads in the longitudinal joints allowed for continuous placement of the transverse closure joint.

Mixing and Casting Intersecting Joint Specimen

The UHPC mix design that was specifically produced for the SHRP 2 ABC demonstration bridge by Lafarge Canada was used during the constructability testing of the intersecting joint detail. The JS1100RS 60/40 mix design included Ductal® Light Grey Premix, water, Chryso Premia 150, Chryso Optima 100, and steel fibers. A technical service engineer from Lafarge Canada was present during the batching and casting of this specimen.

Table 3.4. Constructability Test Specimen UHPC Mix Design 2 (5.11 ft³ Batch)

JS1100RS 60/40 - Light Grey Premix		
Material	**Weight, lbs**	**Mix Proportion**
Ductal JS1100 Premix	700	*86.74%*
Water	47.8	*5.92%*
Chryso Premia 150	5.7	*0.71%*
Chryso Optima 100	3.8	*0.47%*
Steel Fiber	49.7	*6.16%*
	807	*100.00%*

Under the supervision of the Lafarge Canada representative, batching was completed for the entire intersecting joint specimen. For each 5.11 ft^3 batch, 14 - 50lb bags of Ductal® Light Grey Premix were emptied into the drum and mixed to gain homogeneity. Once completely mixed, the Premia 150, Optima 100, and water were added. The batch was then mixed until the turning point was reached. At the turning point, the wetted batch changed from its granular mix state to a semi-plastic state. Upon reaching this point, the steel fibers were incorporated into the batch. The batch could then be discharged after the steel fibers had been fully integrated (Lafarge Batching Procedure).

The total volume of the intersecting joint specimen was 9.54 ft^3. Because the Imerman 750 mixer used in the lab is limited to a 5.11 ft^3 batch of UHPC and cylinders for compressive strength tests needed to be cast as well, the specimen was cast in three batches. By utilizing the acrylic glass vertical bulkheads, the transverse joint could be partially filled with the first batch and then completed with the second. In an attempt to gain a homogeneous placement and eliminate a horizontal cold joint in the transverse joint, prior to placing the second batch, the first batch was agitated in the mold to disrupt the drying "skin" that began to form within five minutes of placement. The remaining UHPC from the second batch was then used to place one side of the longitudinal joint. Finally, the third batch filled the last portion of the longitudinal joint as well as the 18 – 4x8in cylinders used for compressive strength testing.

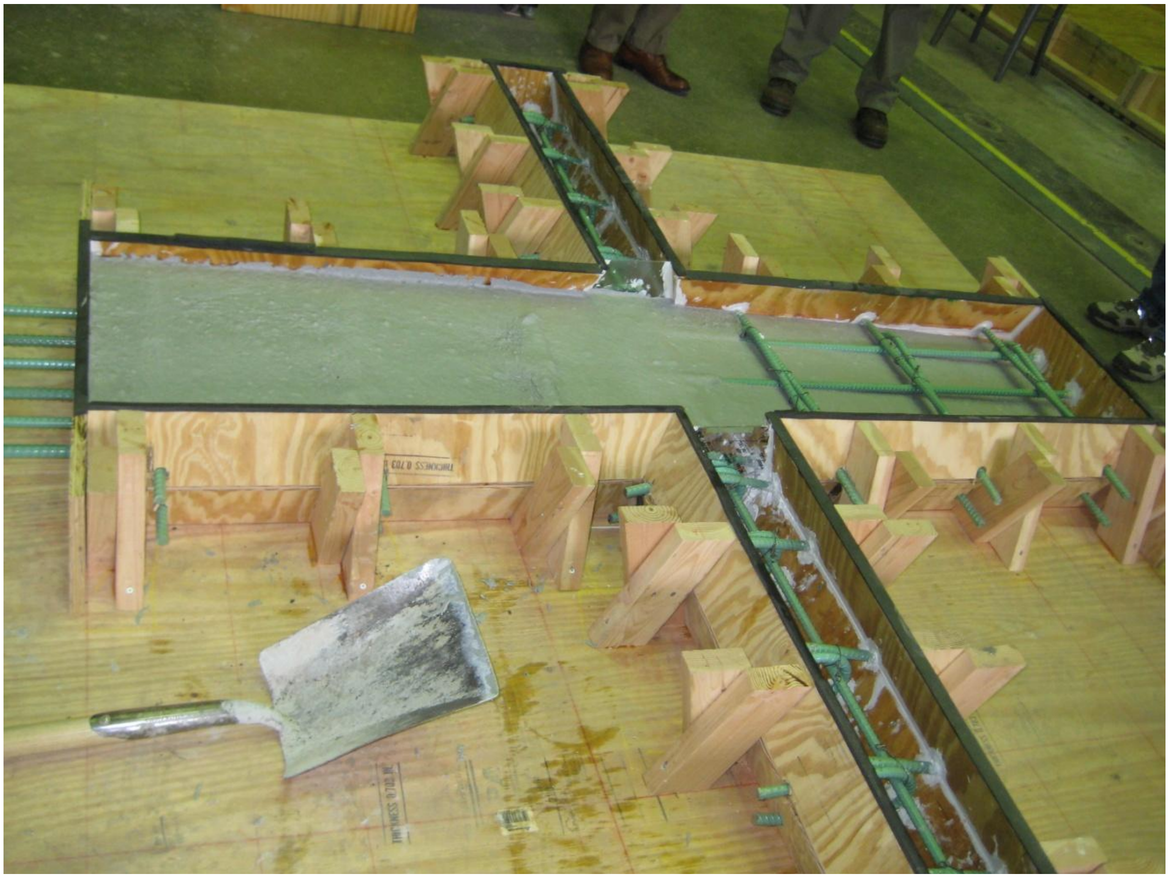


Figure 3.8. Placement of Transverse UHPC Joint

The UHPC for the entire specimen was placed from the low end of the two percent cross slope to the high end. As recommended for the demonstration bridge, plywood top forms were attached as the formwork filled up. At the high end of the joint, “chimneys” were constructed in the top form to overfill with UHPC, build up hydrostatic head pressure, and ensure that the entire joint was filled. While casting the specimen, there was no vibrating of the UHPC because it is a self consolidating material. After the mock-up was cured and removed from the forms, it was cut up into several sections to examine consolidation and locations of potential cold joints.

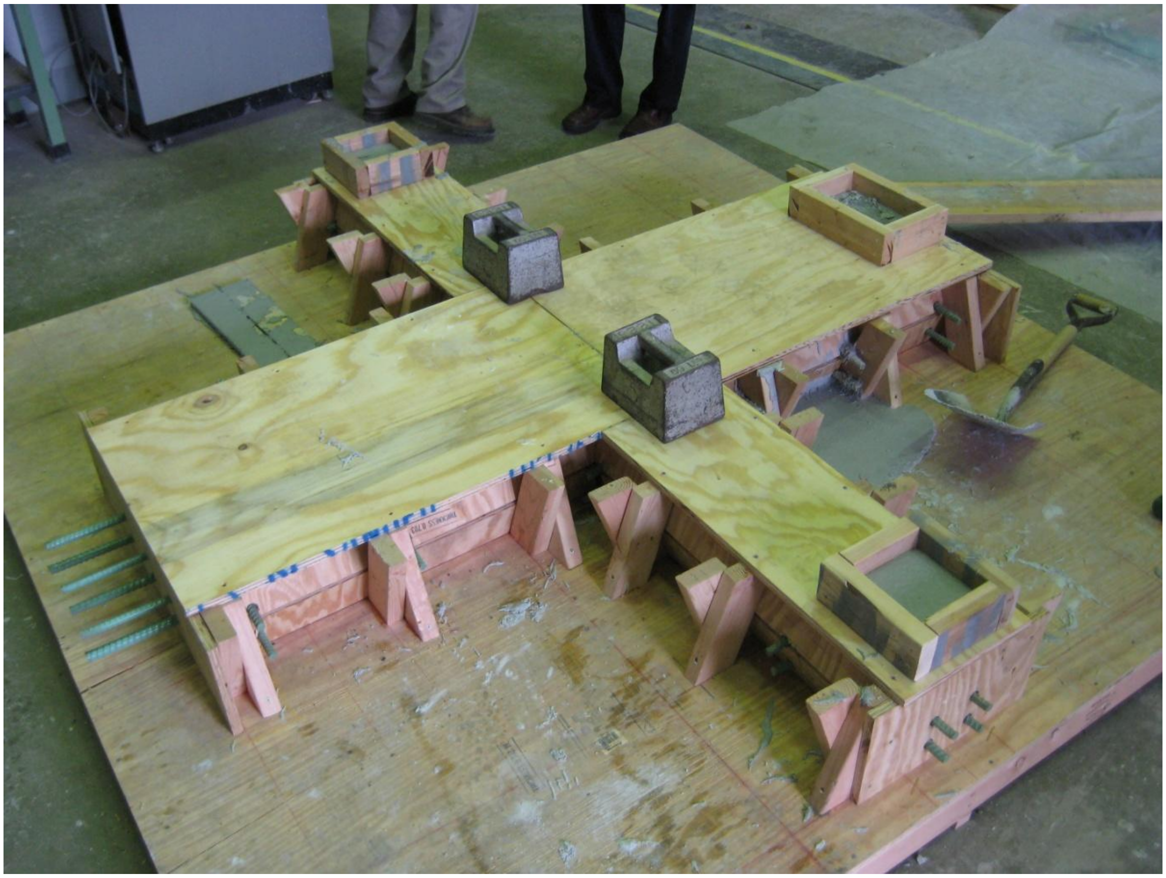


Figure 3.9. Plywood Top Forms

TRANSVERSE JOINT STRENGTH AND SERVICEABILITY TESTING

The module-to-module transverse connection used in the SHPR2 ABC demonstration bridge was a unique and critical detail that had never been implemented in a bridge nor tested to quantify structural performance (Figure 3.10). To evaluate performance of this detail, a mock-up of the connection was constructed and subjected to increasing levels of moment. The transverse connection specimen at the piers provided the opportunity to evaluate the negative bending performance of the detail in order to determine the cracking moment and ultimate moment capacity of this connection detail. This was done through static and cyclic testing at service level conditions as well as static ultimate moment conditions. The performance of this particular detail is critical to the long term durability of the demonstration bridge.

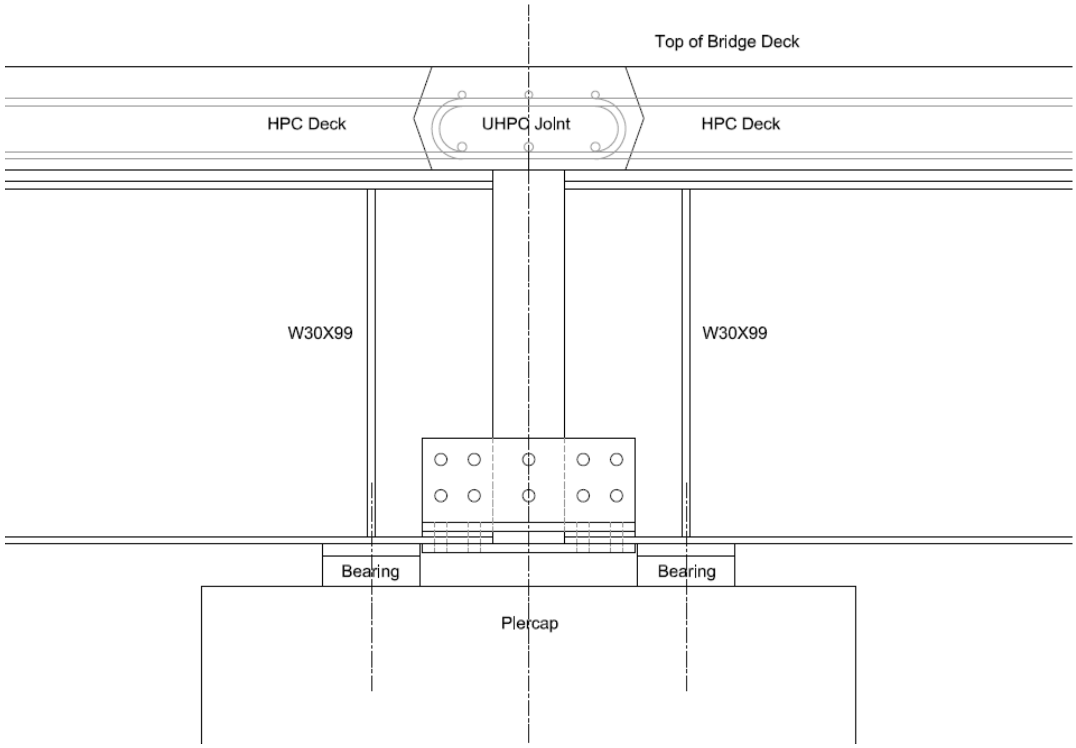


Figure 3.10. Module-to-Module Transverse Connection Detail

Designing and Constructing Full-Scale Transverse Connection Specimen

A full-scale specimen replicating the module-to-module transverse connection at the pier was designed and constructed in the lab to allow for testing of the connection detail. The specimen had a total length of 40 ft 6 in and was comprised of 2 prefabricated deck modules connected with the transverse UHPC joint under investigation. The specimen length was chosen such that the negative moment inflection points were located inside of the specimen's ends. Each precast deck module consisted of 2 – W30X99 steel girders cast compositely with a 7 ft 4 in wide by 8.5 in thick concrete deck. The W30X99's used in the prefabricated deck modules were 20 ft long and were connected by 2 – MC18X42.7 diaphragm members. These members constituted the steel frame of each module.

The steel frames for the prefabricated deck modules were fabricated in Muscatine, Iowa and shipped to the laboratory at Iowa State University. In the laboratory, the modules were constructed in an upside-down orientation such that the concrete deck could be cast on the ground. The two steel frames were placed in their respective deck slab forms upon arrival.

All epoxy coated reinforcing steel bars present in the deck slab were placed and tied prior to setting the frames.

Figure 3.11. Prefabricated Deck Module Construction

See Appendix A for full steel frame fabrication plans. General notes for hardware, structural steel, and reinforcement bars used in the prefabricated deck modules were as follows:

- ASTM A709 Grade 50W Structural Steel
- High-Strength ASTM A325 Type III Bolts
- ASTM A563 Heavy Hex Nut Grade DH3
- ASTM F436 Type III Washers
- Grade 60 Epoxy Coated Rebar

CV-HPC-D mix, an Iowa DOT high performance concrete bridge deck mix specific for the western region of Iowa specified to be used in the demonstration bridge, was used for the specimen’s deck slab. The mix employed river rock commonly found in western Iowa. In order to obtain this material in central Iowa, a special order for the aggregate was placed with the ready mix plant.

Table 3.5. HPC Mix Proportions

Material	Relative Proportion (by Volume)
Cement	0.126
Fly Ash	20% Max replacement by weight (mass)
Water	0.148
Coarse Aggregate	0.300
Class V Aggregate	0.366
Air	0.060

Eight and a half cubic yards of HPC were cast to complete the two deck modules. In addition, 24 – 4x8in cylinders and 6 – 6 in by 6 in by 3 ft beams were cast for compressive and flexural strength tests (ASTM C39 & ASTM C78).

Figure 3.12. Completed Deck Module

As noted previously, the specimen was cast in an upside-down orientation, this was done for safety reasons in the laboratory. Once cast, the deck modules were positioned on supports, one temporary and one permanent, then connected via four steel angle plates (Figure 3.10). The steel angles were the compressive force path through the transverse module-to-module connection detail. The transverse UHPC joint would act as the tension force path for the connection detail. The specimen continued to be supported via screw jacks under the transverse joint formwork until the UHPC material was cured. This casting sequence was chosen to replicate the demonstration bridge design's simply supported condition for dead load and continuously supported condition for live load.

Casting the Transverse UHPC Joint

To complete the specimen for strength and serviceability testing, the transverse UHPC joint had to be cast to connect the two prefabricated deck modules. Using the UHPC mix design and batching procedure specified by Lafarge Canada for the demonstration bridge and used in the constructability testing, the transverse joint was cast in 2 – 5.11 ft³ batches (Table 3.4). In addition, 45 – 4x8in cylinders were cast for compressive strength testing at 1, 2, 4, 7, and 28 days. The cylinders were cured at 60°F, 70°F, and 90°F to evaluate performance based on variation in temperature that could occur during the SHRP 2 ABC demonstration bridge construction period.

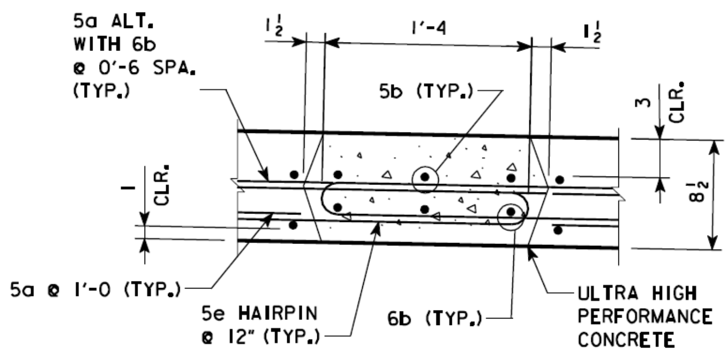


Figure 3.13. Transverse UHPC Deck Joint Detail

Prior to placing the UHPC joint, the adjoining HPC deck surfaces were prepared as recommended by Lafarge Canada. The HPC surfaces were removed from the forms, brushed with a steel brush grinding head, and pressure washed with water. On the morning of the UHPC pour, the HPC surfaces were wetted in order to attain saturated surface dry conditions during placement.

Figure 3.14. UHPC Joint Preparation and Placement

According to the project specifications, once the UHPC material reaches 14,000psi of compressive strength, traffic may be allowed on the demonstration bridge. This meant that specimen load testing needed to commence immediately upon reaching the 14,000psi compressive strength threshold. Based on results from the UHPC in the constructability testing, the threshold would likely be reached four days after joint placement. The entire load testing frame for the specimen was constructed and load actuators positioned prior to placement of the transverse UHPC joint so that testing could commence as soon as the UHPC reached 14,000psi compressive strength.

Instrumentation for the Transverse Module-to-Module Connection Specimen

To monitor cracking of the deck and/or deck joint, stain levels were monitored throughout the thickness of the deck at locations on or near the joint. A combination of embedded bonded strain gages, surface mounted strain gages, and string potentiometers for deflection were installed on the specimen to analyze the performance of the entire transverse module-to-module connection. Bonded strain gages were affixed to reinforcing bars in both precast HPC deck slabs and in the transverse UHPC joint.

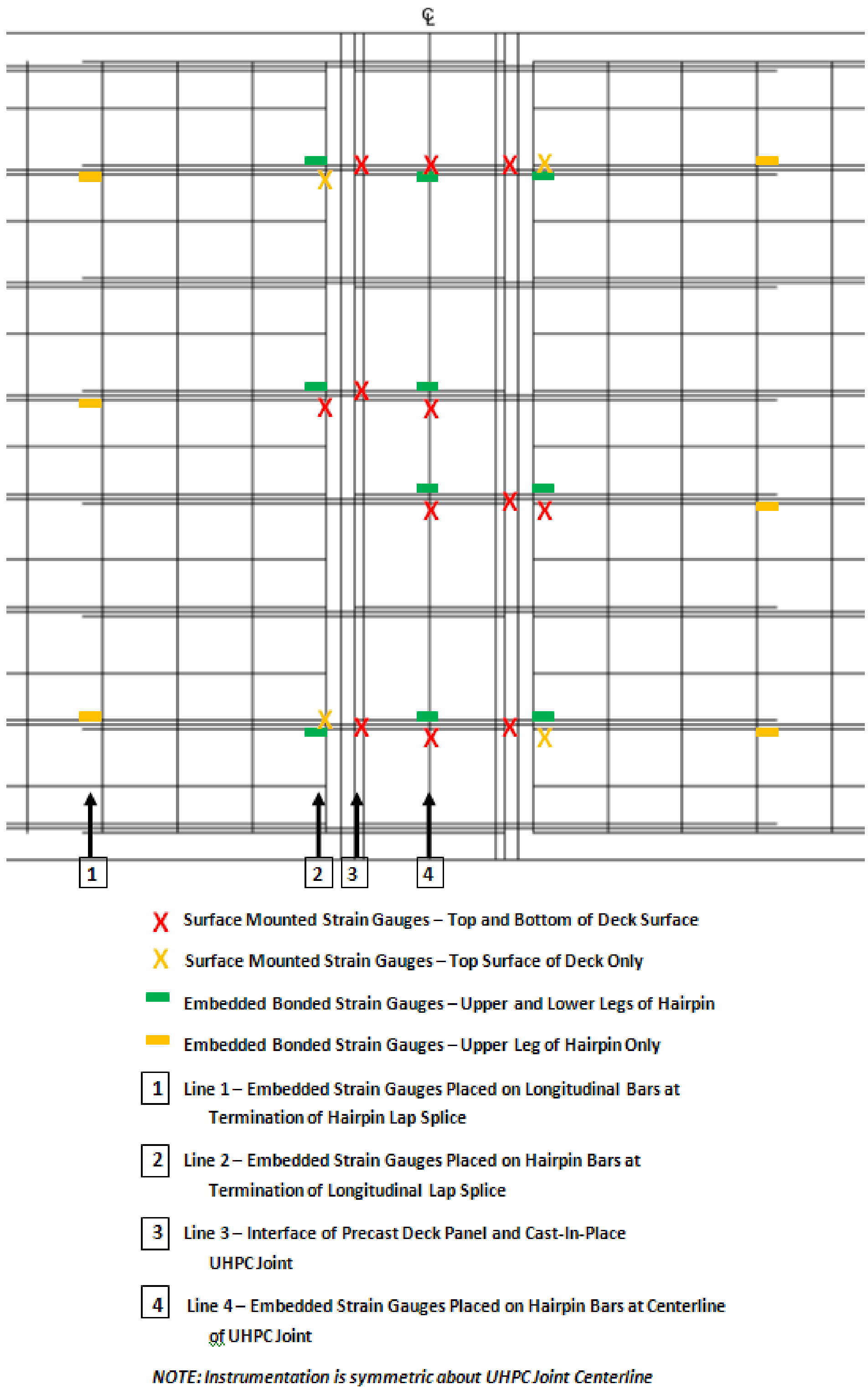


Figure 3.15. Connection Instrumentation Locations, Plan View

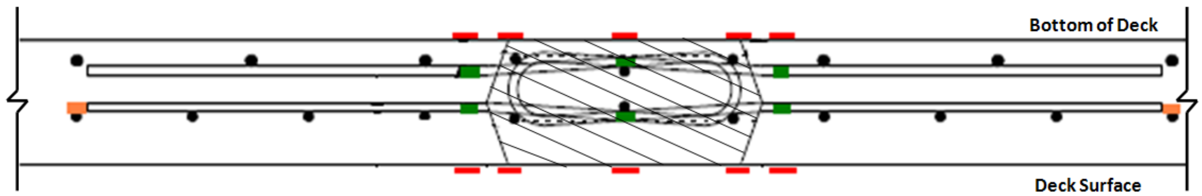


Figure 3.16. Connection Instrumentation Locations, Section View

The specimen was instrumented at likely locations for cracks to occur. Those locations were as follows:

1. To monitor strains in the prefabricated deck panels, embedded bonded gages were placed on the steel hairpins where the longitudinal deck reinforcement terminates. Twelve embedded bonded strain gages were used on the upper and lower legs of the hairpins at locations directly over the steel beams and at midspan of the deck between the beams on both superstructure modules. In addition, eight surface mounted strain gages (three on the top surface of the deck and one on the bottom for each module) were used as well.

   An additional six embedded bonded strain gages (three in each deck module on the top mat of reinforcement) were placed on longitudinal reinforcing bars at the termination of the hairpin lap splice.
2. To identify strain levels at the interface of precast HPC deck panel and the UHPC joint, 12 surface mounted gages (3 on the top surface, 3 on bottom surface for each interface) were used.
3. To quantify strain at the centerline of the UHPC joints, 12 embedded bonded strain gages were used on the upper and lower legs of the hairpins at the transverse centerline of the joint corresponding to the locations of each steel beam and at the longitudinal centerline of the deck between the beams. In addition, six surface mounted strain gages were mounted on the top and bottom surfaces of the UHPC joint at corresponding locations to the embedded strain gages.

4. In addition, eight embedded bonded strain gages were placed on the straight transverse lacer bars within the UHPC joint.
5. Additional gages included four embedded bonded strain gages mounted to the surfaces of the steel angle connectors between opposing steel beams across the joint.
6. Specimen displacements were measured with seven string potentiometers mounted to the lab floor (three directly under the centerline of the transverse joint, one at each bearing location, and one at the transverse centerline of each module).

Complete figures of instrumentation location and labeling are located in Appendix B.

Figure 3.17. Embedded Bonded Strain Gages in HPC Deck

Figure 3.18. Surface Mounted Strain Gages on Top of Deck

Calculating Testing Load Levels

Design moment values were obtained from the demonstration bridge design engineer, HNTB, and validated at Iowa State University. Using the AASHTO HS-20 vehicle and a conservative lateral live load distribution factor of 1.0, Service Level I design live load moment was calculated at -394 kip-feet. The design moment for Service Level II was -512.2 kip-feet. Ultimate design moment capacity for the module-to-module transverse connection was -2,016 kip-feet. Due to the possibility of an HS-20 load on the other spans of the continuous bridge deck, a +74 kip-foot live load moment was initially applied to the specimen.

Actuator loads for Service Level I and Service Level II in the testing regimen were calculated to be 14 kips and 21 kips, respectively, per actuator. The force per actuator necessary to load up to the design ultimate moment capacity was expected to be 103 kips.

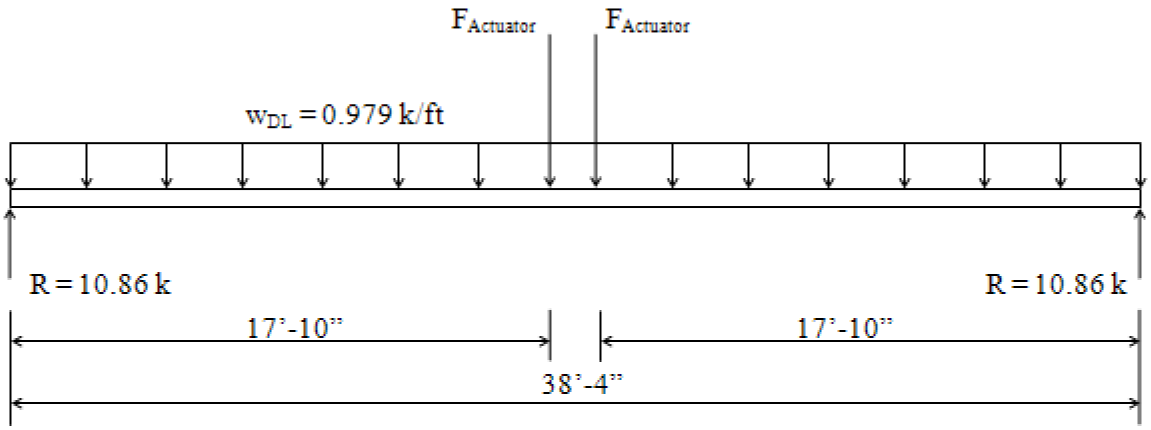


Figure 3.19. Test Specimen Free Body Diagram

$$\text{Design Moment} = \frac{wl^2}{8} + \mathrm{F}_{\text{actuator}} * a$$

$$\text{Service Level I Moment} = 394\ \text{k-ft} = \frac{0.979\text{k/ft} * 38.333^2\text{ft}}{8} + \mathrm{F}_{\text{actuator}} * 17.833\text{ft}$$

$$\mathrm{F}_{\text{actuator}} = 12\text{k per actuator}$$

$$\text{Service Level II Moment} = 512.2\ \text{k-ft} = \frac{0.979\text{k/ft} * 38.333^2\text{ft}}{8} + \mathrm{F}_{\text{actuator}} * 17.833\text{ft}$$

$$\mathrm{F}_{\text{actuator}} = 18.7\text{k per actuator}$$

$$\text{Service Level II Moment} = 2016\ \text{k-ft} = \frac{0.979\text{k/ft} * 38.333^2\text{ft}}{8} + \mathrm{F}_{\text{actuator}} * 17.833\text{ft}$$

$$\mathrm{F}_{\text{actuator}} = 103\text{k per actuator}$$

Conducting Service Level Static Testing

Load testing of the module-to-module transverse connection specimen was performed through Service Level I and up to Service Level II moment. Loads were applied with two hydraulic actuators each fitted with load cells and connected to spreader beams (Figure 3.21 and Figure 3.22). The spreader beams allowed for load application replicating the demonstration bridge's bearing support conditions at the pier. Lubricated steel plates acted as the bearing points for load application. The lubricated plates allowed the specimen and spreader beam to act independently. Four days after the placement of the transverse UHPC joint, upon reaching the specified 14,000psi compressive strength threshold for the cast-in-place UHPC, the actuators were placed in deflection control and the joint formwork removed.

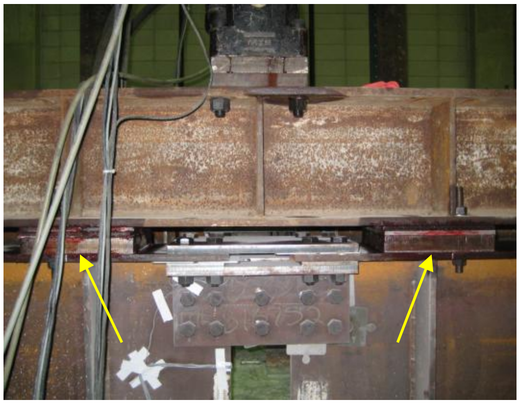


Figure 3.20. Lubricated Steel Bearing Plates

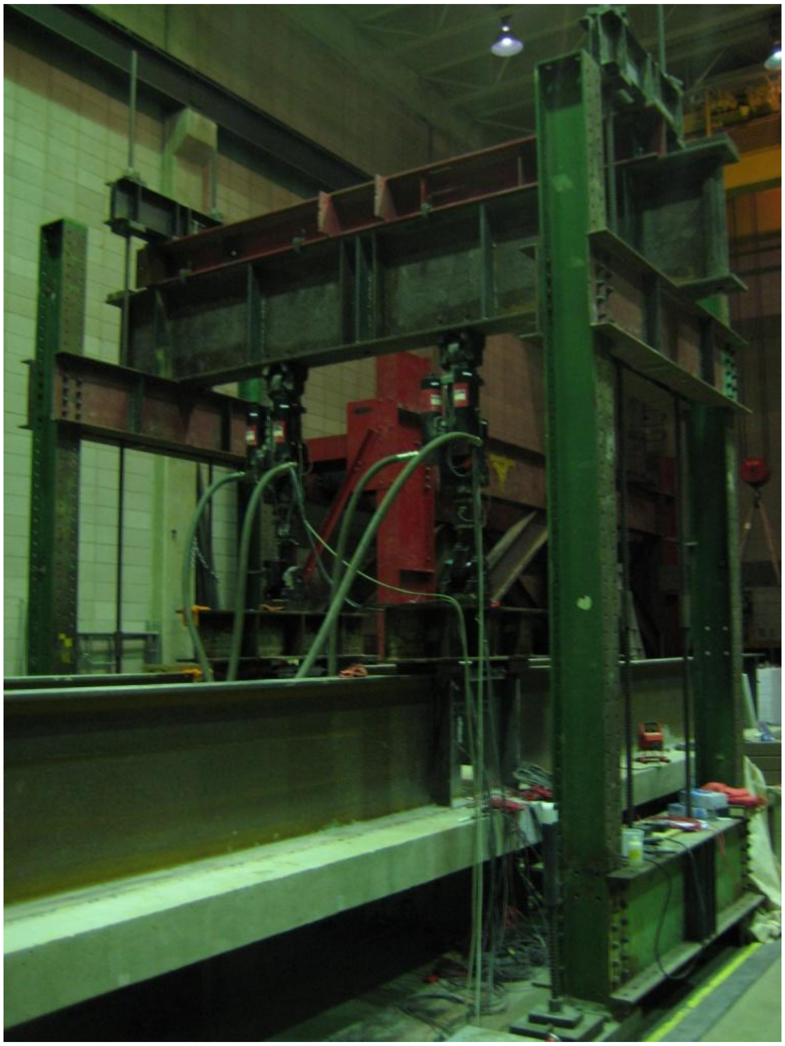


Figure 3.21. Load Frame for Service Level Testing

Three incremental load tests were performed through Service Level I and up to Service Level II moment conditions. The incremental load test was completed with the actuators in load control. Using load control and lubricated steel bearing plates for the tests allowed for replication of the bearing conditions in the demonstration bridge. During the incremental tests, the specimen underwent visual inspection on the top and bottom deck surfaces.

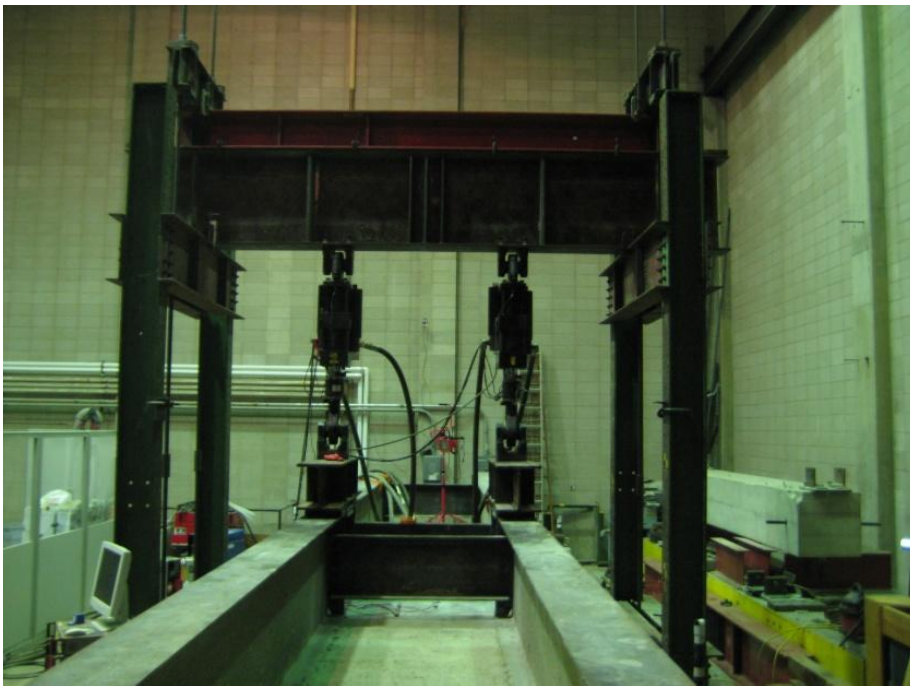


Figure 3.22. Load Test Setup

Upon completion of the incremental load testing, the cyclic load testing through the full range of service level conditions commenced. One million load cycles were completed over a period of ten days. Visual inspection and marking of cracking took place every 250,000 cycles. This allowed for the detection of cracks and further monitoring for crack propagation in the HPC deck panels and in the transverse UHPC joint.

Conducting Service Level Testing with Connection Retrofit

After analyzing results from the initial service level load testing and observing the formation of undesirable cracks, HNTB, the design engineer, designed a retrofit for the transverse module-to-module connection (Figure 3.23 & Figure 3.24). The retrofit employed high-strength steel rods mounted just under the deck surface to post-tension the entire connection and lower tensile strain levels present in the HPC deck and UHPC joint to below the expected cracking threshold in order to prevent any possible cracking or debonding in the deck at the interface.

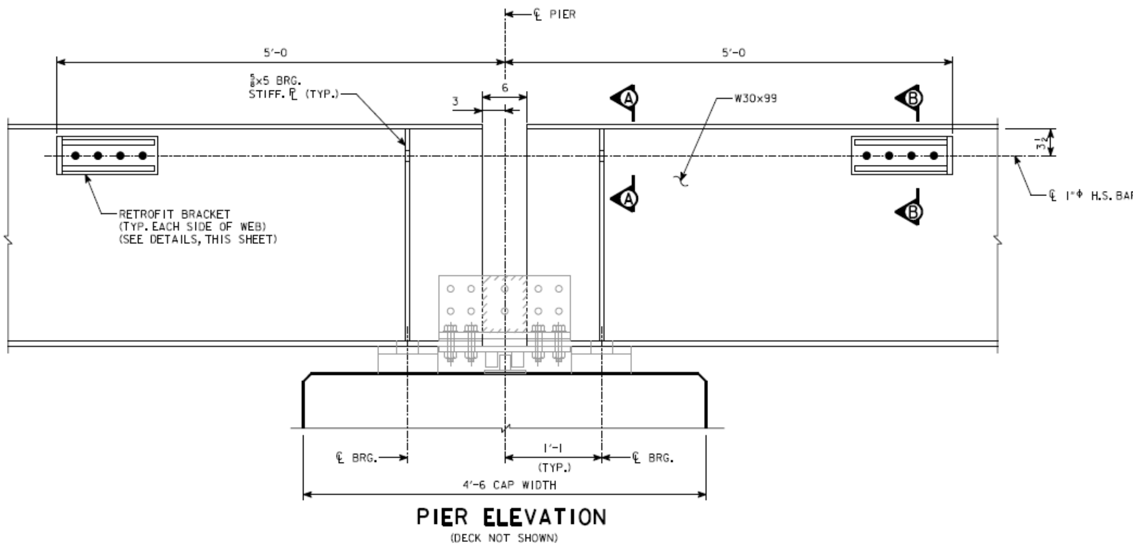


Figure 3.23. Connection Retrofit Detail

(Iowa Department of Transportation 2011)

Figure 3.24. Connection Retrofit Installed

Four-one in diameter high strength threaded rods were installed following the attachment of the ASTM A709 Grade 50W steel brackets to the W30X99 beams. The high strength rods were initially post-tensioned to achieve an effective force of 60 kips per rod at each location. Once the retrofit detail was successfully installed, the incremental static service level load testing through Service Level II moment was conducted in the same manner as previously described. Upon completing service level testing of the 60 kip per rod post-tensioned retrofit, the high strength rods were post-tensioned up to 70 kips per rod. The service level testing was repeated at the 70 kip post-tensioning force level as previously described.

Conducting Ultimate Moment Capacity Testing

Once the static testing of the modified detail was completed, the post-tensioning rods were removed. The final testing done on the specimen identified the ultimate moment capacity of the original transverse module-to-module connection at the pier. In order to complete the ultimate load testing, larger capacity actuators replaced those used for the service level static and cyclic testing (Figure 3.25). By incrementally loading the specimen in load control through its expected capacity (2,016 kip-ft) to failure, the performance of the connection was analyzed and the ultimate moment capacity of the connection was verified.

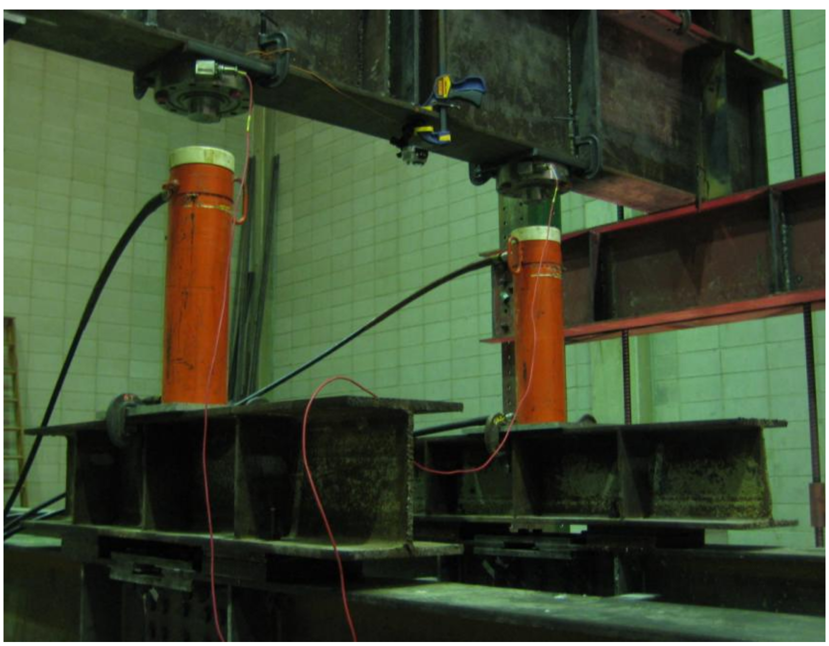


Figure 3.25. Actuators for Ultimate Moment Capacity Testing

CHAPTER 4. RESULTS & DISCUSSION

Quantitative and qualitative results of the primary laboratory testing regimen carried out for abrasion testing, joint constructability testing, and transverse joint strength and serviceability testing are presented in this chapter along with the results of various material tests (e.g., compressive strength of concrete, flexural strength of concrete, etc.) which accompanied the primary testing regimen.

MATERIAL PROPERTY TESTS

UHPC Quality Control Tests

Quality control testing during the UHPC batching process in the laboratory at Iowa State University included temperature readings as well as static and dynamic flow testing according to Lafarge's flow testing procedure based on ASTM C230. Results of the quality control testing are presented in Table 4.1 and Table 4.2.

Table 4.1. Joint Constructability – UHPC Quality Control Test Results

	Time		Mix Temp	Ambient	Flow	
Batch #	**Start**	**Finish**	**Finish (°F)**	**Temp (°F)**	**Static (in)**	**Dynamic (in)**
1	9:55am	10:10am	85.0	64.0	8.50	9.25
2	10:27am	10:38am	84.0	64.0	9.13	10.00
3	11:07am	11:22am	82.0	66.0	9.13	10.00

Table 4.2. Joint Strength & Serviceability – UHPC Quality Control Test Results

	Time		Mix Temp	Ambient	Flow	
Batch #	**Start**	**Finish**	**Finish (°F)**	**Temp (°F)**	**Static (in)**	**Dynamic (in)**
1*	10:00am	10:23am	100.0	75.5	6.00	N/A
2	11:16am	11:36am	60.0	75.5	9.75	10.00
3	11:48am	12:08pm	60.1	75.6	9.75	10.00
4	12:40pm	1:02pm	60.0	75.6	N/A	N/A

**Batch not used*

UHPC Strength Tests

Abrasion Testing

Twelve four-inch diameter cylinders were tested for compressive strength in accordance with ASTM C39 to establish the maturity of UHPC mix design 1 (Table 3.2) used in the abrasion testing. Compressive strength results from the 4-inch UHPC cylinders are presented in Figure 4.1.

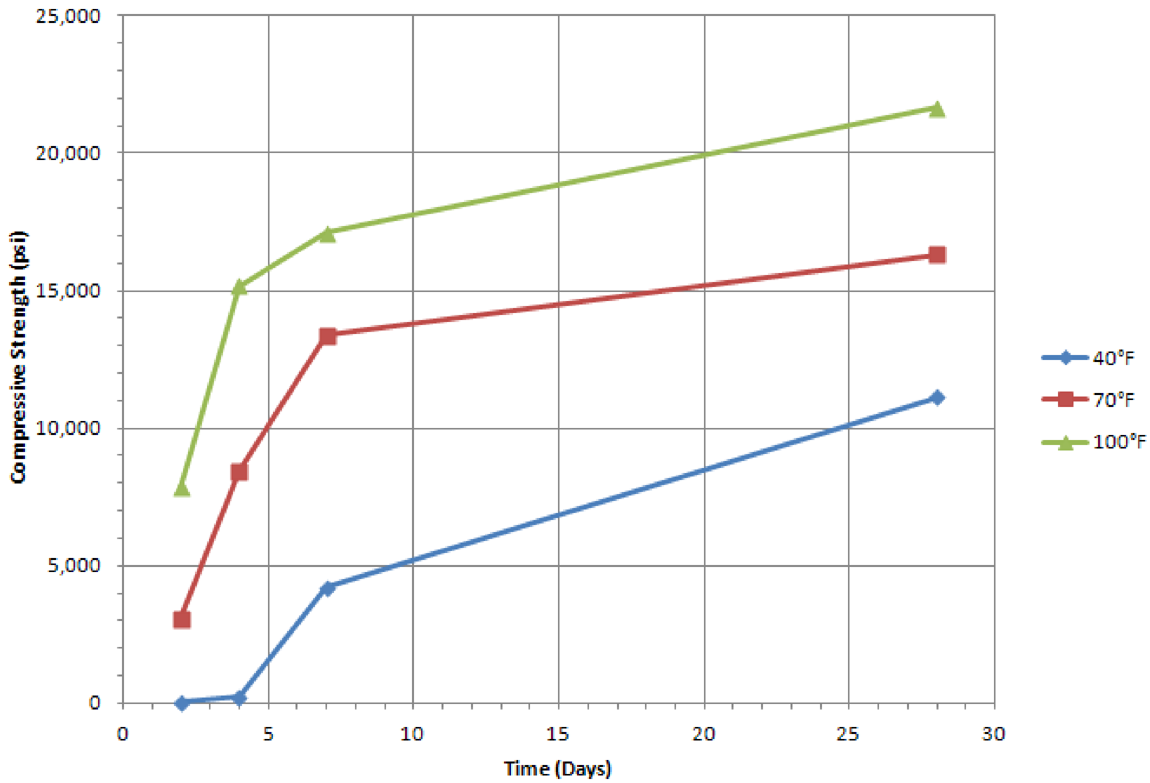


Figure 4.1. Compressive Strength of UHPC Mix Design 1 (Abrasion Testing)

From the compressive tests, the compressive strengths varied for the curing temperatures of 40°F, 70°F, and 100°F. The 28-day compressive strengths (f'_c) for 40°F, 70°F, and 100°F were 11,100 psi, 16,300 psi, and 21,600 psi, respectively.

Joint Constructability Testing

Eighteen four-inch diameter cylinders were tested for compressive strength in accordance with ASTM C39 to establish the maturity of UHPC mix design 2 (Table 3.4) used in the joint constructability testing. Compressive strength results from the 4-inch UHPC cylinders cured at 70°F are presented in Figure 4.2.

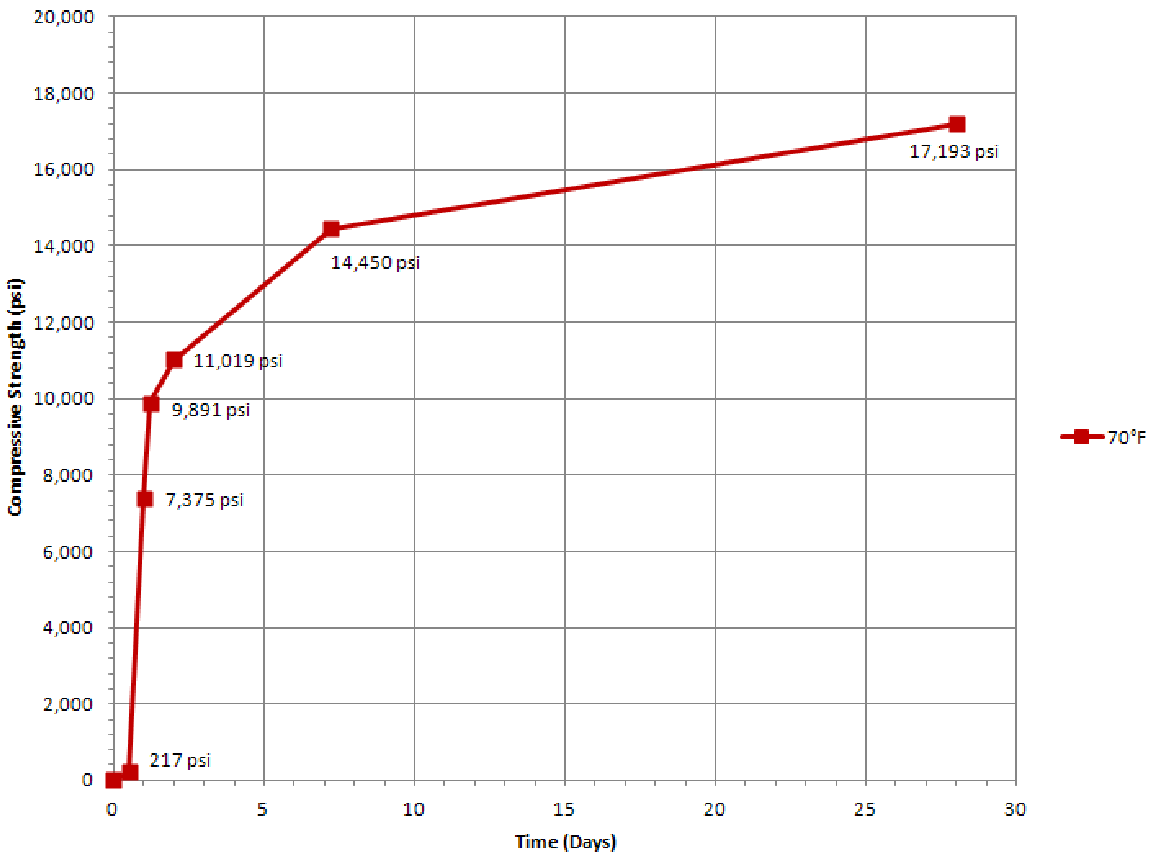
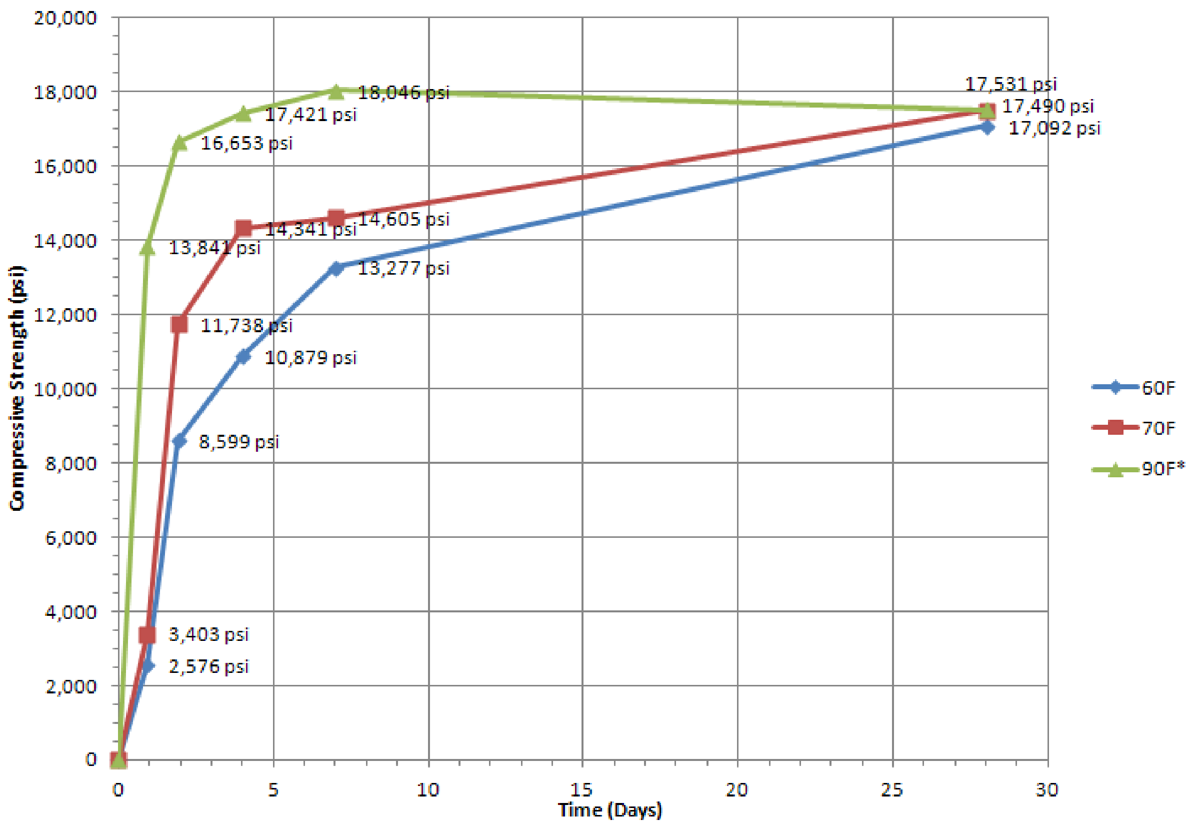


Figure 4.2. Compressive Strength of UHPC Mix Design 2 (Joint Constructability)

Transverse Joint Strength and Serviceability Testing

Forty-five four-inch diameter cylinders were tested for compressive strength in accordance with ASTM C39 to establish the maturity of UHPC mix design 2 used in the transverse joint strength and serviceability testing. Fifteen cylinders each were cured at 60°F, 70°F, and 90°F to replicate potential field curing temperatures and determine strength variations of the mix design. Compressive strength results from the 4-inch UHPC cylinders are presented in Figure 4.3.



** - Due to oven error, 90°F cylinders cured at 130°F for first 24 hours*

Figure 4.3. Compressive Strength of UHPC Mix Design 2 (Strength & Serviceability)

UHPC mix design 2, designed specifically for the SHRP2 Project R04 demonstration bridge and used in the joint constructability and transverse joint strength and serviceability testing, reached 10,000 psi compressive strength in approximately 2 days and 14,000 psi compressive strength in 4 days. The 28-day strength (f'_c) of the UHPC used in the final two testing procedures was approximately 17,000 psi.

HPC Strength Test

Transverse Joint Strength and Serviceability Testing

Twenty-four four-inch diameter cylinders were tested for compressive strength in accordance with ASTM C39 to establish the maturity of Iowa DOT CV-HPC-D mix design used in the prefabricated deck modules for the transverse joint strength and serviceability testing. Compressive strength results from the 4-inch HPC cylinders are presented in Figure 4.4. The 28-day compressive strength (f'_c) of the deck HPC for the prefabricated modules was approximately 5,800 psi.

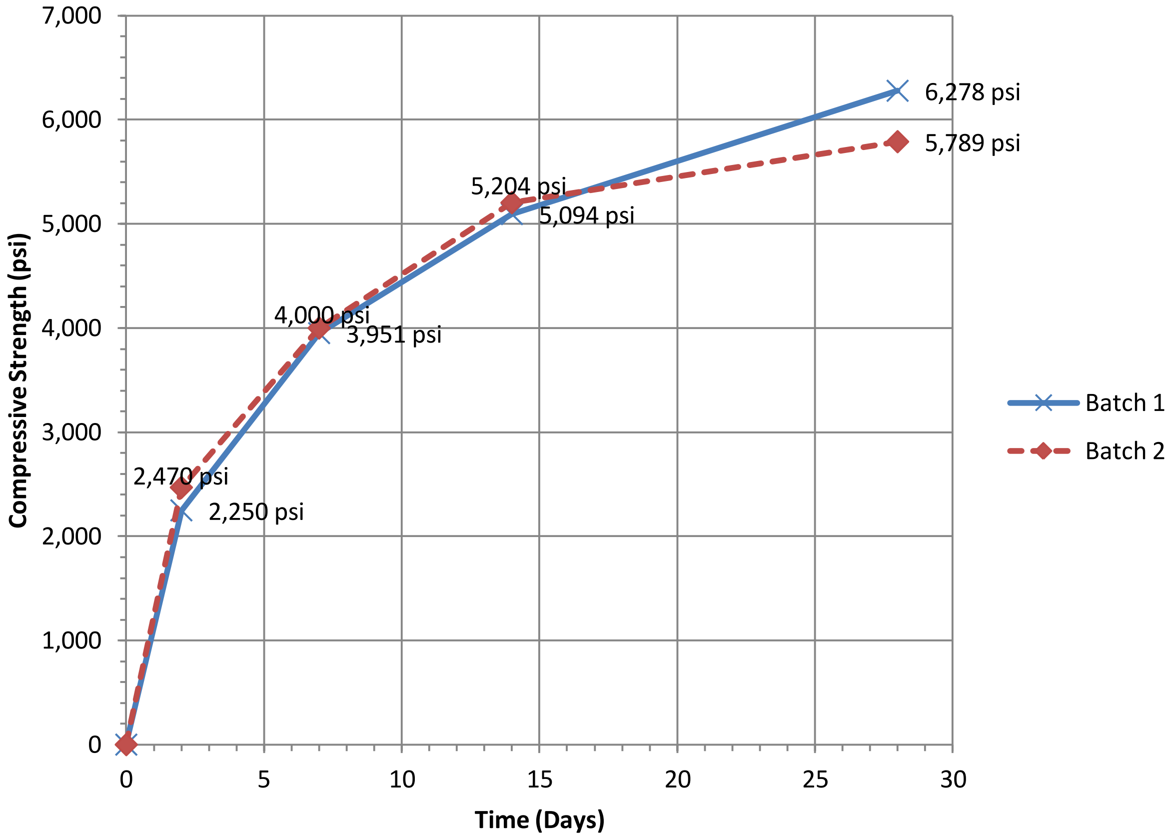


Figure 4.4. Compressive Strength of HPC (Joint Strength & Serviceability)

Flexural Strength Test

Transverse Joint Strength and Serviceability Testing

Three beams were tested in order to determine the modulus of rupture of the HPC used in the prefabricated deck modules. The beams tested had cross sectional dimensions of 6 in by 6 in and a length of 18 in and were tested in accordance with ASTM C78 to obtain the modulus of rupture. Flexural strength results for the 6 in by 6 in by 18 in beams are presented in Table 4.3. The modulus of rupture (f_r) for the HPC used in the deck modules was 439 psi.

Taking the modulus of elasticity (E_c) as $57{,}000\sqrt{f'_c}$ for the HPC, E_c was calculated to be 4,030 ksi (ACI 318-05). Thus, with the modulus of rupture of 439 psi, the expected cracking strain for the precast HPC deck was 110με.

Because no flexural strength testing was completed on the UHPC material, the modulus of elasticity was calculated as $46{,}200\sqrt{f'_c}$ (Graybeal 2007) which fell within Berg's E_c range of 5,800 to 7,800 ksi, allowing the modulus of rupture to be approximated as 1,855 psi (Berg 2010). The expected cracking strain of the UHPC was then calculated to be approximately 250με.

Table 4.3. Joint Strength & Serviceability – HPC Flexural Strength

Specimen ID	Max. Applied Load P (lbs)	Span Length L (in)	Width @ Fracture b (in)	Depth @ Fracture d (in)	Modulus of Rupture R (psi)
B1-28	5301	18	6	6	441.75
B2-28	5433	18	6	6	452.75
B3-28	5081	18	6	6	423.42
				Modulus of Rupture	**439.31**

UHPC ABRASION TESTING

Abrasion testing was completed on the UHPC material in order to determine the early age grindability of the joints in the demonstration bridge. Testing of the UHPC material for abrasion resistance was completed at Iowa State University in February and March 2011.

Abrasion Test Results

Twelve cylinders were cut into 36 specimens resulting in 3 different surface finishes and subjected to abrasion resistance testing in accordance with ASTM C944. Results of the abrasion resistance testing at 2, 4, 7, and 28 days are presented in Table 4.5 through Table 4.8. The specimen identification matrix and identification terminology is presented in Table 4.4.

Table 4.4. Abrasion Specimen Identification Matrix

	2-Day	4-Day	7-Day	28-Day
A: 40° F	A2-1	A4-1	A7-1	A28-1
	A2-2	A4-2	A7-2	A28-2
	A2-3	A4-3	A7-3	A28-3
	A2-4	A4-4	A7-4	A28-4
	A2-5	A4-5	A7-5	A28-5
B: 70°F	B2-1	B4-1	B7-1	B28-1
	B2-2	B4-2	B7-2	B28-2
	B2-3	B4-3	B7-3	B28-3
	B2-4	B4-4	B7-4	B28-4
	B2-5	B4-5	B7-5	B28-5
C: 100° F	C2-1	C4-1	C7-1	C28-1
	C2-2	C4-2	C7-2	C28-2
	C2-3	C4-3	C7-3	C28-3
	C2-4	C4-4	C7-4	C28-4
	C2-5	C4-5	C7-5	C28-5

Example: A2-1
"A" - Curing temperature
"2" - Days after pour in which test occurs
"1" - Specimen test #

Notes:

-*Specimen tests # 1 - 4 are the rough, cut, or formed surface abrasion resistance tests*

-*Specimen test # 5 is the compressive strength test*

Table 4.5. 2-Day Abrasion Test Results

Specimen Age:	**2 Days**	*ASTM C 944: Abrasion Resistance of Concrete Surfaces by Rotating Cutter Method*
Test Date:	*2/24/2011*	

Specimen ID	Surface	Initial Mass	Mass 1	Mass 2	Final Mass	Wear Depth	Loss of Mass		Additional Notes
		g	*g*	*g*	*g*	*mm*	*g*	*%*	
A2-1	NA						0.00	0.00%	**too soft to test*
A2-2	NA						0.00	0.00%	**too soft to test*
A2-3	NA						0.00	0.00%	**too soft to test*
A2-4	NA						0.00	0.00%	**too soft to test*
B2-1	rough	1993.10	1991.08	1989.31	1987.85	0.41	5.25	0.26%	
B2-2	cut	1987.85	1986.63	1985.05	1983.48	0.76	4.37	0.22%	
B2-3	cut	2020.20	2017.20	2015.90	2014.10	0.59	6.10	0.30%	
B2-4	form	2016.88	2014.01	2008.08	2002.86	1.25	14.02	0.70%	
C2-1	rough	1951.30	1950.57	1949.88	1949.40	0.48	1.90	0.10%	
C2-2	cut	1949.37	1948.48	1948.22	1947.95	0.26	1.42	0.07%	
C2-3	cut	2016.96	2016.73	2016.52	2016.23	0.23	0.73	0.04%	
C2-4	cut	1771.24	1770.67	1770.26	1769.93	0.26	1.31	0.07%	

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Table 4.6. 4-Day Abrasion Test Results

Specimen Age:	**4 Days**	*ASTM C 944: Abrasion Resistance of Concrete Surfaces by Rotating Cutter Method*
Test Date:	*2/26/2011*	

Specimen ID	**Surface**	**Initial Mass**	**Mass 1**	**Mass 2**	**Final Mass**	**Wear Depth**	**Loss of Mass**		**Additional Notes**
		g	*g*	*g*	*g*	*mm*	*g*	*%*	
A4-1	rough	1454.66	1336.17	N/A	1336.17	7.29	118.49	8.15%	**Maxed out at 55 sec*
A4-2							0.00	0.00%	**too soft to cut cylinder*
A4-3							0.00	0.00%	**too soft to cut cylinder*
A4-4	form	2242.74	2121.95	N/A	2121.95	6.96	120.79	5.39%	**Maxed out at 68 sec*
B4-1	rough	1841.91	1841.25	1840.79	1840.36		1.55	0.08%	
B4-2	cut	1840.36	1840.14	1839.96	1839.78		0.58	0.03%	
B4-3	cut	2004.69	2004.54	2004.39	2004.24		0.45	0.02%	
B4-4	form	2004.24	2002.23	1999.73	1997.95		6.29	0.31%	
C4-1	cut	1779.75	1779.6	1779.51	1779.38		0.37	0.02%	
C4-2	cut	1779.38	1779.25	1779.17	1779.12		0.26	0.01%	
C4-3	cut	2094.86	2094.77	2094.61	2094.5		0.36	0.02%	
C4-4	form	2094.5	2094.3	2093.89	2093.39		1.11	0.05%	

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Table 4.7. 7-Day Abrasion Test Results

Specimen Age:	**7 Days**	*ASTM C 944: Abrasion Resistance of Concrete Surfaces by Rotating Cutter Method*							
Test Date:	*3/1/2011*								
Specimen ID	**Surface**	**Initial Mass**	**Mass 1**	**Mass 2**	**Final Mass**	**Wear Depth**	**Loss of Mass**		**Additional Notes**
		g	*g*	*g*	*g*	*mm*	*g*	*%*	
A7-1	rough	2128.75	2120.59	2116.72	2113.48		7.11	0.34%	
A7-2	cut	2113.48	2112.62	2111.56	2110.63		1.99	0.09%	
A7-3	cut	1955.9	1955.23	1954.52	1953.82		1.41	0.07%	
A7-4	form	1953.82	1951.01	1946.51	1943.64		7.37	0.38%	
B7-1	rough	1841.96	1839.46	1838.7	1838.17		1.29	0.07%	
B7-2	cut	1838.17	1837.96	1837.84	1837.75		0.21	0.01%	
B7-3	cut	2162.22	2162.15	2162.09	2162.02		0.13	0.01%	
B7-4	form	2162.02	2161.65	2161.09	2160.53		1.12	0.05%	
C7-1	rough	1983.47	1981.9	1981.03	1980.4		1.50	0.08%	
C7-2	cut	1980.4	1980.33	1980.22	1980.15		0.18	0.01%	
C7-4	cut	2103.96	2103.88	2103.84	2103.77		0.11	0.01%	
C7-3	form	2103.77	2103.63	2103.38	2103.12		0.51	0.02%	

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Table 4.8. 28-Day Abrasion Test Results

Specimen Age:	28 Days	ASTM C 944: Abrasion Resistance of Concrete Surfaces by Rotating Cutter Method						
Test Date:	3/22/2011							

Specimen ID	Surface	Initial Mass	Mass 1	Mass 2	Final Mass	Wear Depth	Loss of Mass		Additional Notes
		g	*g*	*g*	*g*	*mm*	*g*	*%*	
A28-1	rough	1700.7	1699.1	1698.8	1698.5		2.20	0.13%	
A28-2	cut	1698.5	1698.4	1698.2	1698.1		0.40	0.02%	
A28-3	cut	1855.3	1855.2	1855.1	1854.9		0.40	0.02%	
A28-4	form	1854.9	1854.5	1853.9	1853.4		1.50	0.08%	
B28-1	rough	1959.8	1959	1958.6	1958.3		1.50	0.08%	
B28-2	cut	1958.3	1958.1	1958	1957.9		0.40	0.02%	
B28-3	cut	2020.6	2020.4	2020.3	2020.2		0.40	0.02%	
B28-4	form	2020.2	2019.1	2017.8	2016.4		3.80	0.19%	
C28-1	rough	1985.4	1984.7	1984.4	1983.7		1.70	0.09%	
C28-2	cut	1803.7	1803.3	1803.2	1803.1		0.60	0.03%	
C28-3	cut	2054.1	2053.7	2053.3	2053.1		1.00	0.05%	
C28-4	form	2053.1	2053	2052.9	2052.6		0.50	0.02%	

SHRP2 Project No R04 - Phase III - Task 10C: Test 2

Taking the maturity of the UHPC into consideration, a plot of the percent mass loss vs. compressive strength for the three different surface finish conditions is presented in Figure 4.5.

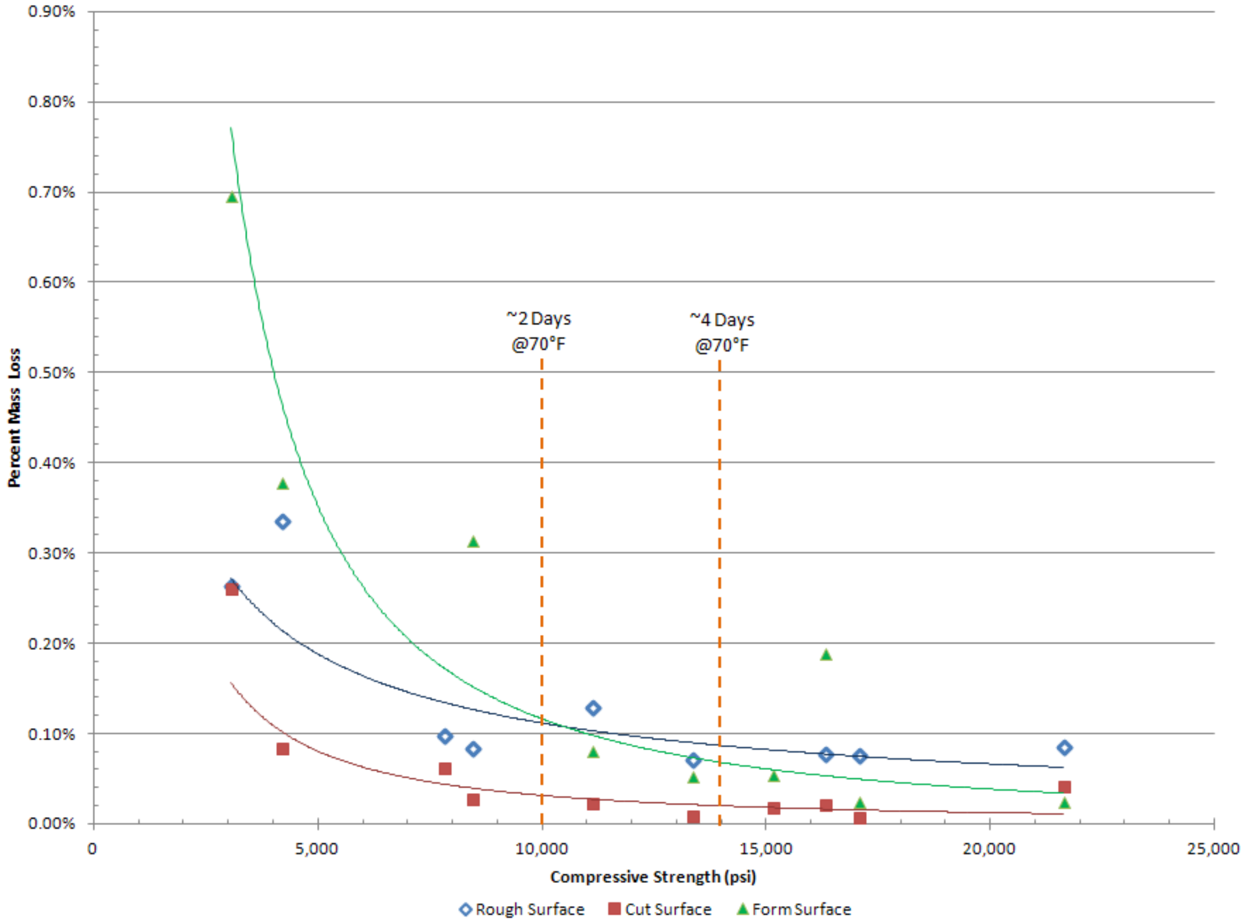

Figure 4.5. Abrasion Testing – Percentage Mass Loss vs. Strength

Based on the compressive strength test results for the SHRP2 Project R04 UHPC mix design used in the constructability and strength and serviceability testing, mix design 2, the UHPC will reach the 10,000 psi compressive strength required for grinding in the project specifications for the demonstration bridge at approximately 2 days if cured at 70°F. The 14,000 psi compressive strength threshold, required in the demonstration bridge project specifications for opening the bridge to traffic, will likely be reached 4 days after placement. Thus, the contractor will have roughly 2 days to perform grinding of the joints from the time the 10,000 psi threshold is reached prior to opening of the bridge to traffic at 14,000 psi compressive strength. The percentage mass loss for both formed and top finishes at the 10,000 psi compressive strength threshold is approximately 0.12%. At 14,000 psi compressive strength of the UHPC mix, the percentage mass loss is approximately 0.07%. Over that 2-day duration of time, the UHPC's resistance to abrasion increases by approximately 40 percent. That would be a significant factor for the contractor in terms of grinding time and accelerated scheduling.

From Figure 4.5, it can be seen that the formed surface and rough surface finishes displayed the lowest abrasion resistance. Specimens with formed surface finishes exhibited lower abrasion resistance than cut surfaces because of the steel fibers present in the UHPC. At the formed surface, the steel fibers were aligned preferentially, parallel with the surface. Thus, the fibers tended to pull off easily. The fibers lay parallel with the form surface because as the UHPC flowed along the bottom of the form, the fibers tended to align and lay flat. The rough surface finish generally included small entrapped air bubbles which allowed for easier removal of the UHPC material. As was expected, the cut surface finish had the highest abrasion resistance. Because the cast-in-place UHPC joints in the SHRP2 Project R04 demonstration bridge are a plywood top formed surface, the abrasion resistance in the field is expected to most nearly resemble that of the formed surface finish seen in the abrasion tests.

JOINT CONSTRUCTABILITY TESTING

Joint constructability testing was completed in order to qualitatively evaluate the intersecting, cast-in-place UHPC deck joints to be used in the demonstration bridge. Specifically, the full scale mock-up of the intersection between one longitudinal and one transverse UHPC deck joint was constructed to investigate issues relating to casting sequence, material mixing and placement rates, effects of ambient temperature on construction, flow characteristics of the UHPC, and consolidation of material at congested locations. Testing of the UHPC joints for constructability was completed at Iowa State University in April 2011.

Constructability Test Results

Casting Sequence

The original proposal for the construction sequence of the demonstration bridge outlined continuous placement of the entire grid of UHPC deck joints (longitudinal and transverse). Through discussions with the engineer, contractor, and material supplier, several logistical issues arose which challenged the feasibility of full deck continuous placement. Typical mixers used by Lafarge Canada for UHPC placement mix 5.11 ft^3 per batch. On the jobsite, the mixers are used in pairs in order to provide a continuous supply of UHPC. Each batch is then discharged into buggies and transported onto the bridge to the placement location.

With the large volume of UHPC necessary in the bridge deck joints, continuous placement could only be achieved using a large number of mixers and laborers. Without employing many mixers and laborers, cold joints could potentially form in the UHPC deck joints. Instead, stay-in-place acrylic vertical bulkheads were proposed by Lafarge to control the location of potential cold joints.

A new construction sequence, which limited continuous placement to the transverse joints and allowed vertical cold joints in the longitudinal joints, was suggested for the joint constructability testing and demonstration bridge as a result of these discussions. A prototype of the stay-in-place acrylic vertical bulkheads (Figure 3.7) was fabricated and used during the joint constructability testing so its performance could be evaluated. The acrylic vertical bulkheads successfully acted to limit the placement of UHPC to the transverse joint.

Ambient Temperature Effects on UHPC

The extent of the susceptibility to variations in temperature for the workability and flow characteristics of the UHPC mix design was observed during batching of the joint constructability test specimen and the transverse joint strength and serviceability test specimen to follow. Ambient air temperatures, seen previously in Table 4.1, were steady at around 65°F at the time of batching for the intersecting joint specimen. However, during the batching for the transverse joint strength and serviceability specimen, ambient temperatures were 75.5°F (Table 4.2). Without compensating for the change in ambient air temperature, the workability and flow characteristics of the mixes were much different.

When ambient temperatures were 65°F, the temperature of the UHPC upon discharge from the mixer ranged from 82 to 85°F for the intersecting joint specimen's three batches. Within this range, the UHPC had acceptable flow characteristics for placement. The temperature of the UHPC upon discharge from the mixer for ambient temperatures around 75.5°F was over 100°F. At this ambient temperature, the UHPC never reached its anticipated flow characteristics in the mixer, thus the batch was rejected. Flow characteristics were inadequate at an ambient temperature of 75.5°F. To correct the issue, water in the mix design was replaced by mass with ice and the UHPC material temperature was reduced. When ice was used, the anticipated flow and workability characteristics were noted and the batches could be successfully discharged and placed. The temperature upon discharge from the mixer while utilizing ice was 60°F. This modification, the replacement of water by mass with ice, enabled extended working time and improved the flow relative to the previous batch.

Flow Characteristics and Consolidation of UHPC

Evaluating the flow of the UHPC around the corners at the intersection of the longitudinal and transverse deck joints was a critical aspect for this test. Adequate consolidation of the UHPC in the joint cross section around steel reinforcement is important to the deck joint performance. During UHPC placement, when final mix temperature was limited to a maximum of 85°F, the UHPC material appeared to have adequate flow characteristics to achieve good consolidation and flow around corners at the intersections of longitudinal and transverse joints (Figure 4.8).

After the specimen was cured and removed from the forms, it was cut into several sections to examine consolidation and potential cold joints. Upon investigation of the cut specimen, no significant voids around steel reinforcing bars were observed (Figure 4.6 & Figure 4.7).

Figure 4.6. Section of Transverse Joint (1)

Figure 4.7. Section of Transverse Joint (2)

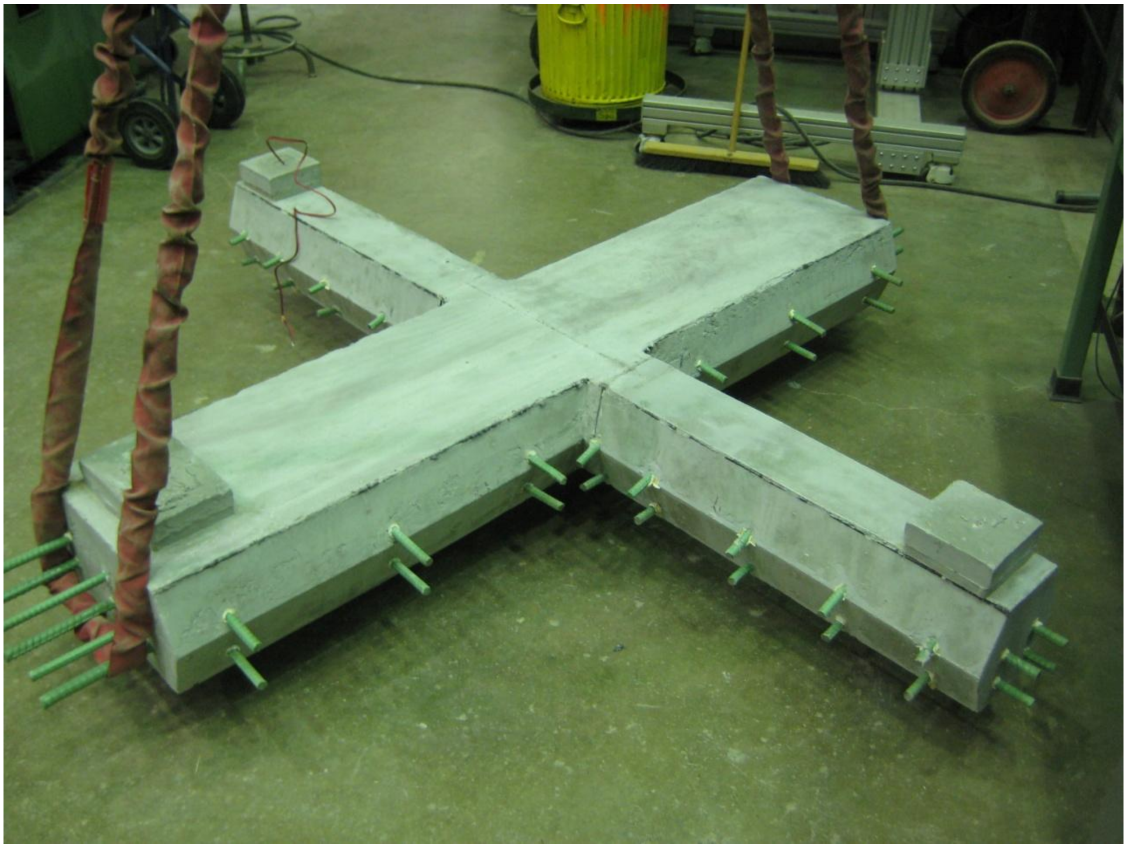

Figure 4.8. Joint Intersection Specimen

The test also validated the use of top forms and chimneys at the high end of the two percent cross slope of the bridge deck at transverse joints. The top forms were applied sequentially as the joint was filled from the lowest elevation to the highest. The chimneys, seen in Figure 3.9 and Figure 4.8, provide additional hydrostatic head in the freshly placed UHPC to aid in consolidation within the joint. It was proposed that top forms and chimneys be used in the demonstration bridge. Instead, three-quarter inch high spacer boards were placed below the top forms to build up small hydrostatic head and produce similar results.

Joint Intersection Detail Recommendations

Final inspection of the specimen upon removal from the forms allowed for additional observations and recommendations. The proposed stay-in-place acrylic bulkhead successfully allowed for sequential placement of the UHPC, but also created a possible infiltration plane where water and chemicals could access the embedded steel joint reinforcement (Figure 4.9).

Figure 4.9. Stay-in-Place Acrylic Bulkhead

To maintain sequential placement of UHPC in the deck joint grid and avoid possible infiltration planes, a detail for a partial-height, removable acrylic bulkhead was developed and suggested for use in the demonstration bridge (Figure 4.10). The removable acrylic bulkheads should be used in the longitudinal joint, in compression zones where possible (Figure 4.11). Placing the bulkheads at those locations will provide better continuity at the interface between the hardened and freshly placed UHPC which will help to prevent the ingress of water and other chemicals. In addition, the placement sequence of the UHPC (Figure 4.11) will be controlled starting at the lowest elevations through the transverse joints over the piers up to the bulkheads. The center span UHPC joints will be placed last.

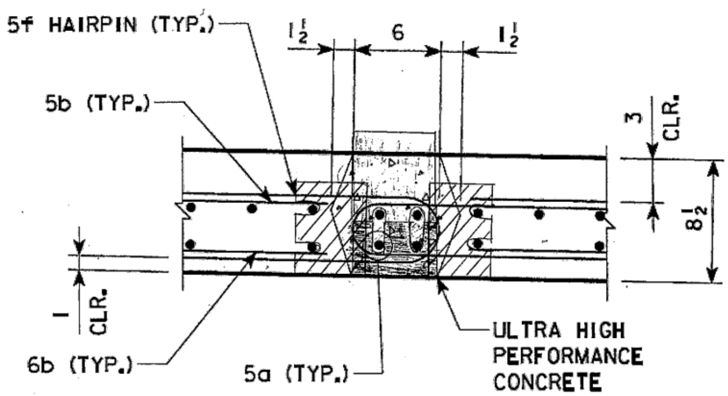

Figure 4.10. Removable Acrylic Bulkhead

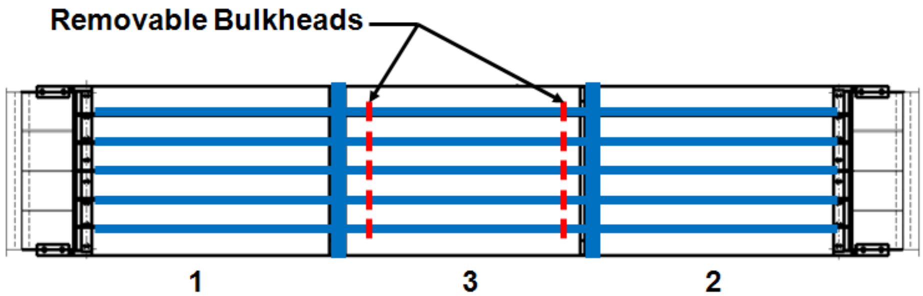

Figure 4.11. Proposed Placement Plan

TRANSVERSE JOINT STRENGTH AND SERVICEABILITY TESTING

Strength and serviceability testing of the module-to-module transverse connection for the SHRP2 Project R04 demonstration bridge was performed in order to evaluate the negative bending performance of this detail over the piers, determine its cracking moment, and verify the ultimate moment capacity. Testing of the module-to-module transverse connection was completed at Iowa State University from July to October 2011.

Results Terminology

Due to the orientation of the testing specimen in the laboratory, the driving surface, or top of deck surface, was located on the bottom of the specimen and the bottom of deck surface was located on the top of the specimen.

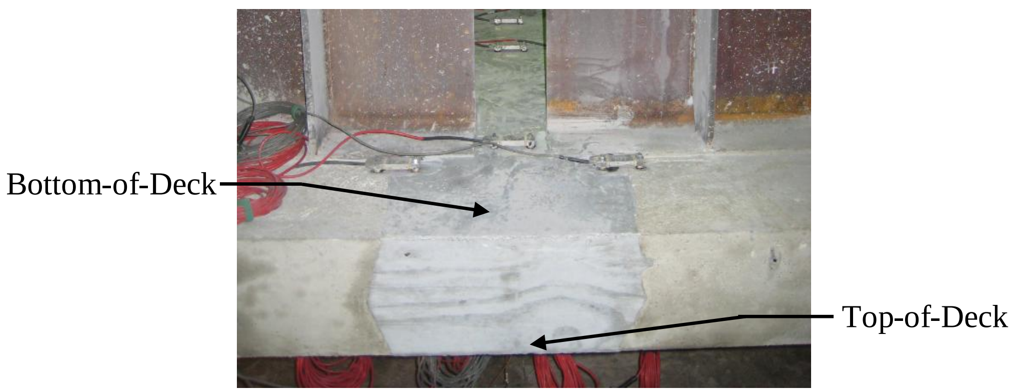

Figure 4.12. Deck Surface Terminology Photograph

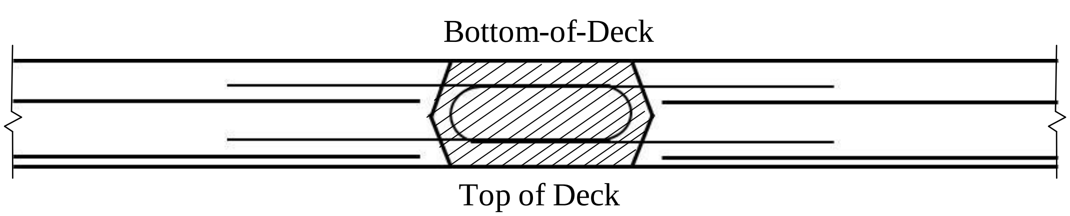

Figure 4.13. Deck Surface Terminology Diagram

Refer to Figure 3.15 and Figure 3.16, which display the locations of all instrumentation for the test specimen presented in the following sections.

Service Level Static Test Results

Load testing through live load Service Levels I and II moment was completed for the specimen (Figure 4.14). The range of expected service level moments for the module connection varied from +74 to -538 k-ft. Loading was completed at 5,000 lbf increments in order to complete visual inspection of the specimen and check for the appearance of cracks and accrual of damage.

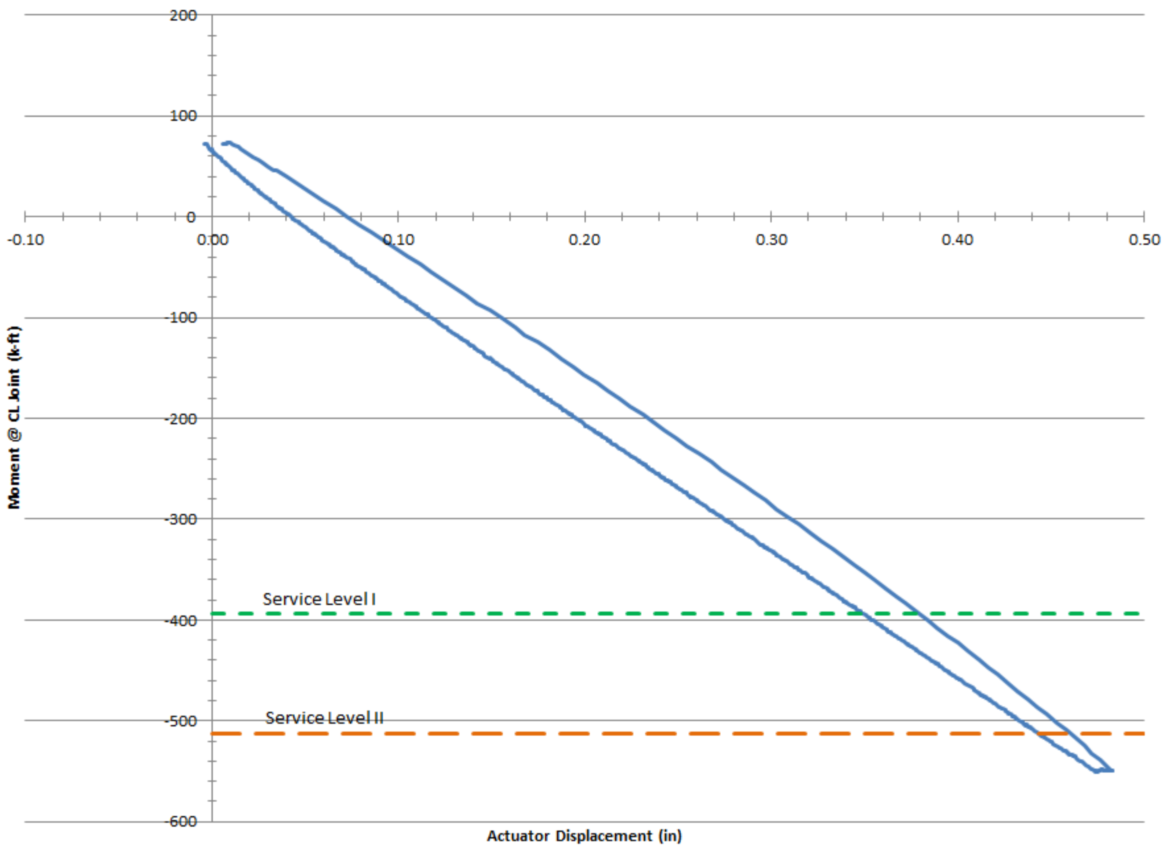

Figure 4.14. Applied Moment vs. Actuator Displacement

Strain levels were monitored through the embedded and surface mounted strain gages located throughout the specimen (Figure 3.15 & Figure 3.16). Strain levels for surface mounted strain gages at locations which spanned the HPC/UHPC interface exceeded 110με, the HPC cracking strain, at approximately halfway to Service Level I moment (Figure 4.15).

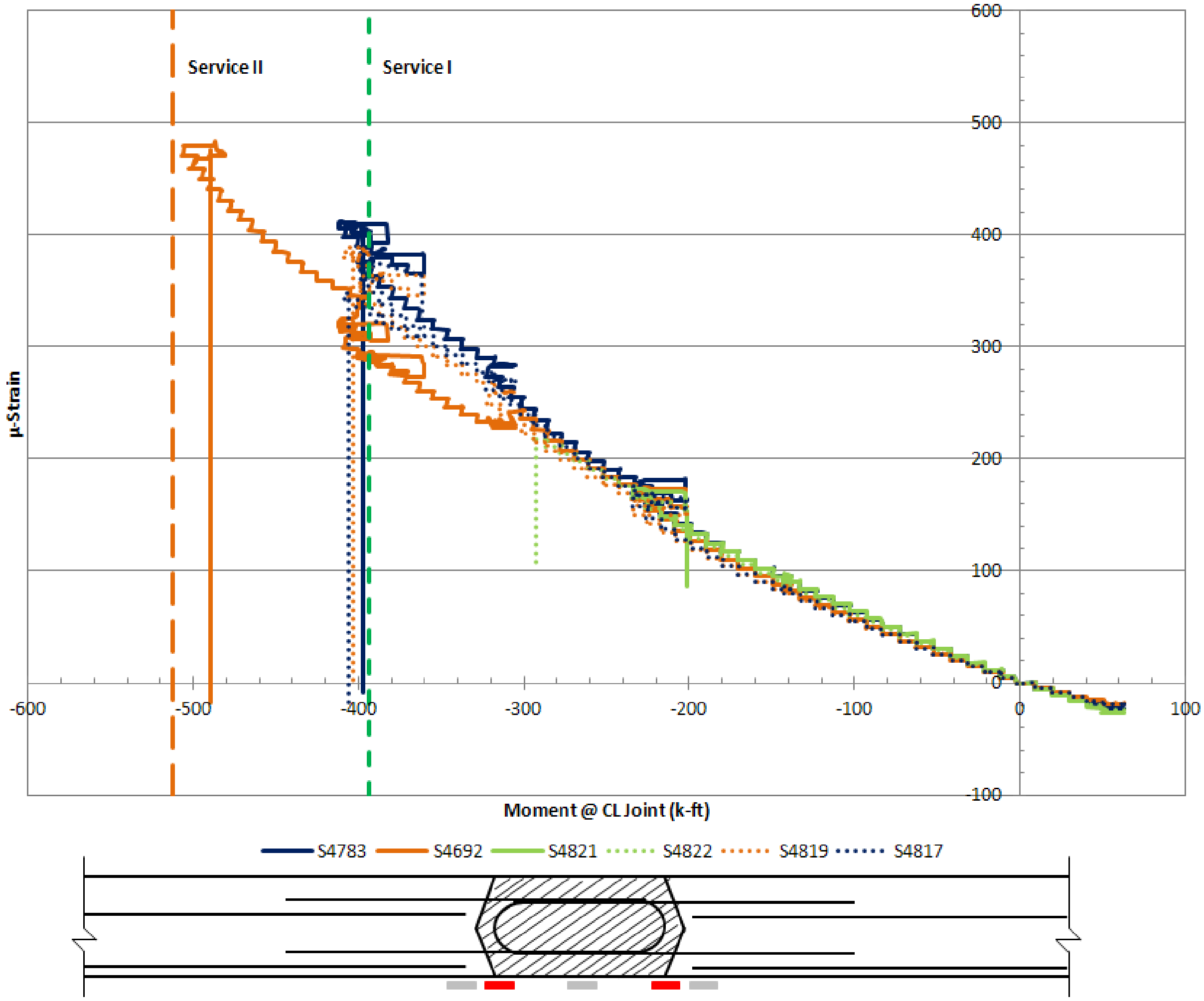

Figure 4.15. Top-of-Deck Surface Mounted Strain Gages over the Joint Interface

Selecting only one longitudinal line of surface mounted strain gages, it was seen that immediately adjacent to the gages spanning the interface, surface strain levels were well below the HPC cracking strain (Figure 4.16). Surface mounted strain gages at these locations registered negligible strains throughout. The disparity between immediately adjacent gages and the strains registering in excess of the HPC cracking strain across the interface suggested debonding and an opening at the interface between the precast HPC deck and the UHPC joint. It should be noted that the intent of the design for the demonstration bridge was to avoid all cracking in the deck at the transverse joint over the pier, as they would be detrimental to the durability of the deck.

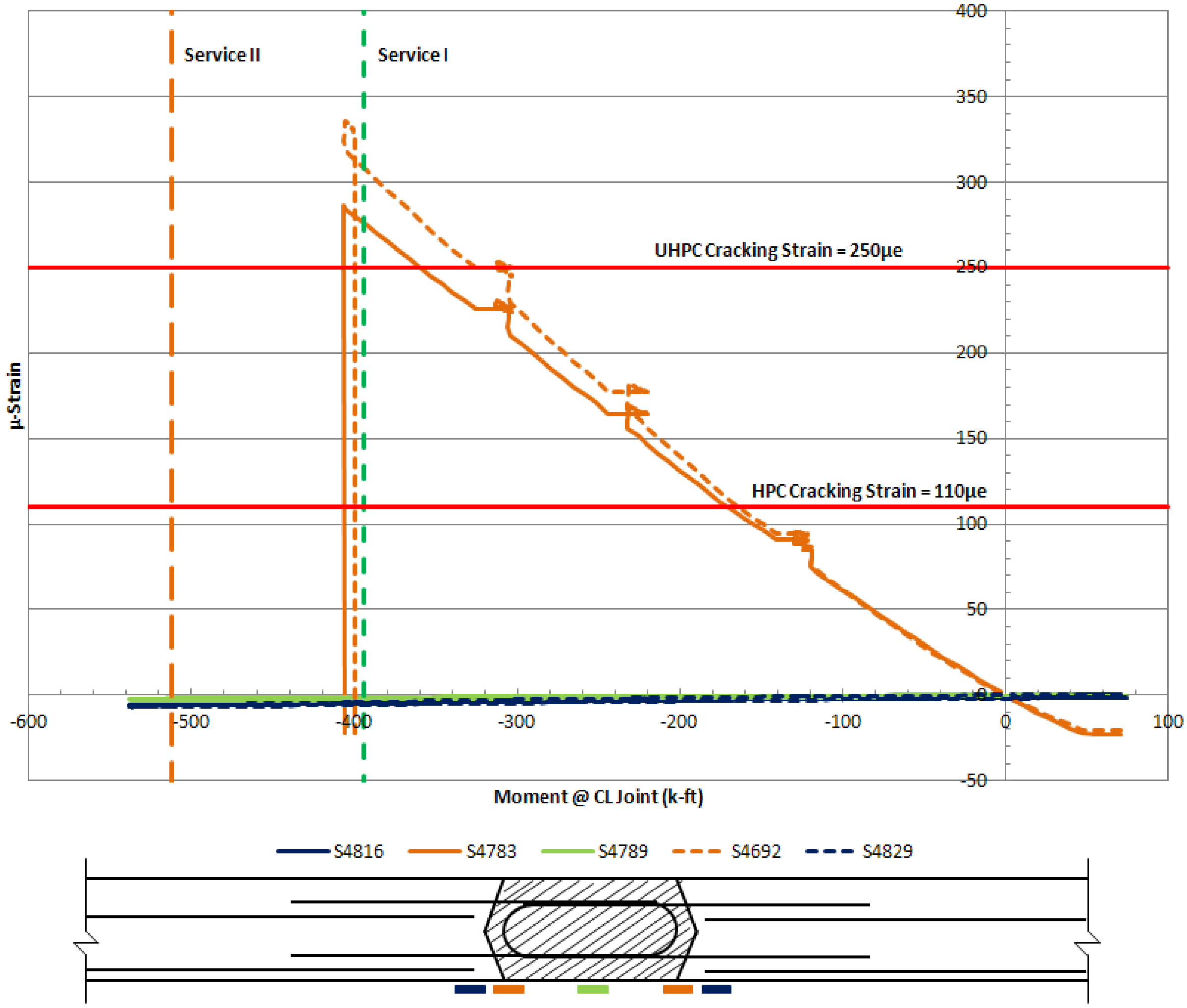

Figure 4.16. Selected Surface Mounted Strain Gages Adjacent to the Joint Interface

Visual inspection of the joint interface at Service Level II confirmed the debonding and substantial opening of the interface suggested in the strain gage data (Figure 4.17). Later, inspection during fatigue testing further confirmed the interfacial debonding and opening occurring below service level conditions.

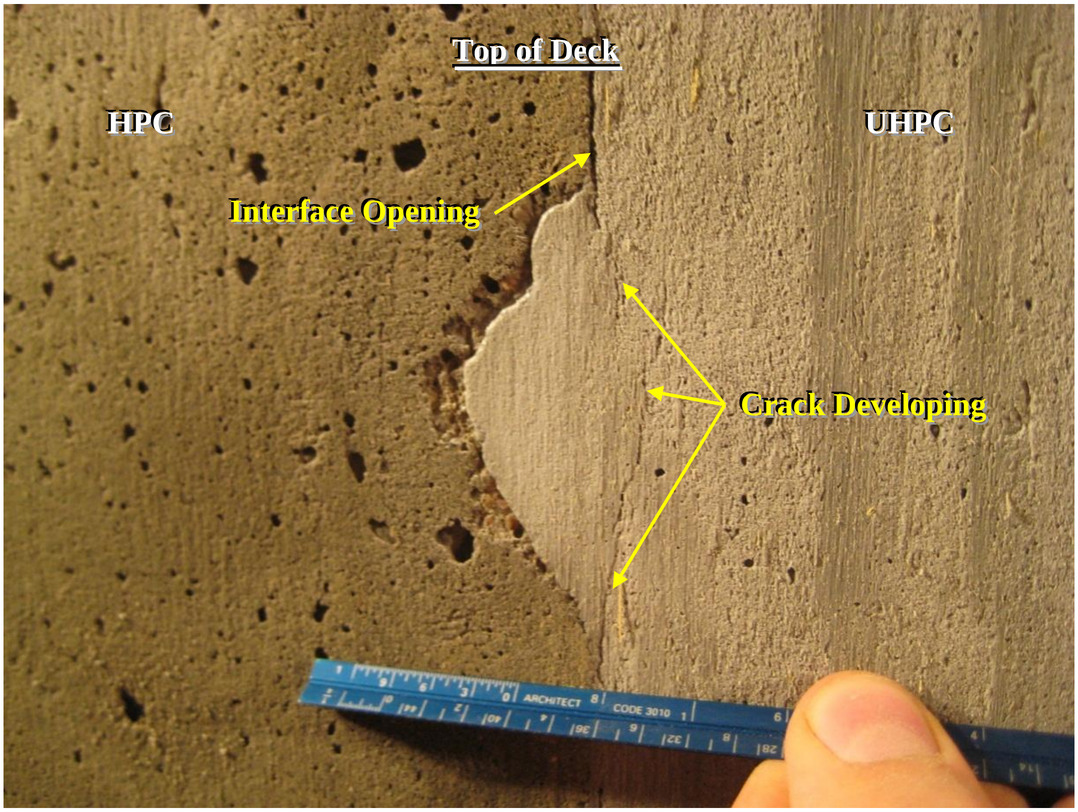

Figure 4.17. Joint Interface Opening

In addition to joint interface debonding and substantial opening, strain levels in embedded strain gages registered above the expected HPC cracking strain as well. Figure 4.19 through Figure 4.21 show the embedded strain gage data for top-of-deck gages along longitudinal reinforcement under the two girder lines in the specimen. Figure 4.22 through Figure 4.24 display strain data for bottom-of-deck gages along the same longitudinal reinforcement lines. Embedded strain gage locations and identifications along with row groupings are shown in Figure 4.18.

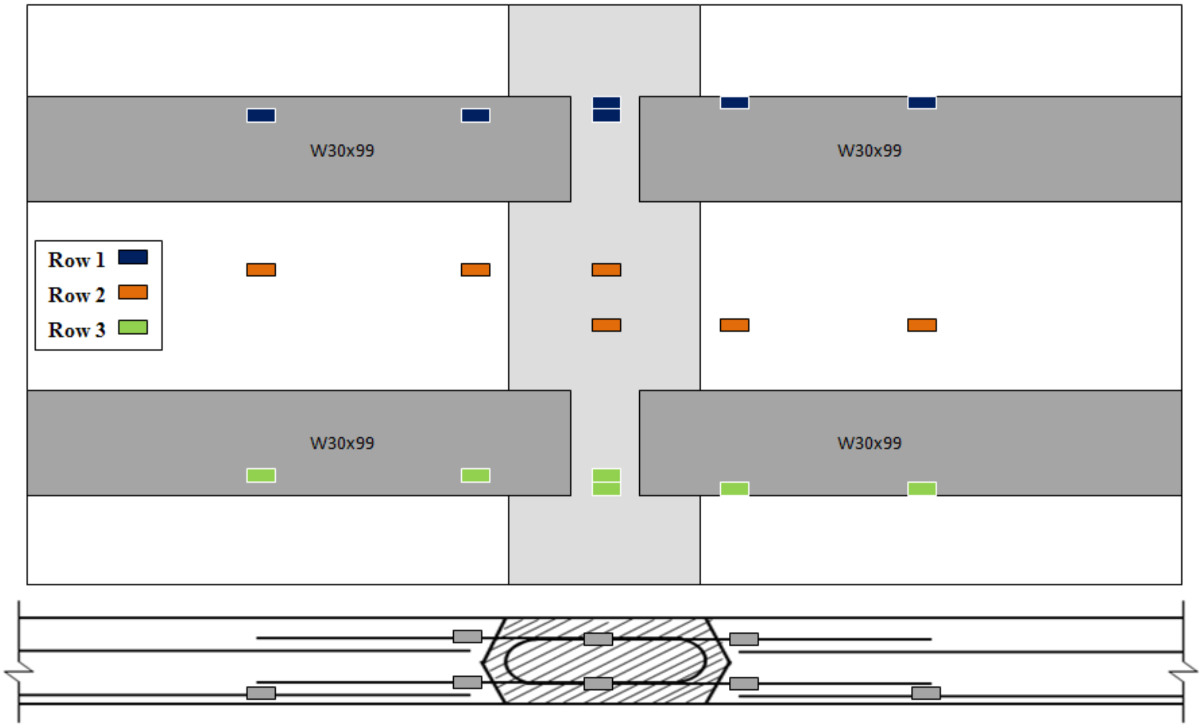

Figure 4.18. Embedded Strain Gage Location and Identification

At the top of deck, these groupings of embedded strain gages show the varying strain seen within the UHPC joint, near the joint interface, and at the hairpin bar termination location three feet from the interface (Figure 4.19 - Figure 4.21). As observed with the surface mounted strain gages, the embedded gages near the interface all exceed the HPC cracking strain prior to reaching Service Level I conditions. In the top-of-deck reinforcement, maximum strains of 540µe, 550µe, and 475µe were recorded in S1-1-1T, S2-2-2T, and S2-3-2T, respectively. Strain in the UHPC joint (J1 and J2 gages) were relatively lower in the top-of-deck, not exceeding 160µe which is below the expected UHPC cracking strain level of 250µe. Nearly all gages located at the termination of the joint hairpin bar registered strain levels exceeding 110µe prior to the Service Level II conditions. This data suggested cracking in the prefabricated HPC deck modules under service level loading. Cracking was not visually confirmed near the joint in the HPC deck during the incremental static loading, but opening and closing of the cracks during cyclic loading made cracking in the HPC clearly visible.

The groupings of embedded strain gages for the bottom-of-deck reinforcement were located within the UHPC joint and near the joint interface only (Figure 4.22 - Figure 4.24). Results similar to those in the top-of-deck were observed. Maximum strains of 460με, 520με, and 420με were registered near the joint interface reinforcement in the bottom-of-deck at S1-3-1B, S1-2-1B, and S1-1-1B, respectively. Localized prying effects of the girders could potentially be responsible for the higher strain levels in the bottom-of-deck reinforcement for the UHPC joint in rows 1 and 3. Strains at two of those locations were observed exceeding the UHPC cracking strain at or before reaching the Service Level II condition.

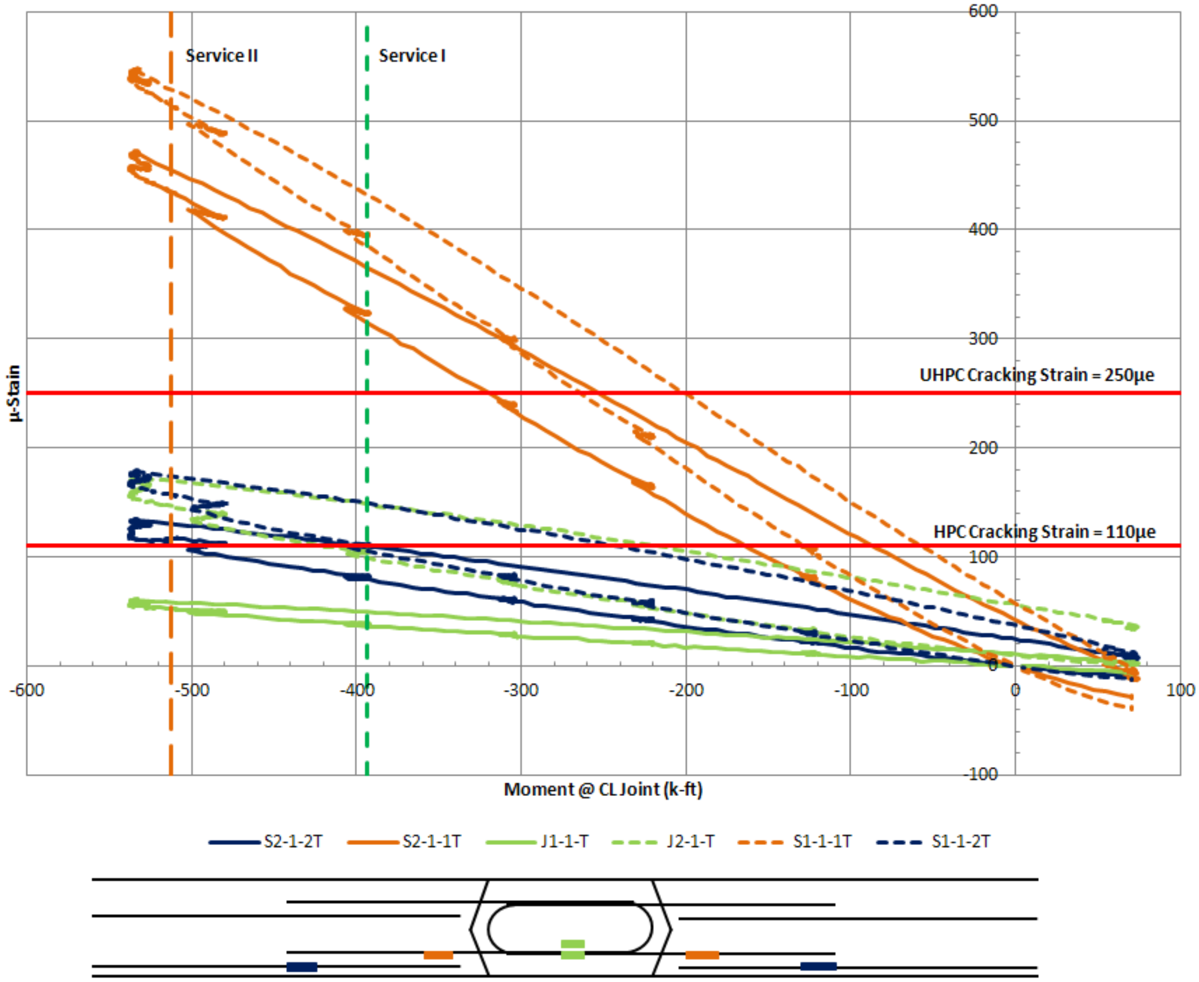

Figure 4.19. Row 1, Top-of-deck Embedded Strain Gages (Static)

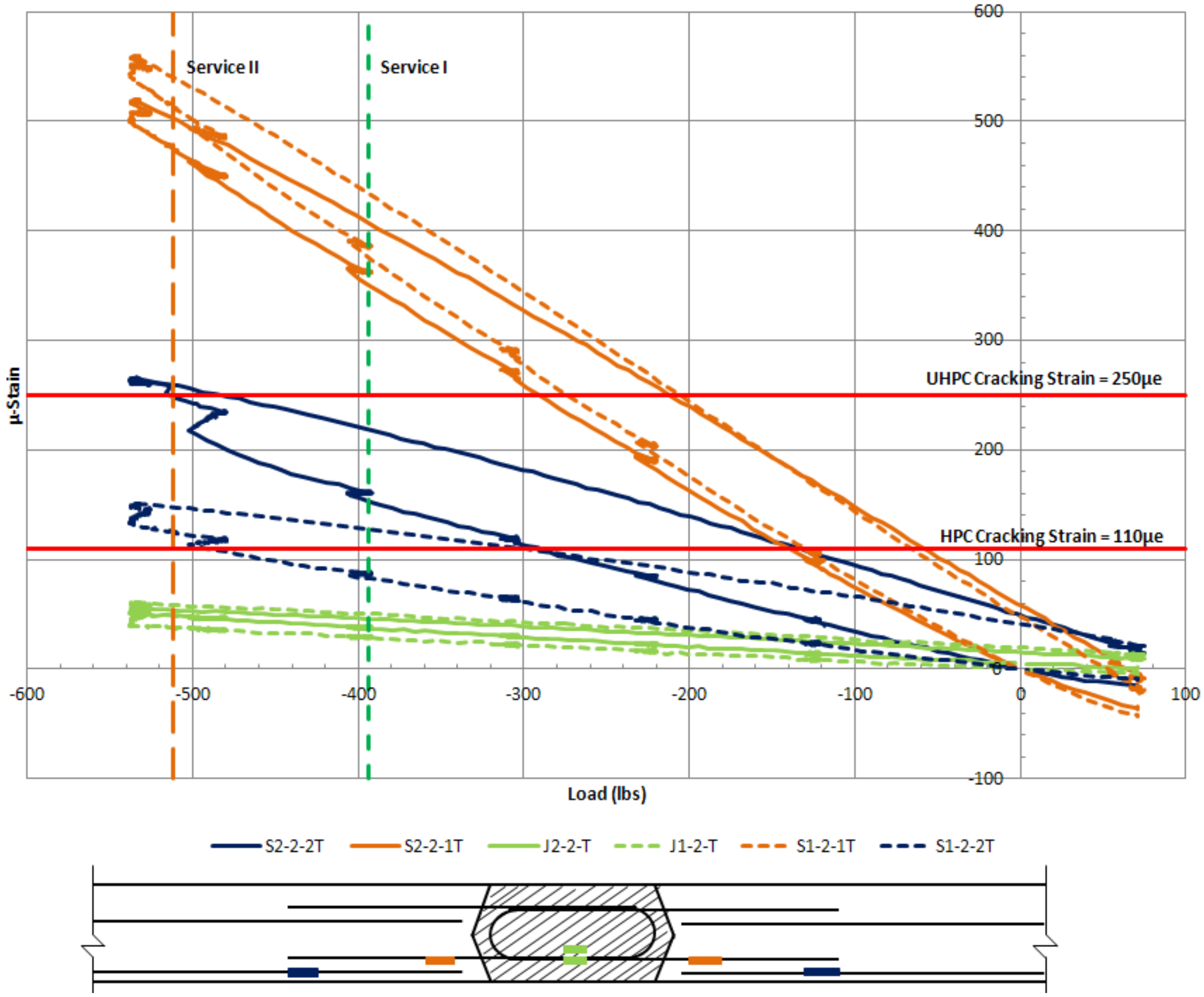

Figure 4.20. Row 2, Top-of-deck Embedded Strain Gages (Static)

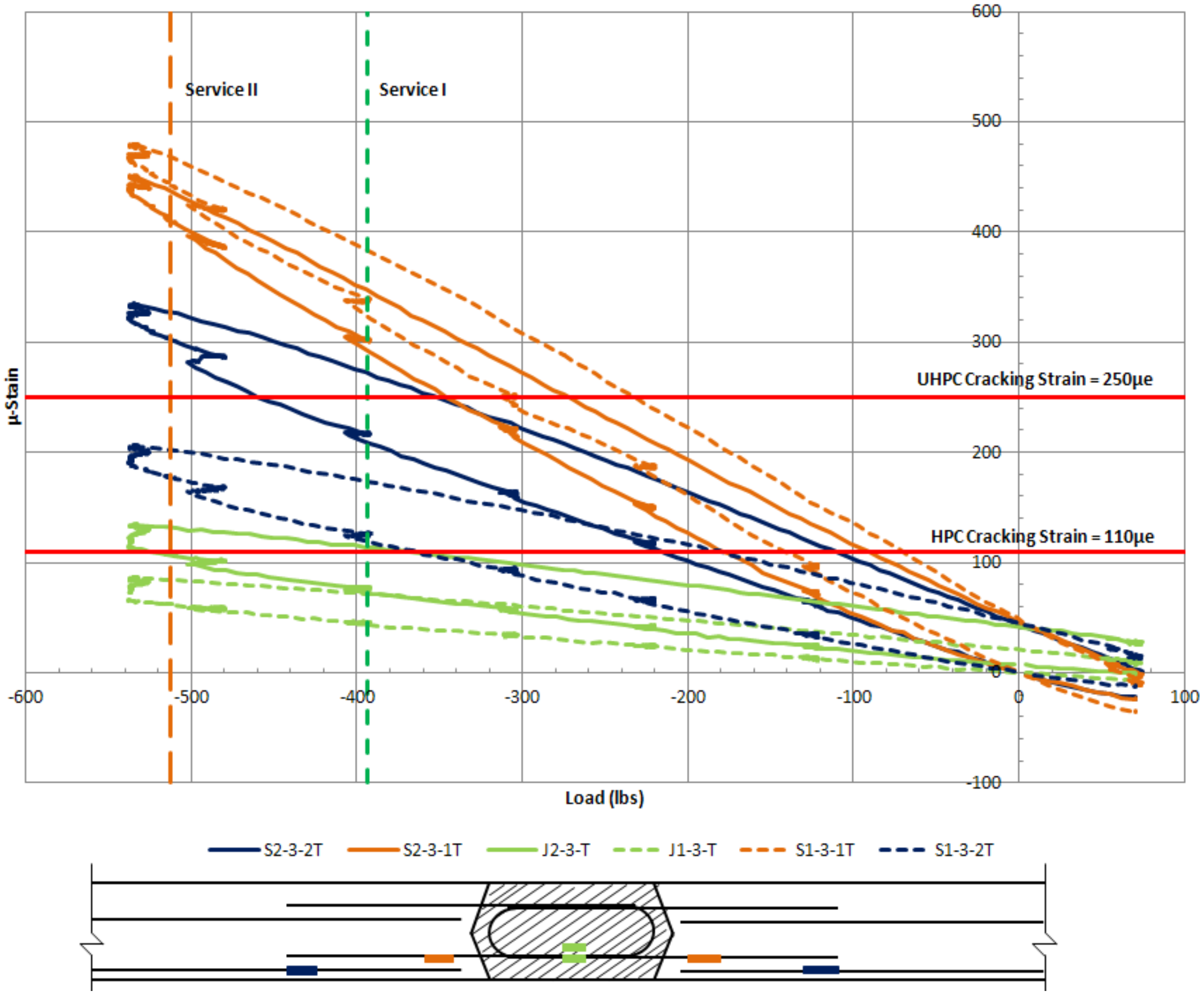

Figure 4.21. Row 3, Top-of-deck Embedded Strain Gages (Static)

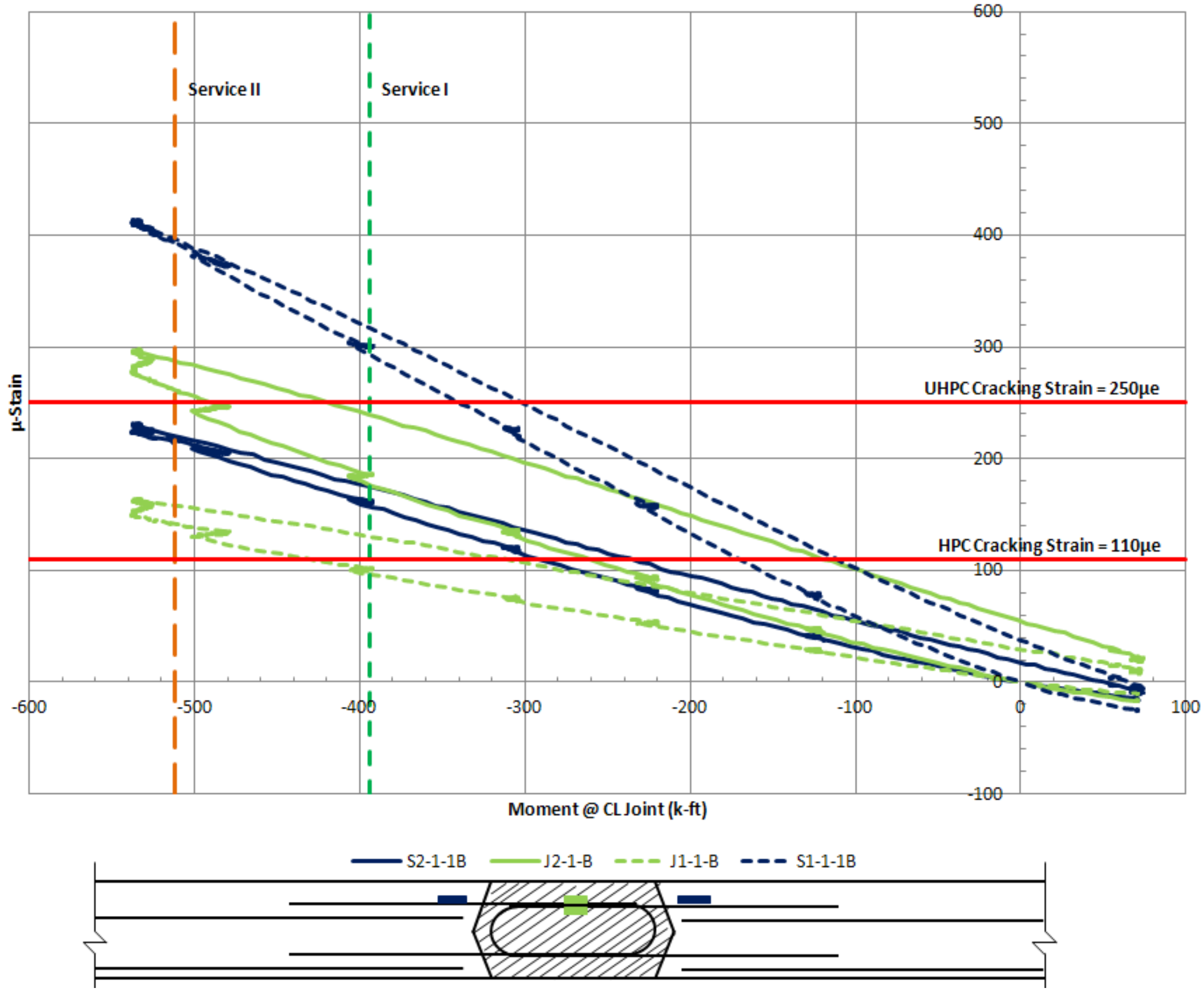

Figure 4.22. Row 1, Bottom-of-Deck Embedded Strain Gages (Static)

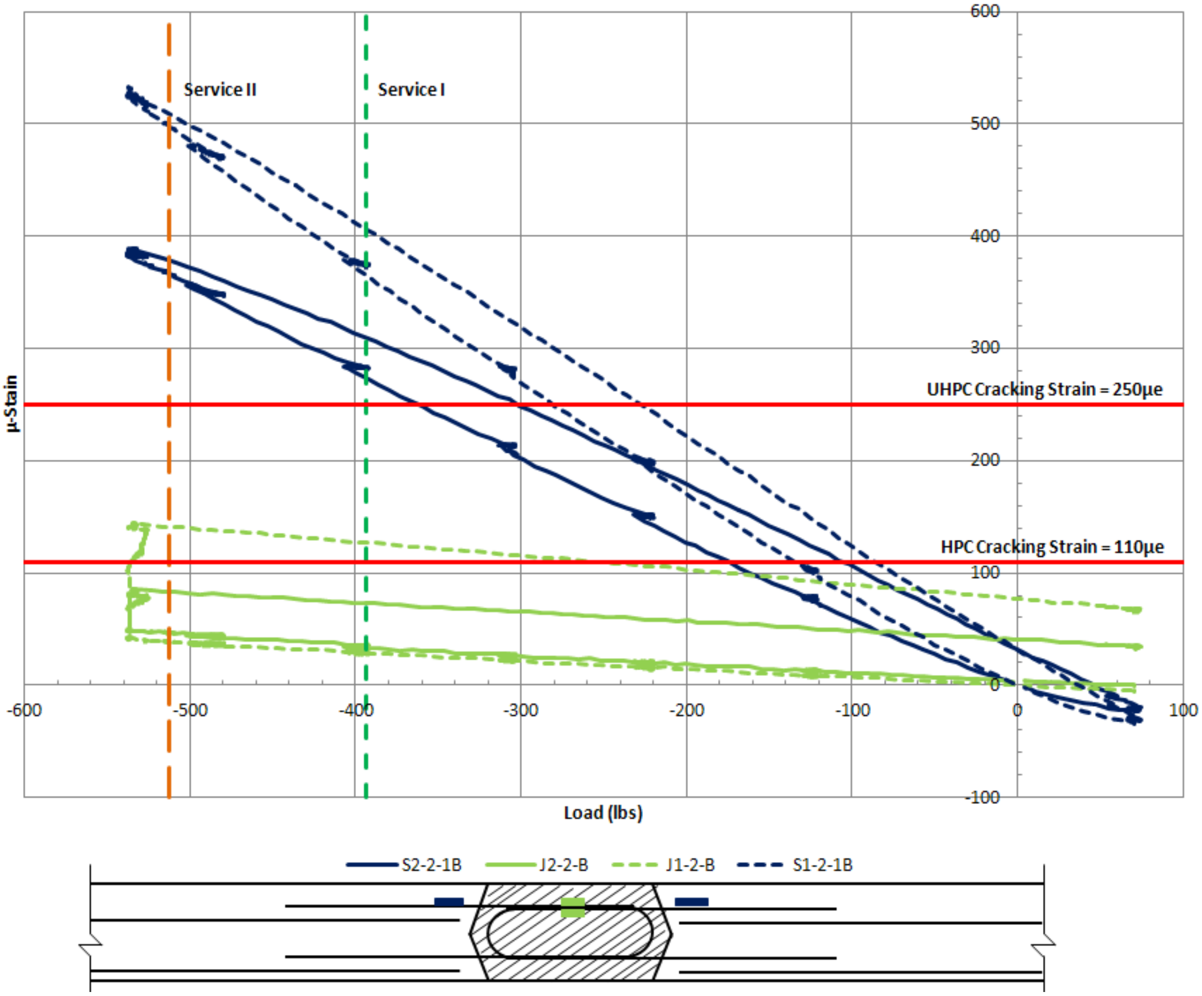

Figure 4.23. Row 2, Bottom-of-Deck Embedded Strain Gages (Static)

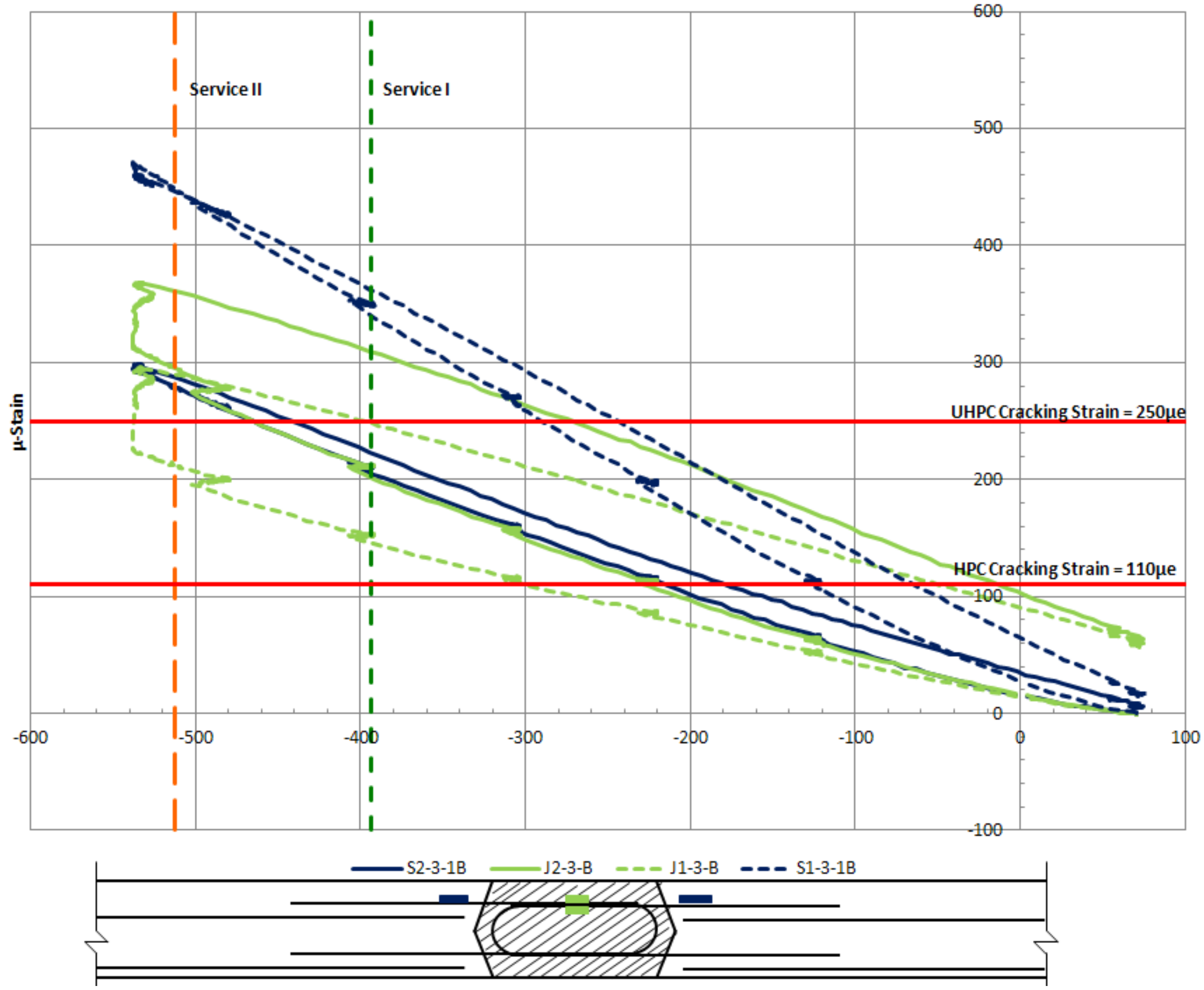

Figure 4.24. Row 3, Bottom-of-Deck Embedded Strain Gages (Static)

In general, embedded strains in the UHPC were lower than in the HPC precast deck at each instrumentation location. This can be attributed to the UHPC's material characteristics and higher modulus of elasticity. As previously discussed, the gages within two inches of the interface in the HPC deck registered the highest strains for all rows in both the bottom and top-of-deck. In addition, the prevalence of the high strains at the termination of the hairpin reinforcement in the top-of-deck means cracking of the HPC is expected. This data suggests that the transverse connection detail was not satisfying the original project aim to avoid cracking in the deck over the pier.

Service Level Fatigue Test Results

After static tests were completed, fatigue testing commenced. Fatigue tests consisted of loading the specimen through the full service level moment range for 1,000,000 cycles. The loading rate was one cycle per second, requiring approximately two weeks to complete. Strain data for embedded gages on the top-of-deck reinforcement after the completion of 1,000,000 cycles are presented for gages in rows 1, 2, and 3 (Figure 24 – Figure 26).

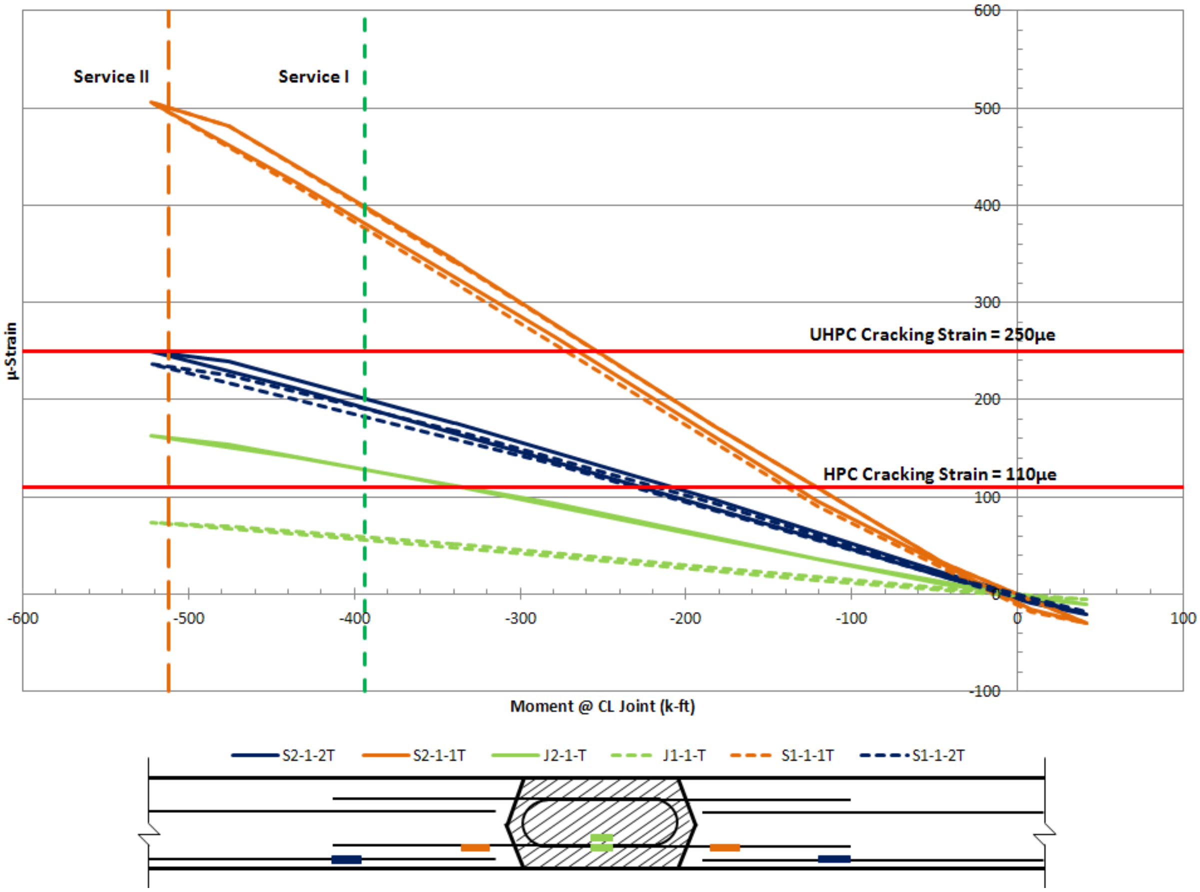

Figure 4.25. Row 1, Top-of-Deck Embedded Strain Gages (1,000,000 cycles)

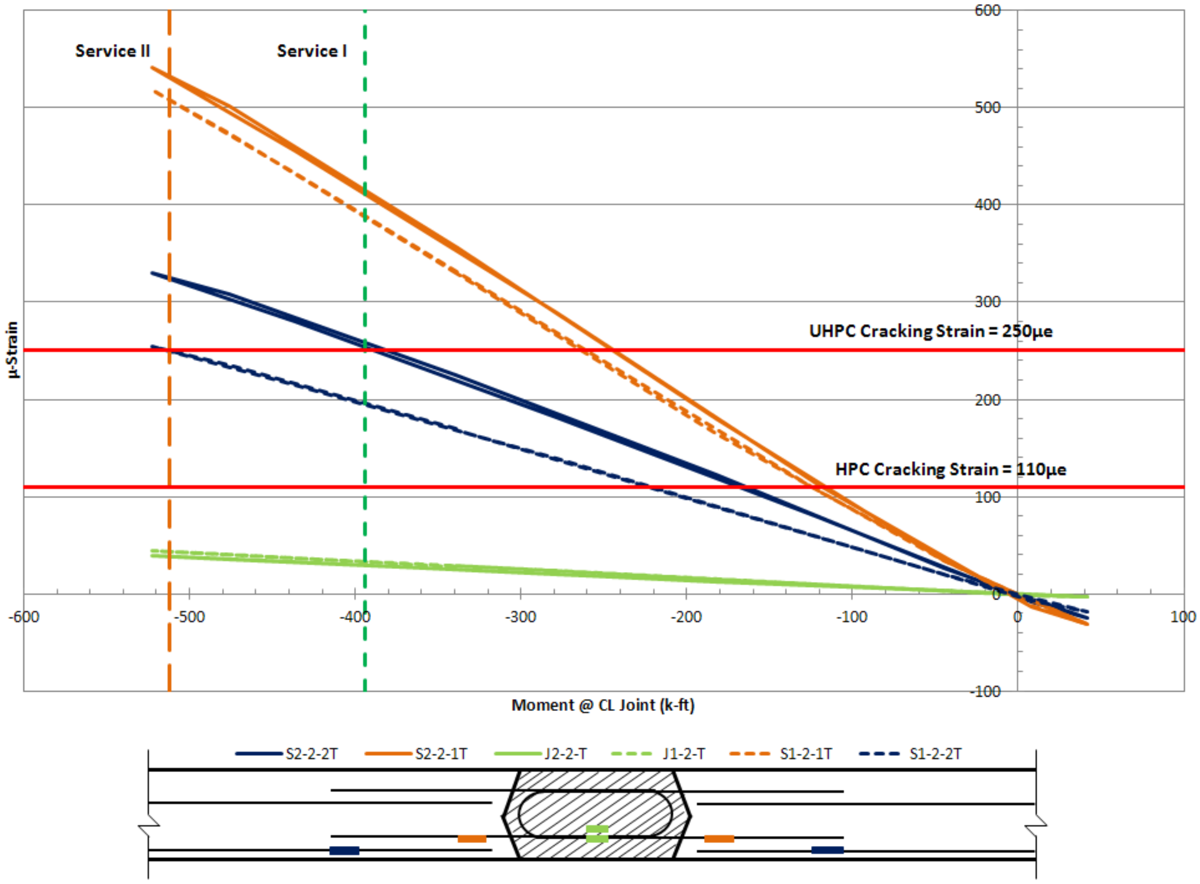

Figure 4.26. Row 2, Top-of-Deck Embedded Strain Gages (1,000,000 cycles)

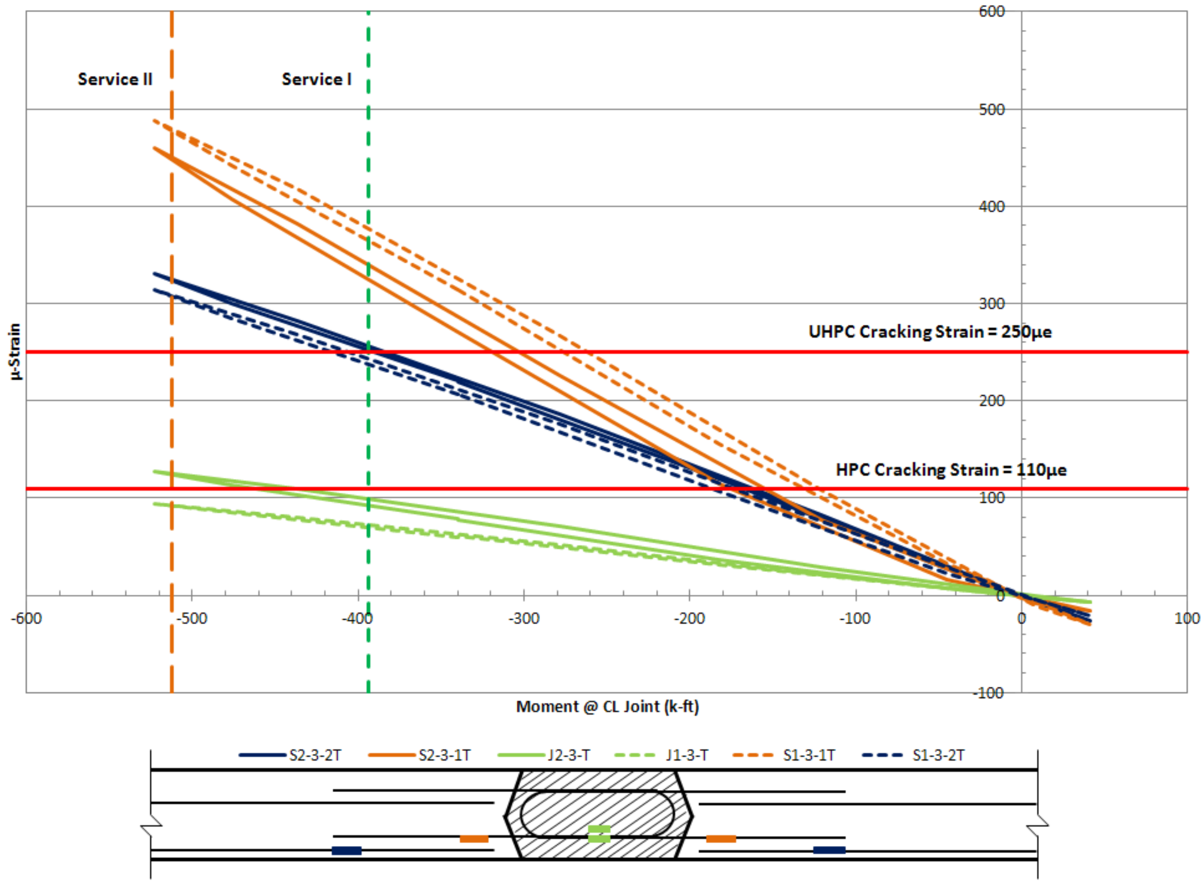

Figure 4.27. Row 3, Top-of-Deck Embedded Strain Gages (1,000,000 cycles)

The embedded strain results for the fatigue testing generally resembled those from the static testing. Similarly, the gages near the interface consistently exhibited the highest strains while the gages within the UHPC registered the lowest in each of the instrumentation rows. Some higher strain levels at 1,000,000 cycles when compared to the static testing results suggested propagation of cracking and damage accrual within the specimen.

Visual inspection at the onset of cyclic loading revealed cracking in the precast HPC deck around the joint at roughly half of Service Level I conditions (Figure 4.28). Upon inspection at 250,000 cycles, cracks were identified in the precast deck up to 10 ft away from the joint.

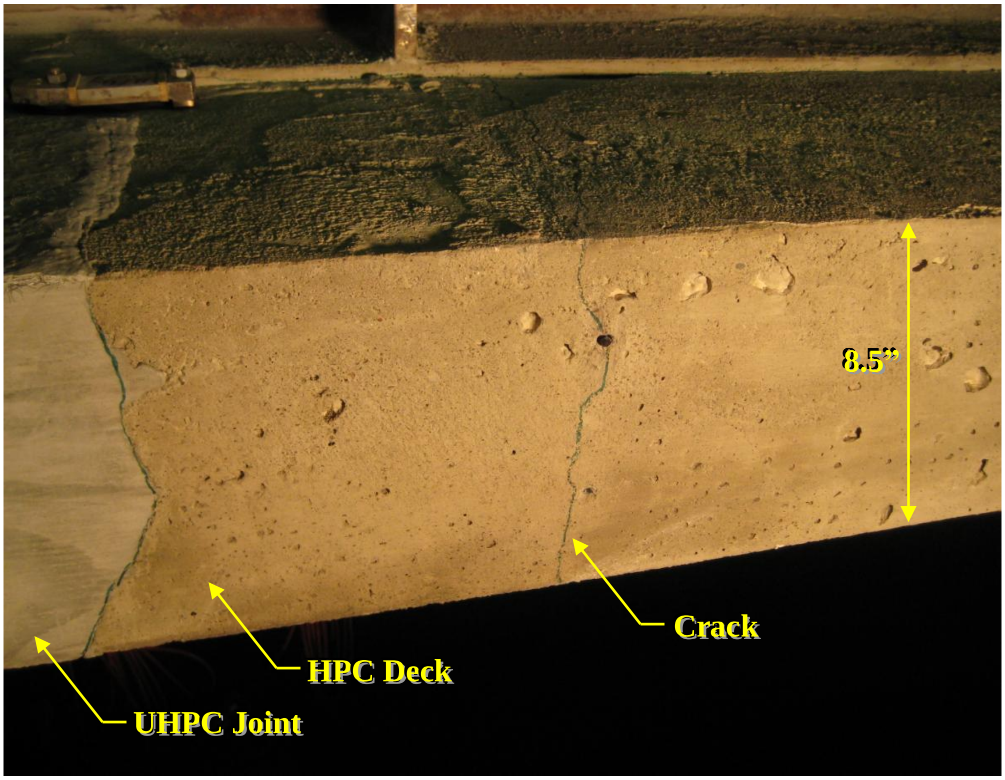

Figure 4.28. Full Depth Cracking in Precast Deck

Damage accrual to the specimen during the fatigue testing was analyzed by comparing strain values at various cycle counts. Strain accrual data is presented for gage groupings in rows 1, 2, and 3 (Figure 4.29 – Figure 4.34 & Table 4.9 – Table 4.14). Increase in strains at embedded gage locations throughout the specimen suggested propagation of the initial cracks from the incremental static service level load tests during the fatigue testing. In the bottom-of-deck data for rows 1 and 3 (Figure 4.30 & Figure 4.34), the high strain levels within the UHPC joint are likely due to the localized prying effects of the girders protruding into the joint on the bottom of the deck.

At 500,000, 750,000, and 1,000,000 cycles, further visual inspection confirmed propagation of the existing cracks and formation of new full-depth cracks in the precast deck panels within 10 ft of the joint.

Table 4.9. Row 1, Top-of-Deck Strain Accrual

Gage	S2-1-2T µe	S2-1-1T µe	J2-1-T µe	J1-1-T µe	S1-1-1T µe	S1-1-2T µe
3,000 Cycles	210	440	146	58	474	172
1,000,000 Cycles	250	506	163	73	505	236
Strain Increase	19.3%	15.0%	12.1%	27.4%	6.6%	37.6%

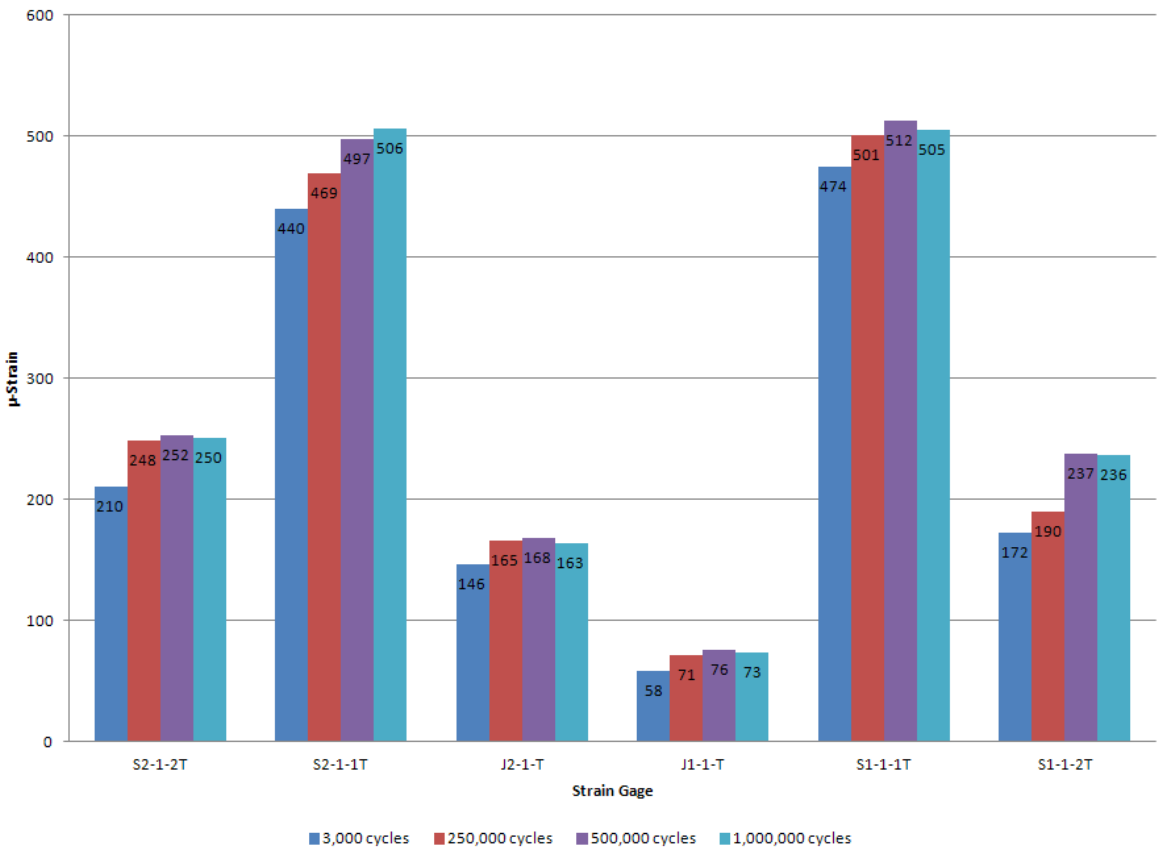

Figure 4.29. Row 1, Top-of-Deck Strain Accrual

Table 4.10. Row 1, Bottom-of-Deck Strain Accrual

Gage	S2-1-1B με	J2-1-B με	J1-1-B με	S1-1-1B με
3,000 Cycles	208	278	155	336
1,000,000 Cycles	259	329	200	336
Strain Increase	24.4%	18.4%	28.6%	0.0%

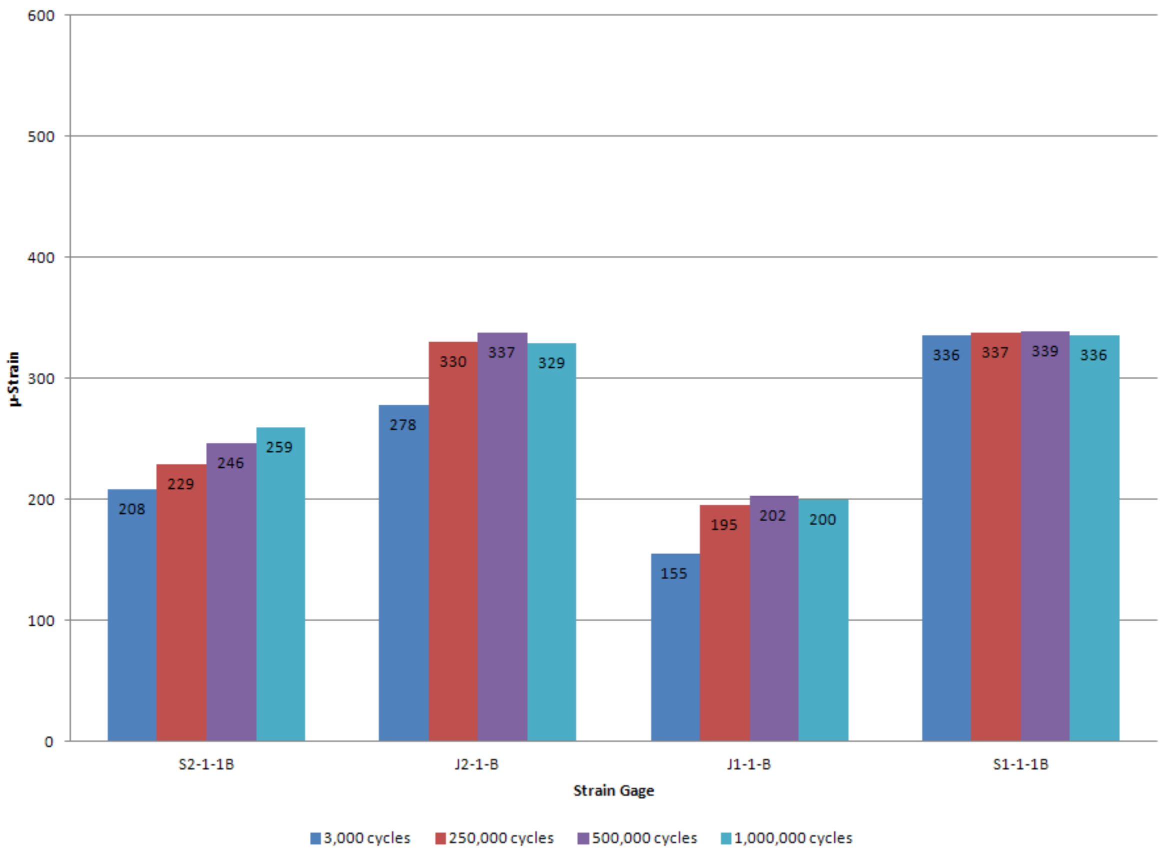

Figure 4.30. Row 1, Bottom-of-Deck Strain Accrual

Table 4.11. Row 2, Top-of-Deck Strain Accrual

Gage	S2-2-2T με	S2-2-1T με	J2-2-T με	J1-2-T με	S1-2-1T με	S1-2-2T με
3,000 Cycles	288	472	38	42	512	159
1,000,000 Cycles	330	542	39	44	518	255
Strain Increase	14.5%	14.7%	1.5%	3.6%	1.2%	60.2%

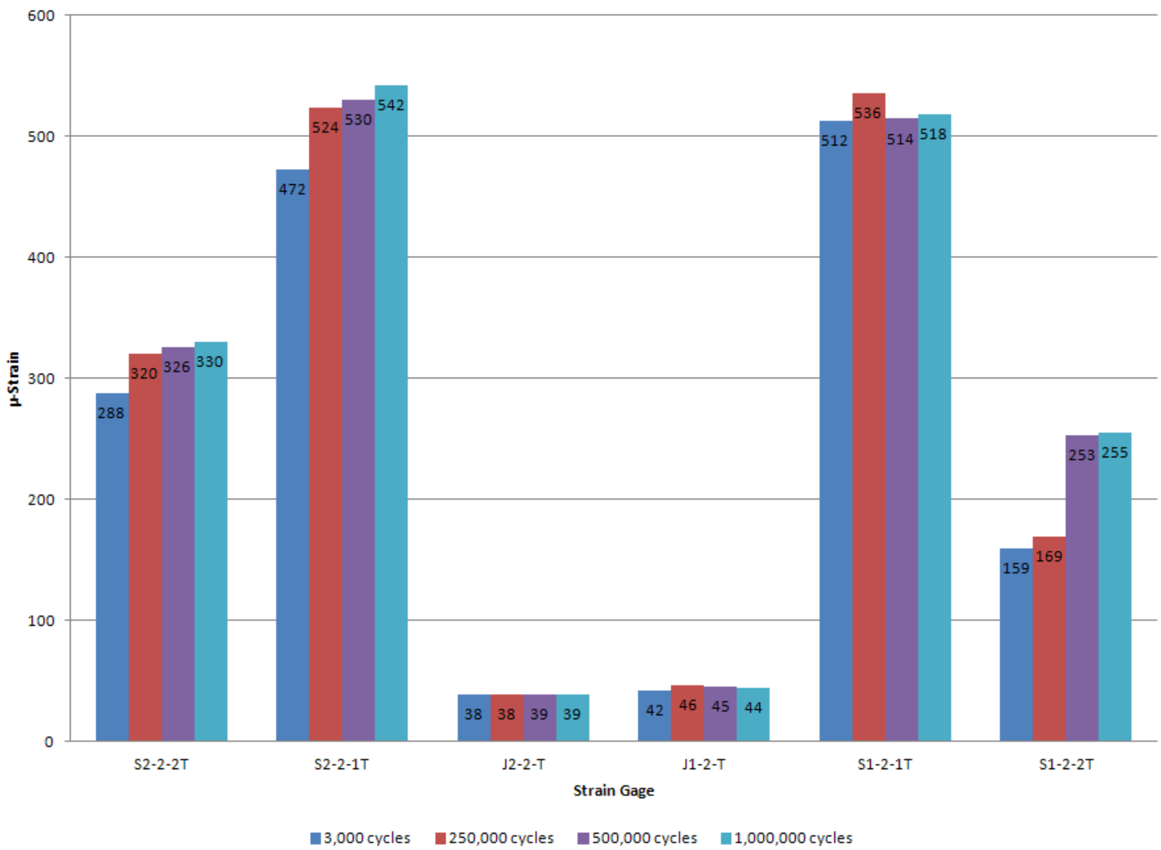

Figure 4.31. Row 2, Top-of-Deck Strain Accrual

Table 4.12. Row 2, Bottom-of-Deck Strain Accrual

Gage	S2-2-1B με	J2-2-B με	J1-2-B με	S1-2-1B με
3,000 Cycles	366	47	78	500
1,000,000 Cycles	411	52	88	492
Strain Increase	12.3%	10.8%	12.1%	-1.7%

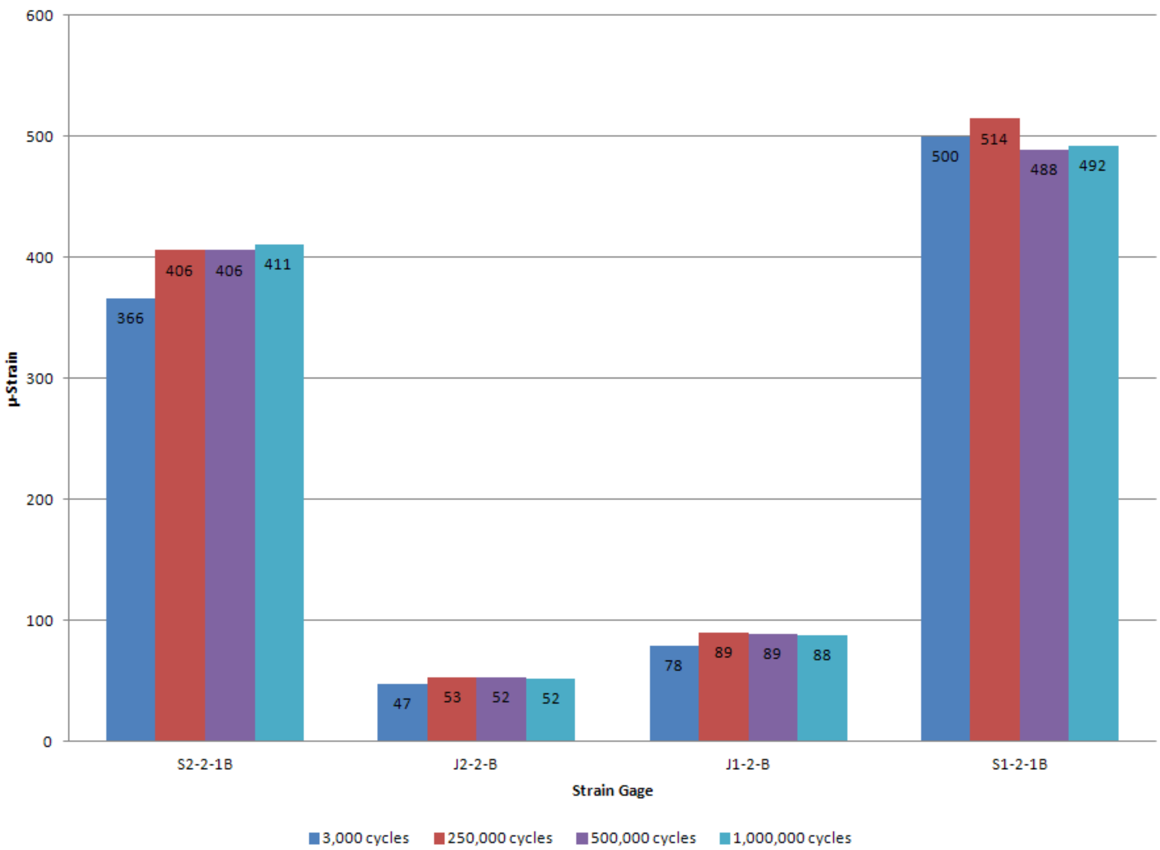

Figure 4.32. Row 2, Bottom-of-Deck Strain Accrual

Table 4.13. Row 3, Top-of-Deck Strain Accrual

Gage	S2-3-2T με	S2-3-1T με	J2-3-T με	J1-3-T με	S1-3-1T με	S1-3-2T με
3,000 Cycles	317	414	113	81	458	203
1,000,000 Cycles	331	460	127	95	488	314
Strain Increase	4.5%	11.1%	12.6%	16.4%	6.5%	54.2%

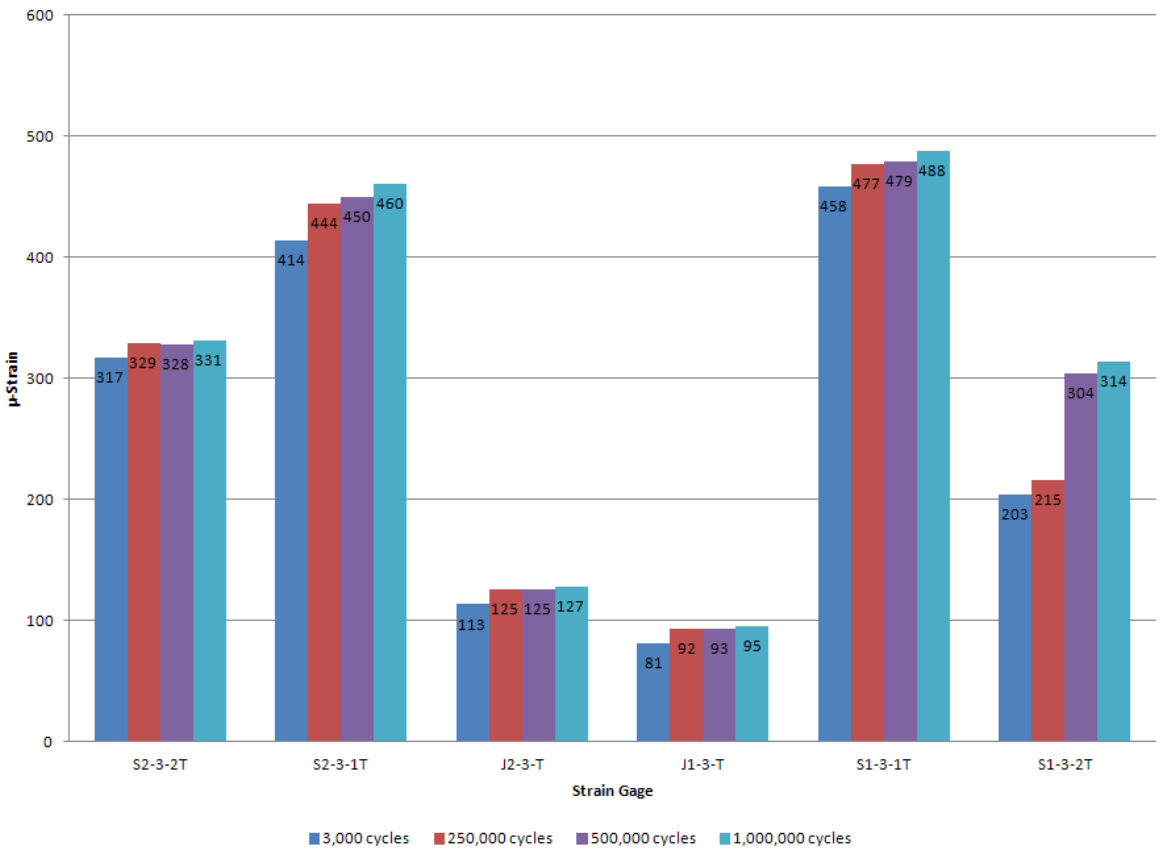

Figure 4.33. Row 3, Top-of-Deck Strain Accrual

Table 4.14. Row 3, Bottom-of-Deck Strain Accrual

Gage	S2-3-1B µe	J2-3-B µe	J1-3-B µe	S1-3-1B µe
3,000 Cycles	241	326	254	365
1,000,000 Cycles	288	377	297	349
Strain Increase	19.5%	15.9%	16.9%	-4.2%

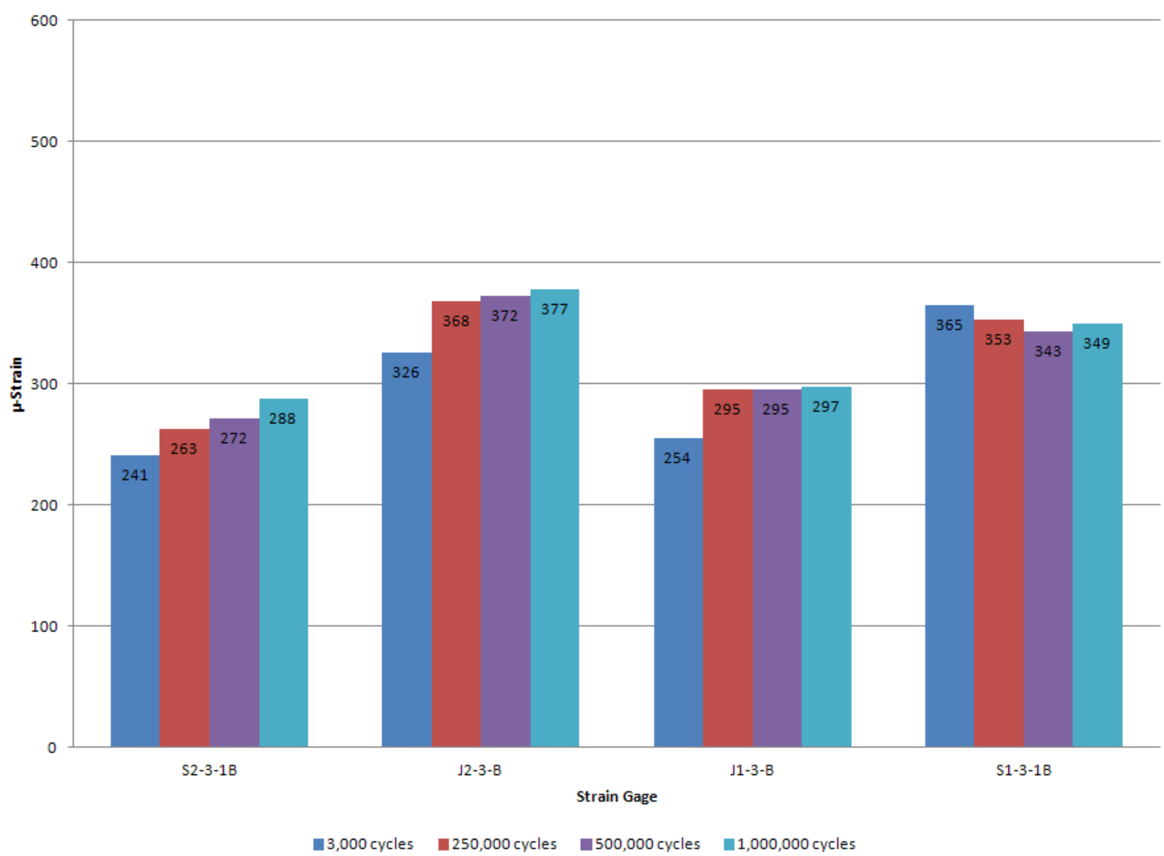

Figure 4.34. Row 3, Bottom-of-Deck Strain Accrual

The strain accrual throughout the fatigue testing varied at each location. Generally, the strain levels increased up to 28 percent. At one location, the strain level increased 60 percent while at others a small decrease was observed. These outliers are most likely due to the highly sensitive nature of embedded strain gages to localized cracking. In addition, because of the cyclic nature of loading and the frequency of data recordings, the peak strain readings for some gages could have been missed causing an apparent decrease in strain at certain locations.

As suggested by static testing results, visual inspection during fatigue testing confirmed early debonding and significant opening at the interface between the precast HPC deck panels and UHPC joint. In addition to debonding at the deck joint interface, cracking in the precast deck panels near the transverse joints was observed below Service Level I conditions. To mitigate these serious durability concerns, a modified detail was devised to post-tension the deck in this region and minimize tensile stresses in the concrete throughout Service Level II without compromising the accelerated construction aspect of the SHRP2 R04 project.

Connection Retrofit Test Results

Following fatigue testing, the specimen was modified to include high strength steel rods mounted just under the deck surface to post-tension the entire joint region and prevent any possible cracking of the deck or joint in this region (Figure 3.23). The retrofit detail was tested through the full range of service level moments with a 60 kip post-tensioning force per rod and again with a 70 kip post-tensioning force per rod. The static test results for surface mounted strain gages across the joint interface (Figure 4.35 & Figure 4.42) and embedded strain gages in rows 1, 2, and 3 (Figure 4.36 – Figure 4.39 & Figure 4.43 – Figure 4.46) are presented in this section.

The 60 kip post-tensioning force in each of the rods reduced tensile strain across the joint interface such that the HPC cracking strain was not reached until Service Level I conditions (Figure 4.35). A maximum tensile strain of 200με was recorded across the joint interface at Service Level I moment. Embedded strain gages never exceeded 110με prior to Service Level I conditions. However, strains did exceed the HPC cracking strain in the top-of-deck embedded gages before reaching Service Level II (Figure 4.36 – Figure 4.39).

By contrast, applying 70 kips post-tensioning force in each of the rods minimized or negated the tensile strain across the interface entirely when loaded to Service Level I. All surface mounted strain gages spanning the interface registered below the HPC cracking strain until after the Service Level I conditions were applied (Figure 4.42). Tensile strain data across the interface revealed a maximum 29με at Service Level I moment. All embedded strain gages, top and bottom-of-deck, did not exceed 110με until Service Level II conditions were applied (Figure 4.43 – Figure 4.46). The 70 kip per rod post-tensioning force was recommended for application in the SHRP2 Project R04 demonstration bridge.

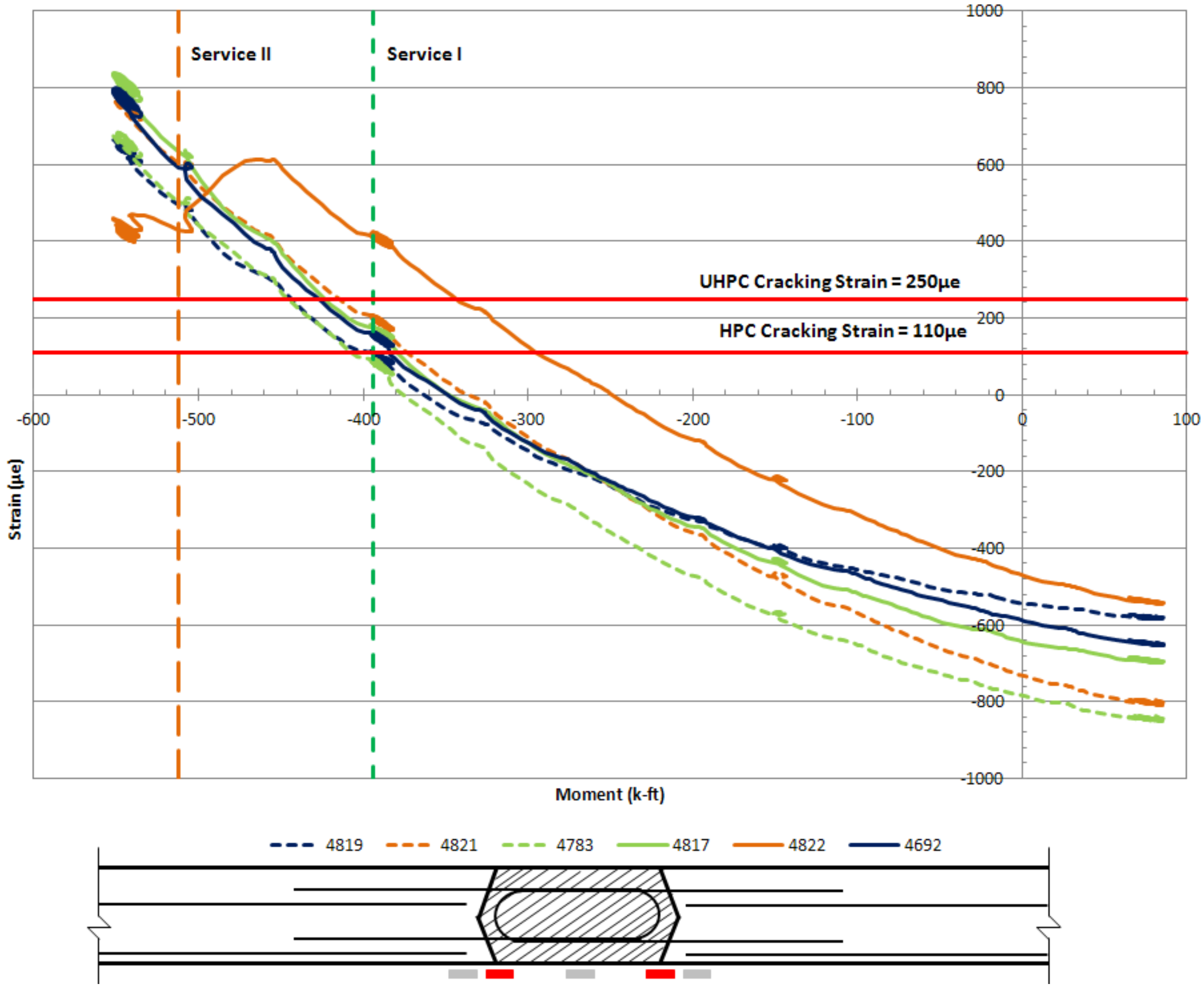

Figure 4.35. Top-of-Deck Surface Mounted Strain Gages over Interface (60k Retrofit)

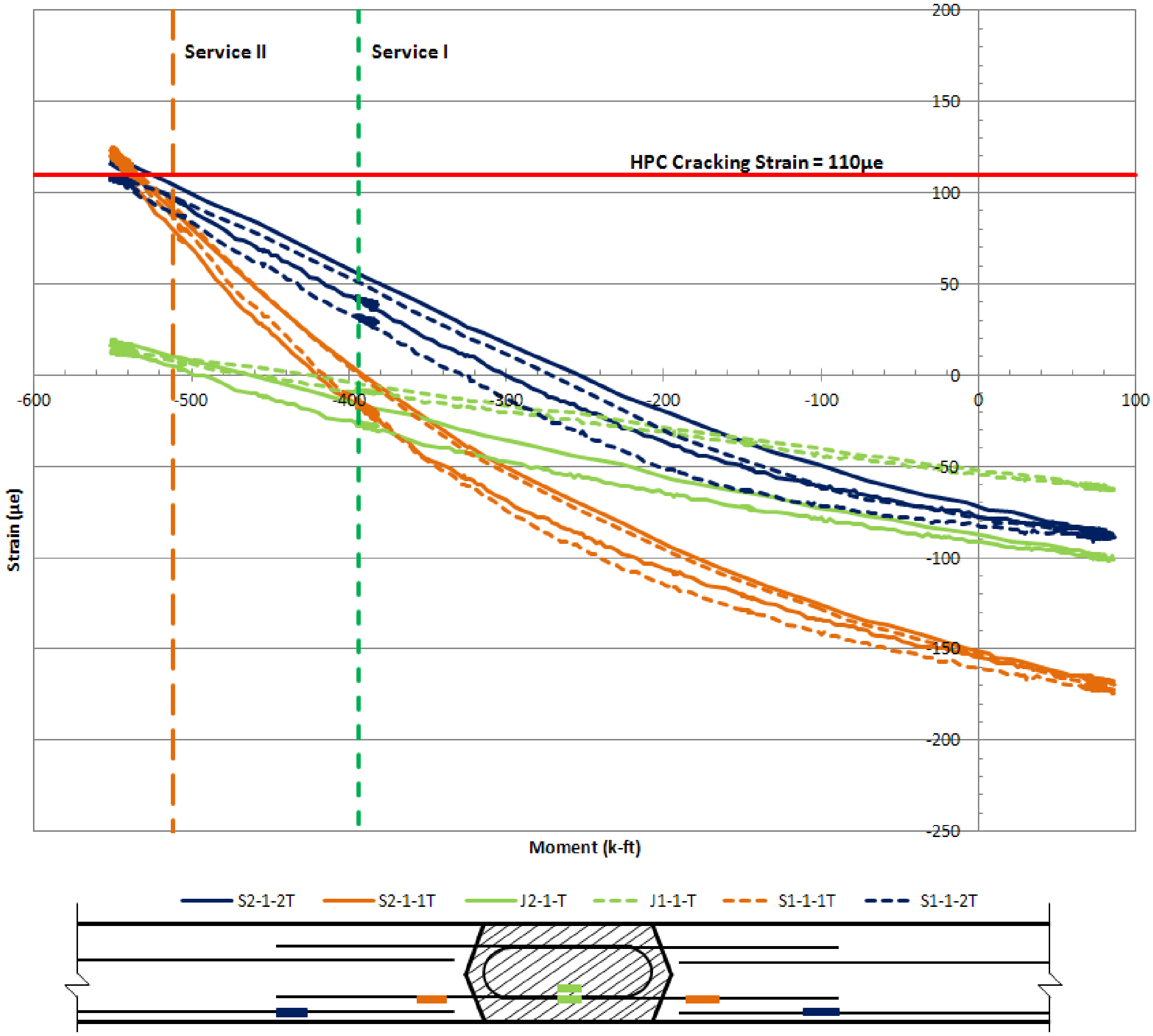

Figure 4.36. Row 1, Top-of-Deck Embedded Strain Gages (60k Retrofit)

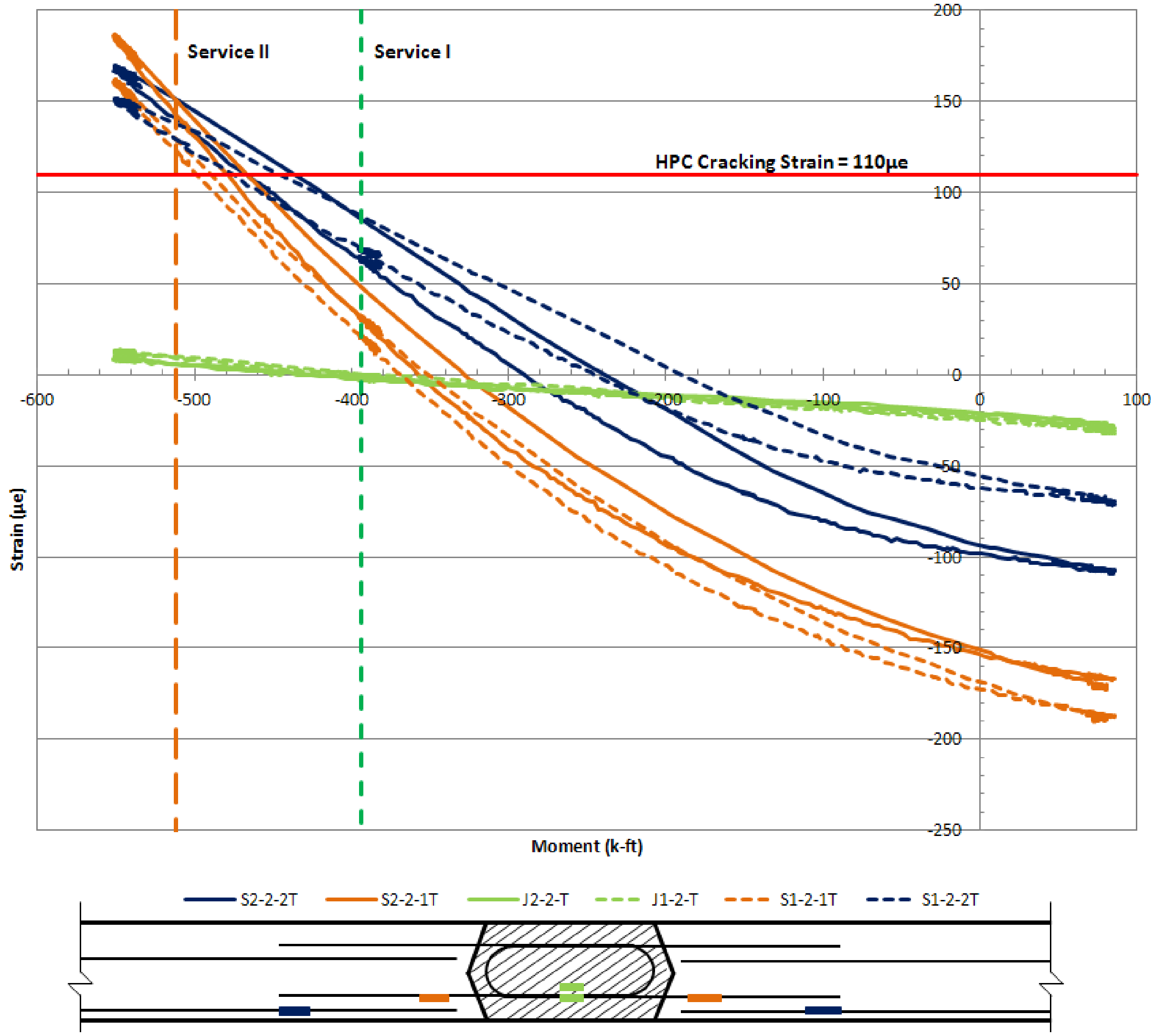

Figure 4.37. Row 2, Top-of-Deck Embedded Strain Gages (60k Retrofit)

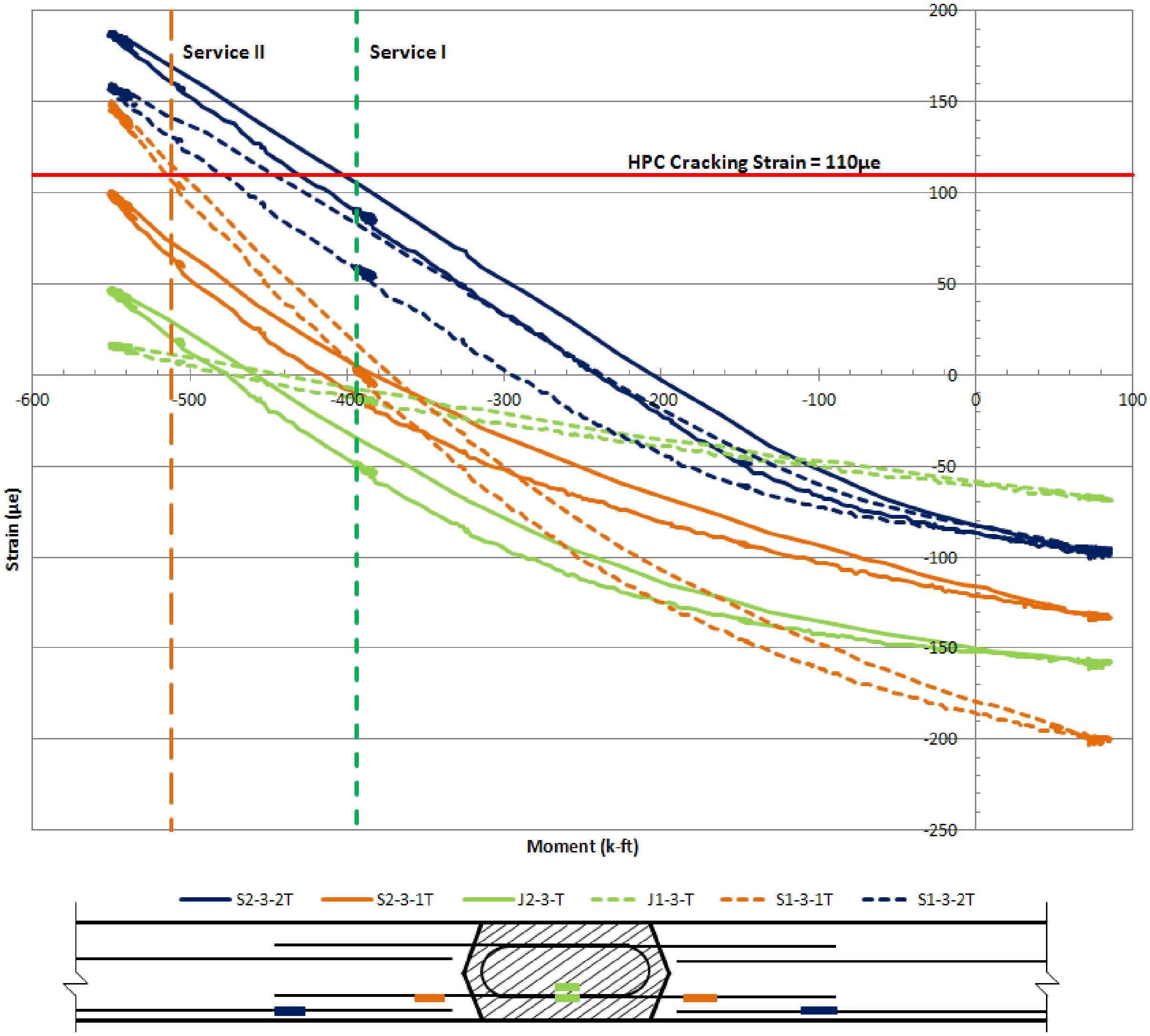

Figure 4.38. Row 3, Top-of-Deck Embedded Strain Gages (60k Retrofit)

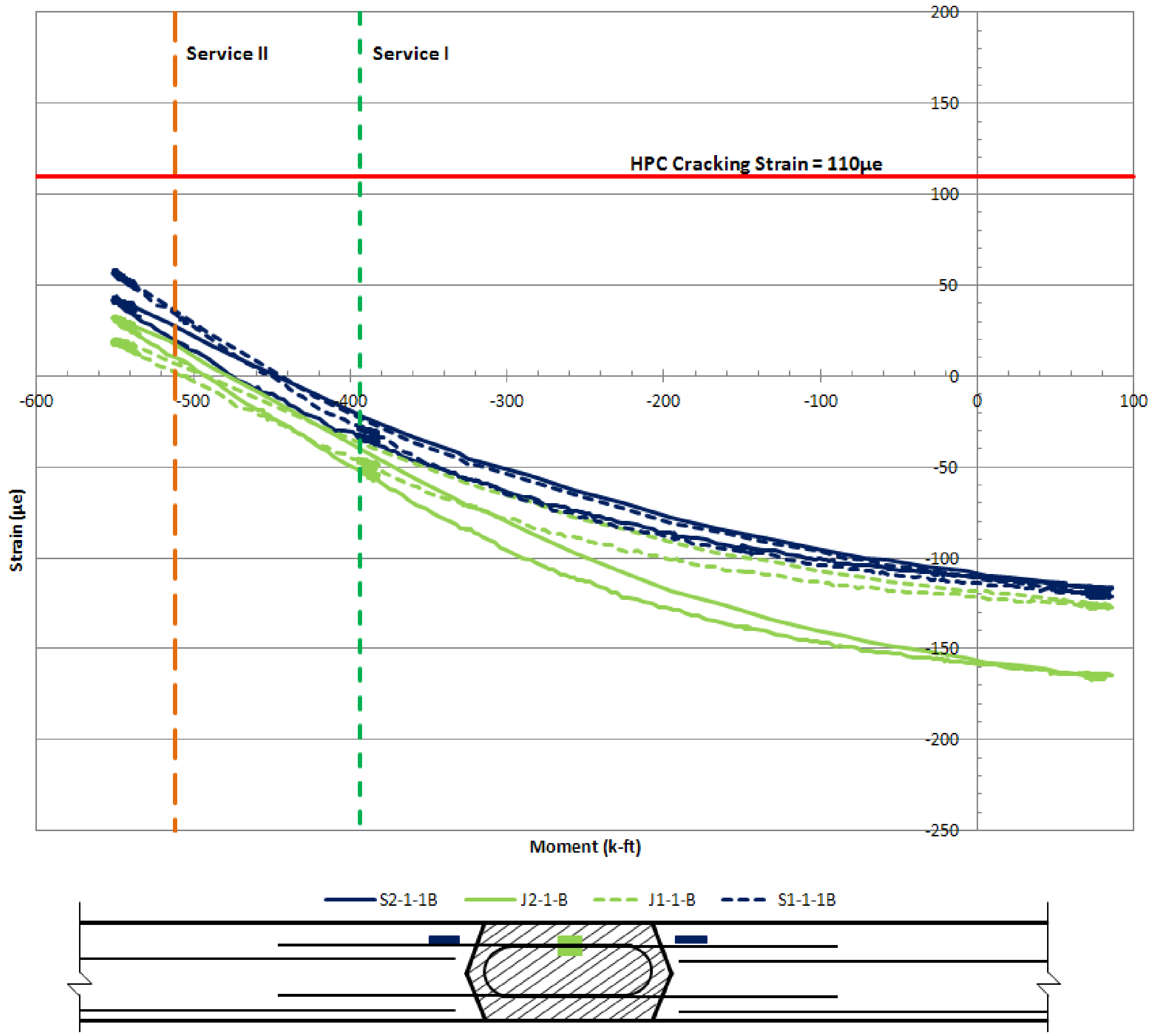

Figure 4.39. Row 1, Bottom-of-Deck Embedded Strain Gages (60k Retrofit)

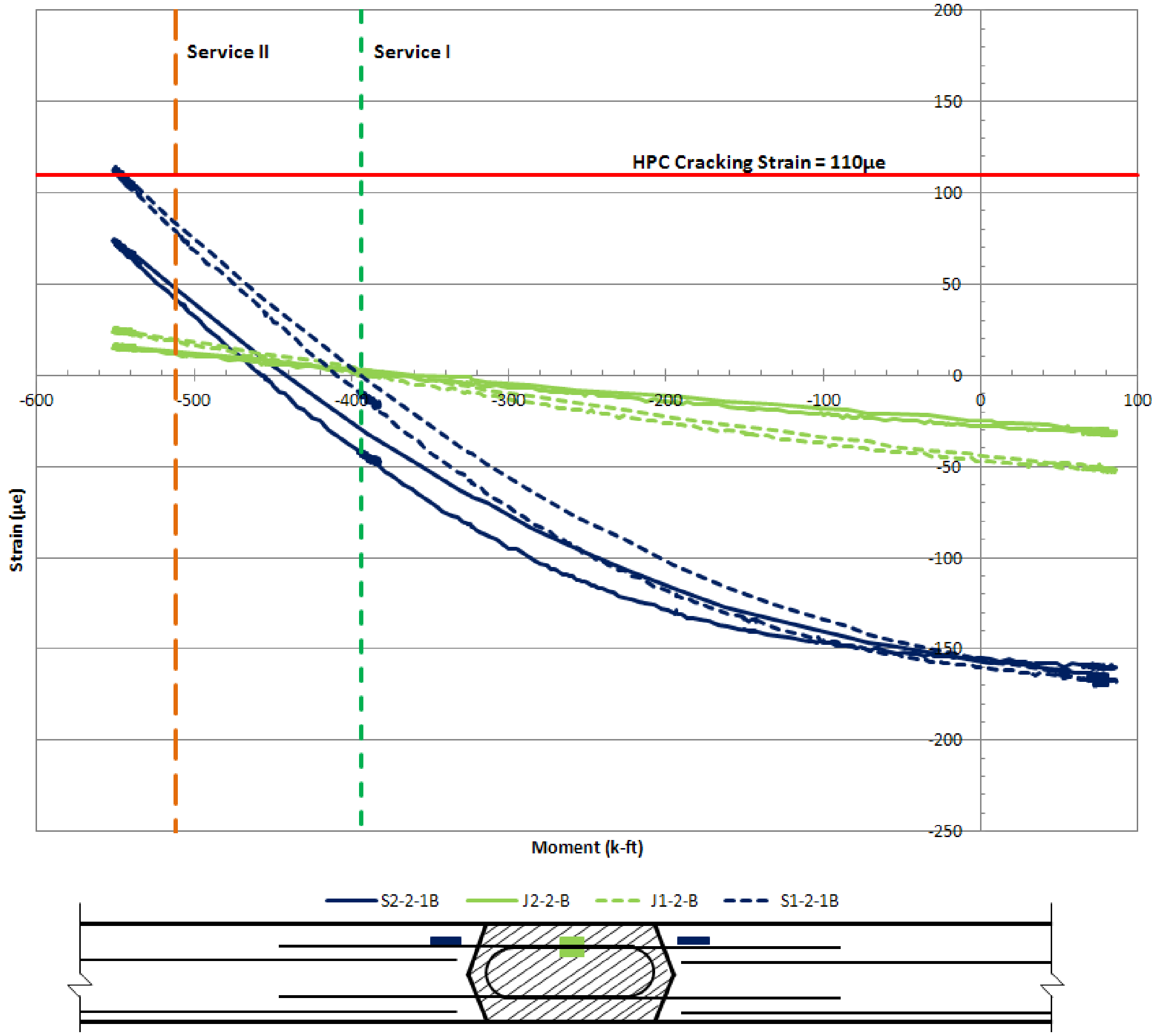

Figure 4.40. Row 2, Bottom-of-Deck Embedded Strain Gages (60k Retrofit)

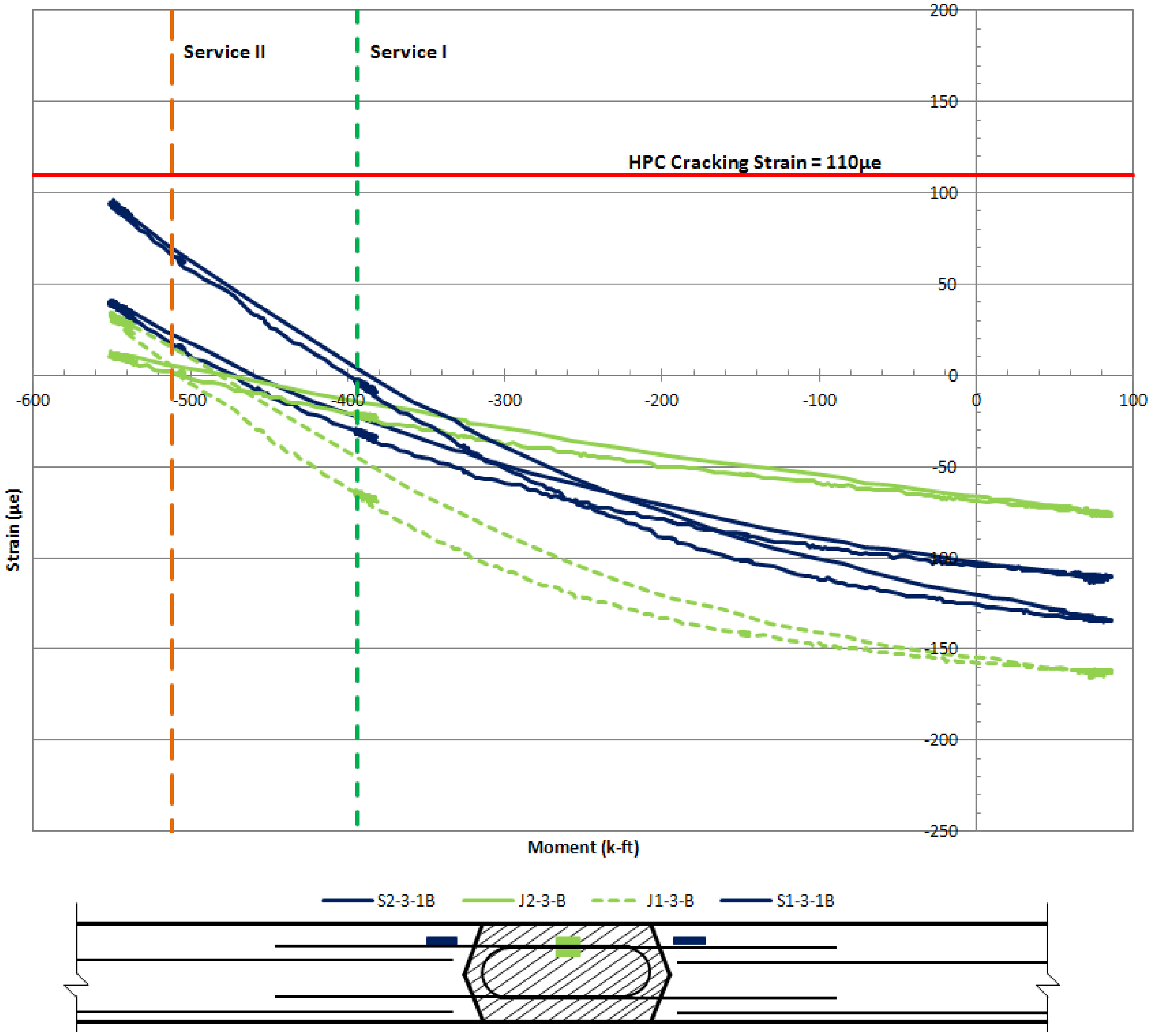

Figure 4.41. Row 3, Bottom-of-Deck Embedded Strain Gages (60k Retrofit)

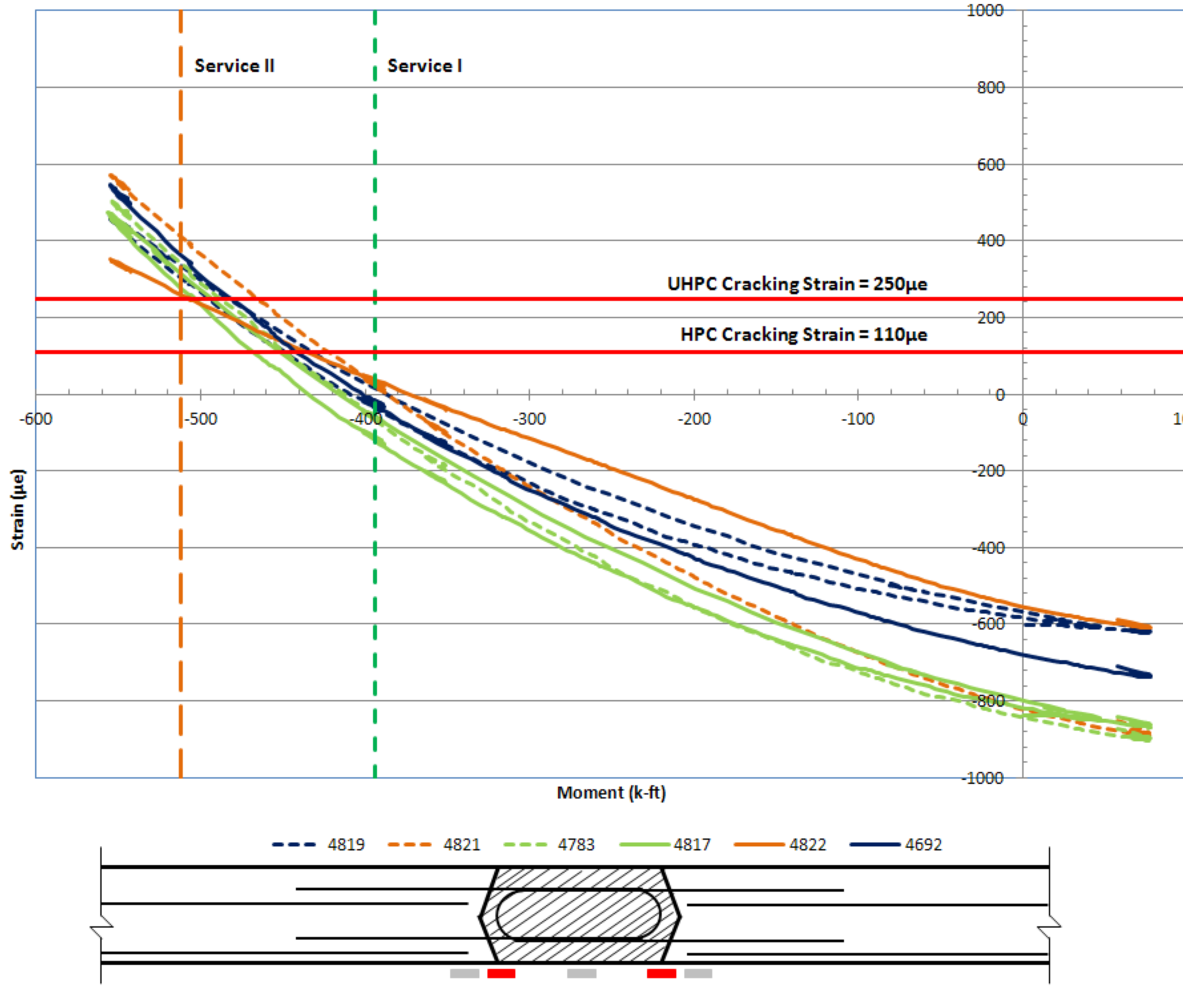

Figure 4.42. Top-of-Deck Surface Mounted Strain Gages over Interface (70k Retrofit)

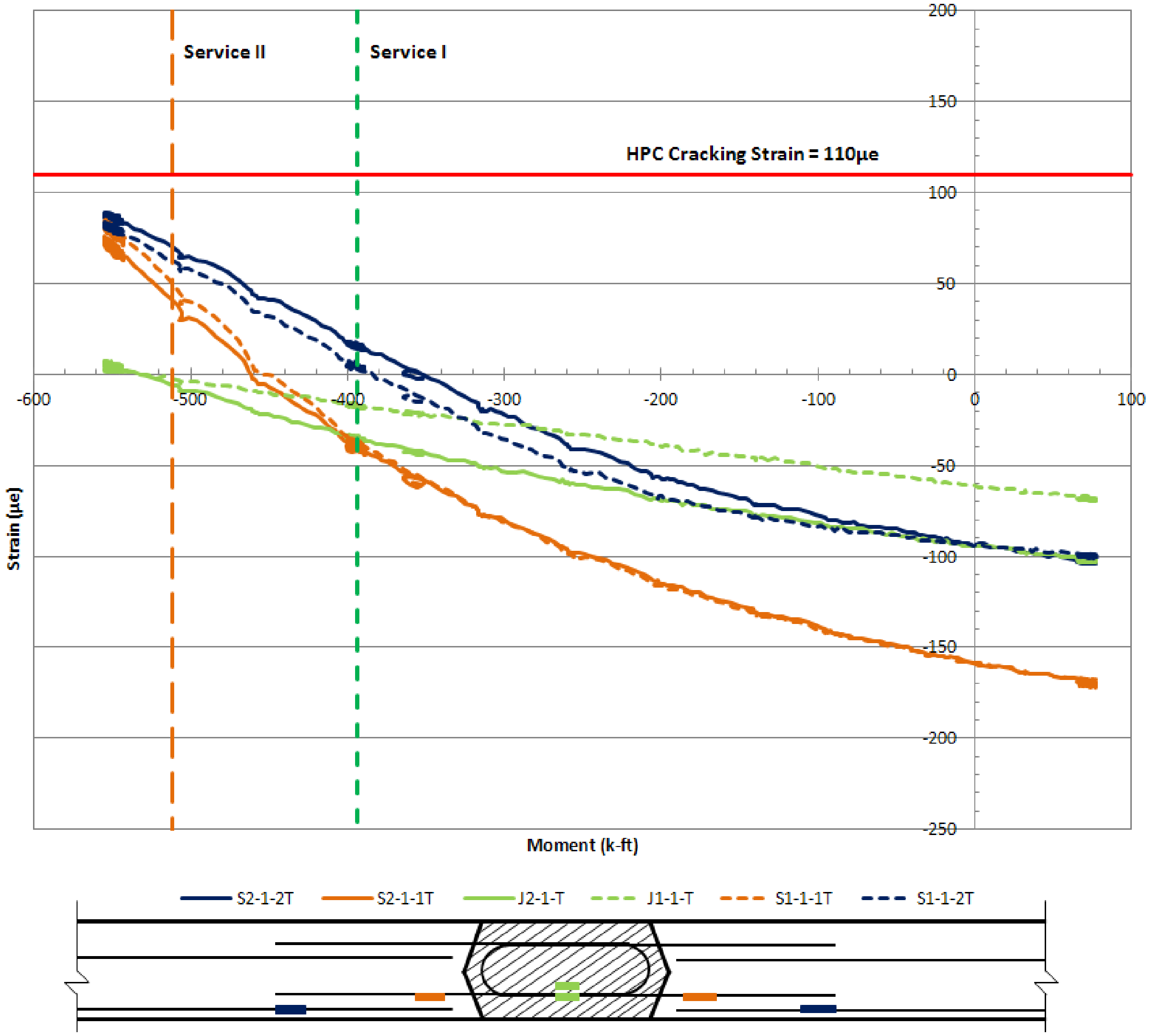

Figure 4.43. Row 1, Top-of-Deck Embedded Strain Gages (70k Retrofit)

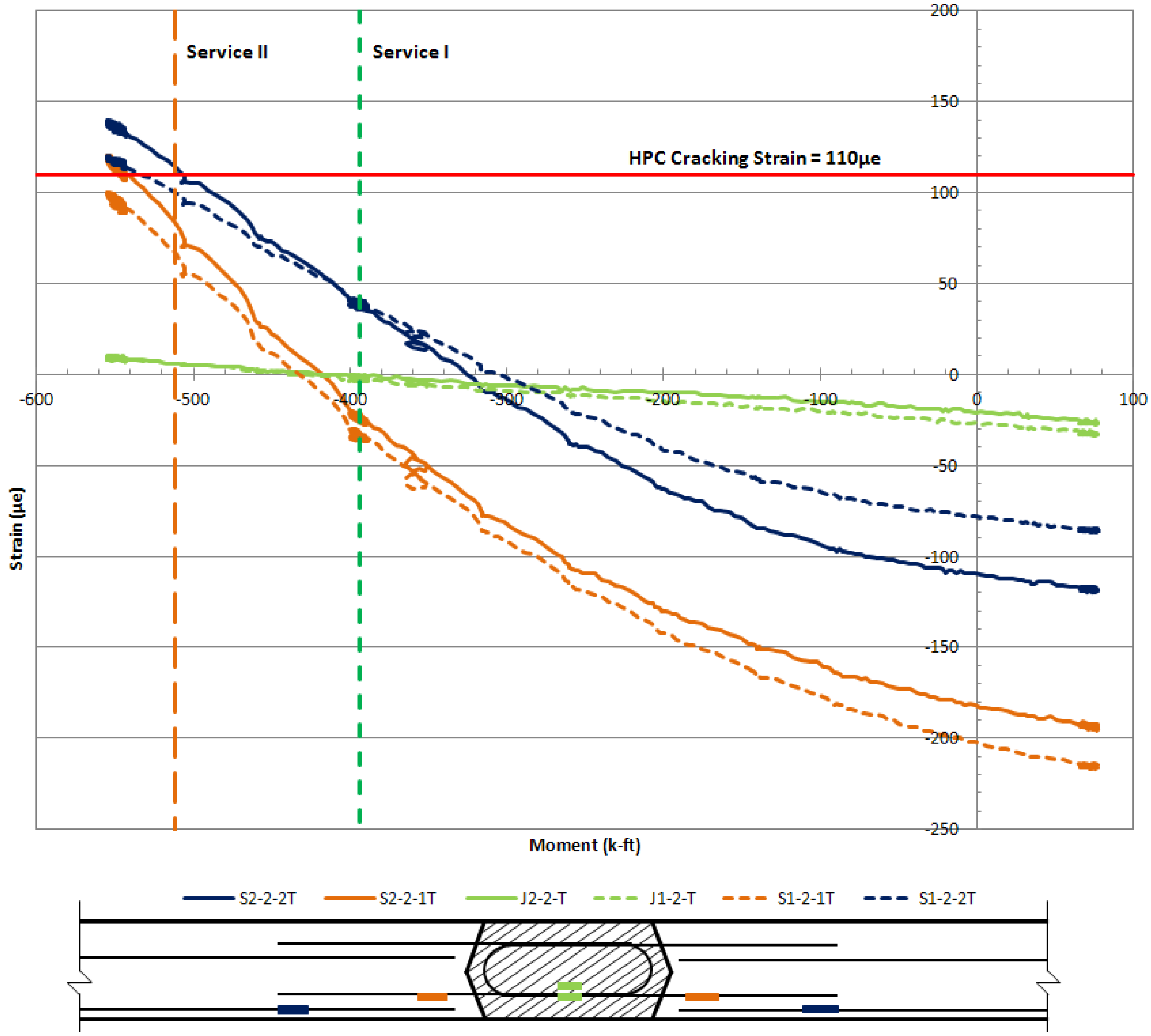

Figure 4.44. Row 2, Top-of-Deck Embedded Strain Gages (70k Retrofit)

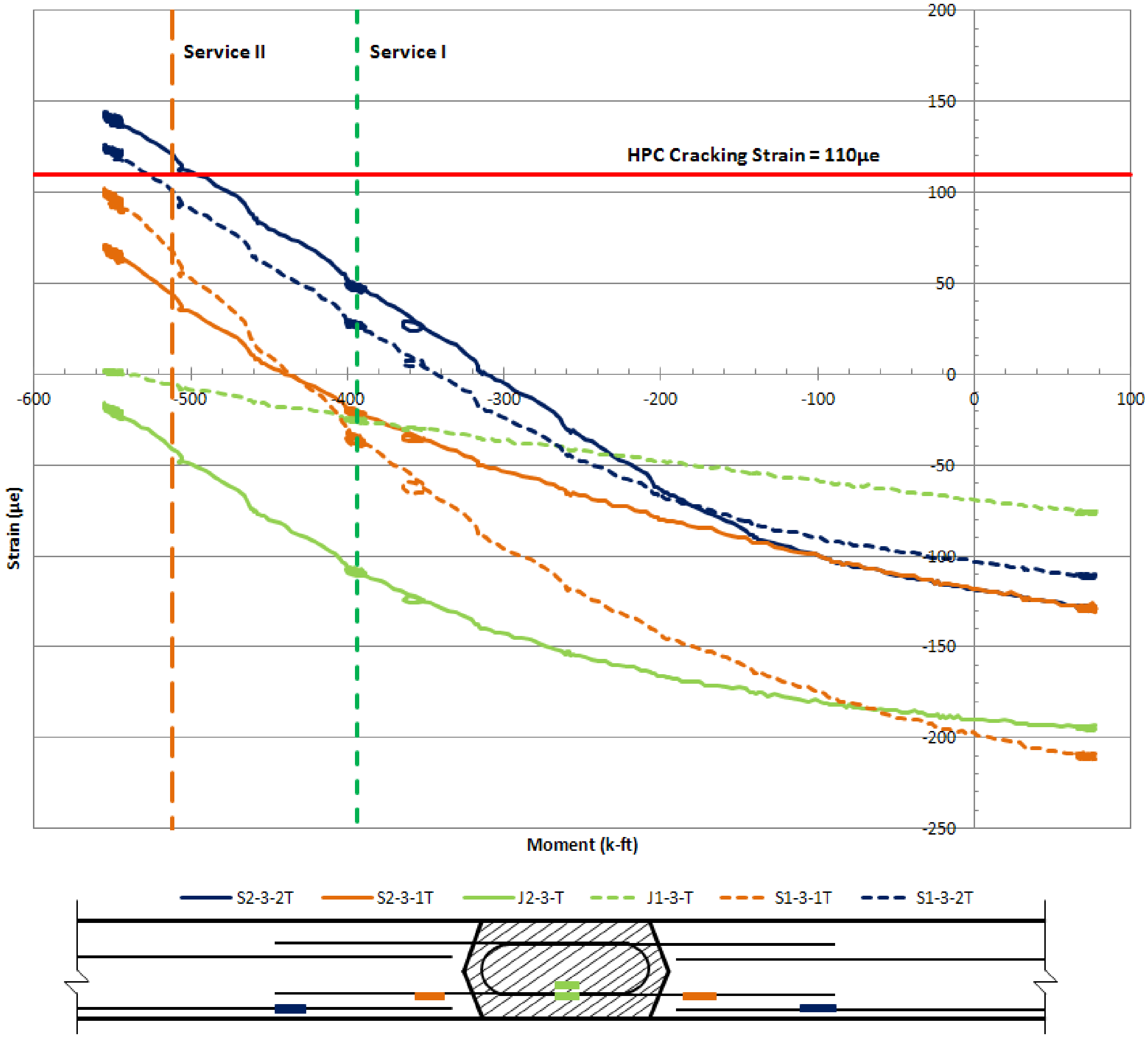

Figure 4.45. Row 3, Top-of-Deck Embedded Strain Gages (70k Retrofit)

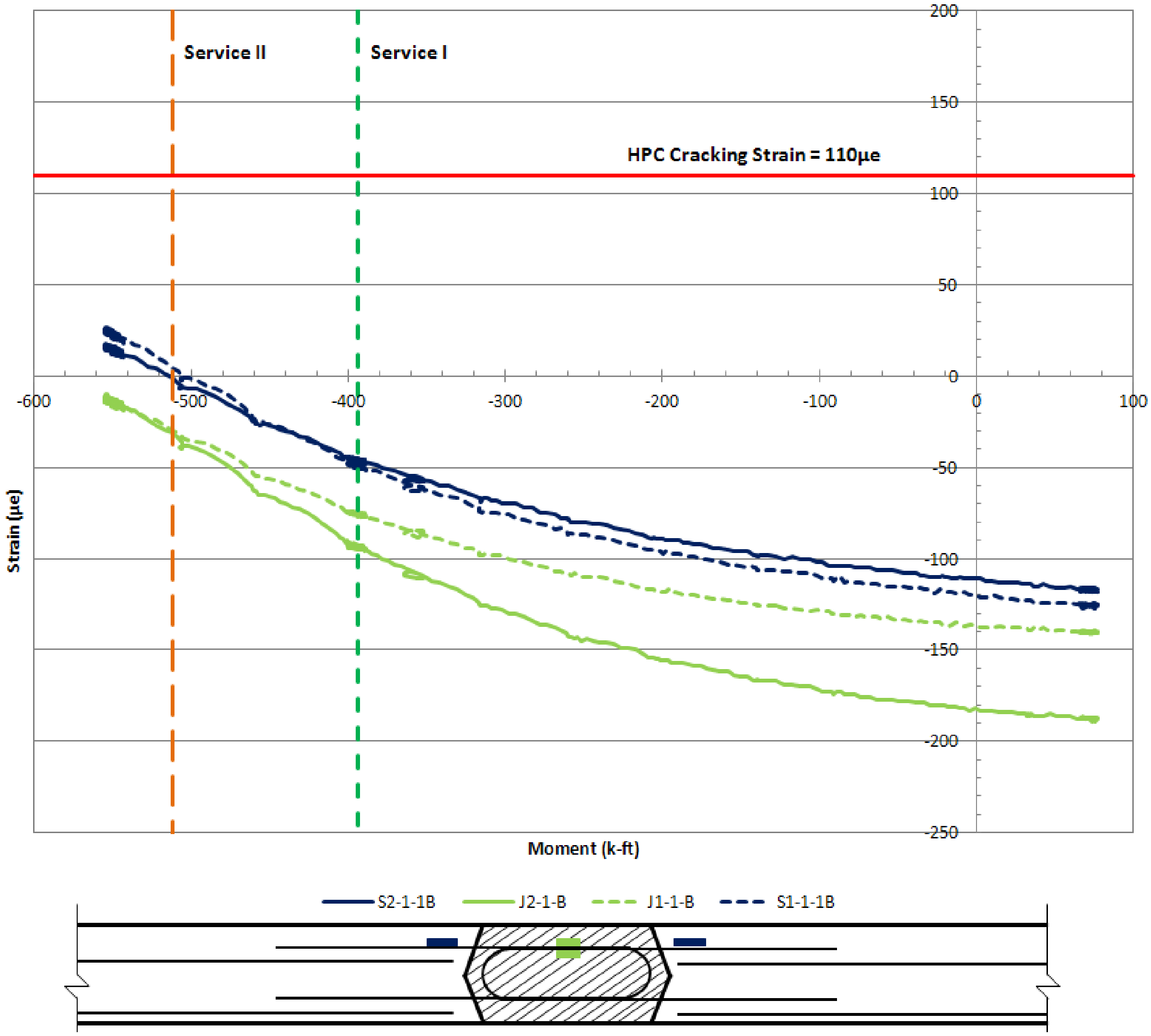

Figure 4.46. Row 1, Bottom-of-Deck Embedded Strain Gages (70k Retrofit)

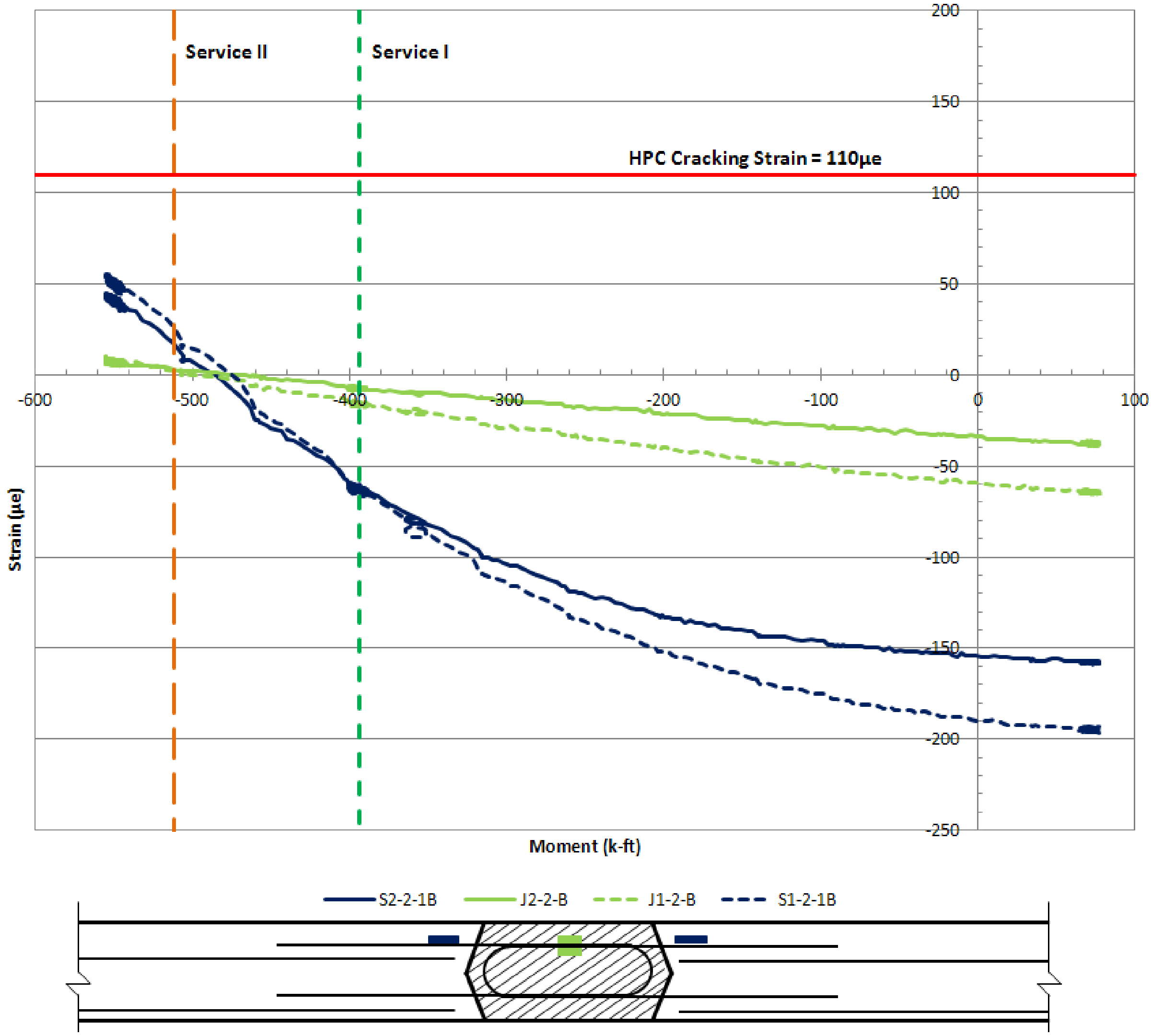

Figure 4.47. Row 2, Bottom-of-Deck Embedded Strain Gages (70k Retrofit)

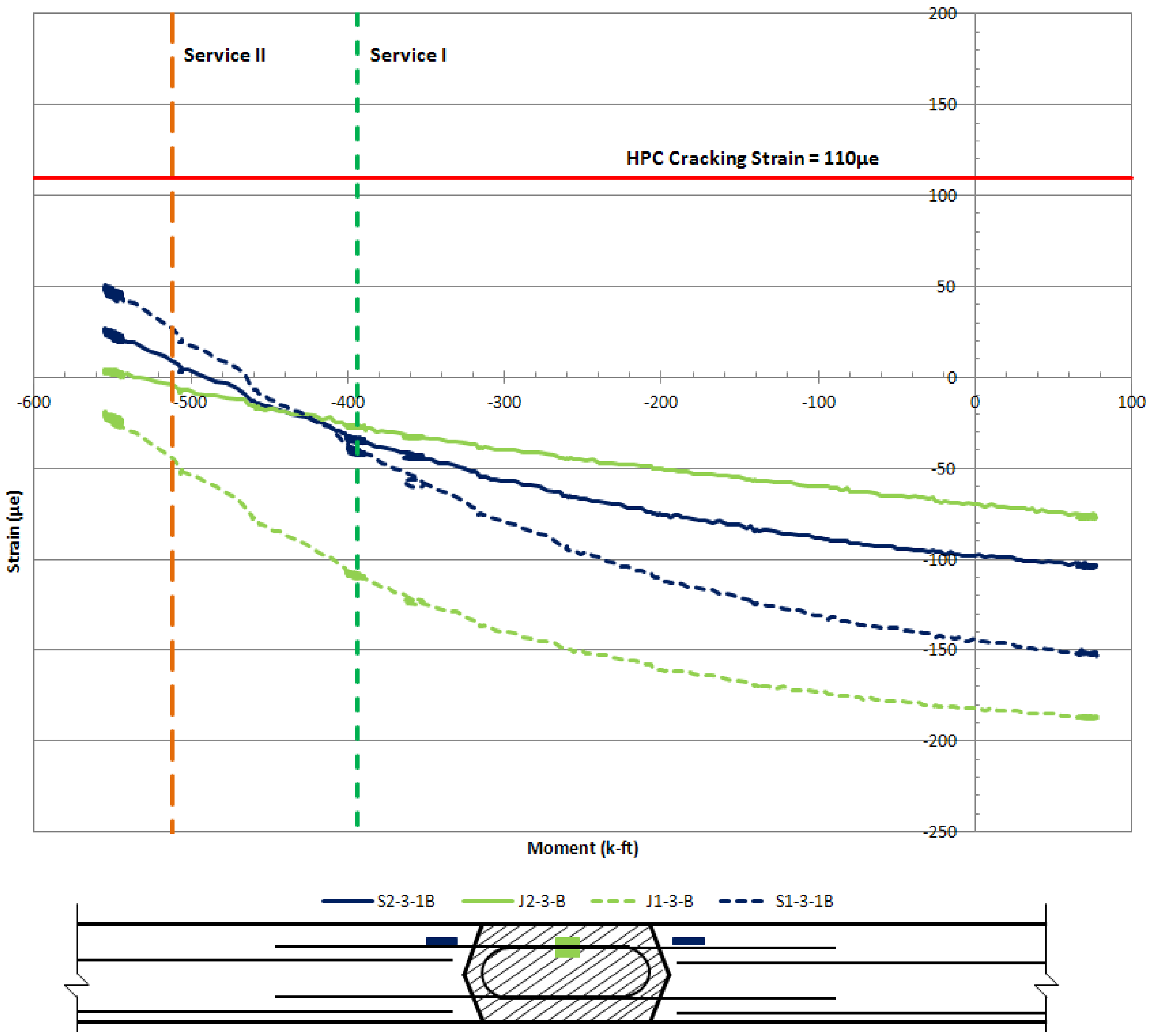

Figure 4.48. Row 3, Bottom-of-Deck Embedded Strain Gages (70k Retrofit)

Once again, embedded strain gages within the UHPC joint consistently register strains below those in the HPC deck for each of the instrumentation rows in both the top and bottom-of-deck reinforcement. The 60 kips of post-tensioning force per rod reduced strain levels from the previous incremental static testing, but not completely below the HPC cracking strain prior to Service Level I moment. The 70 kips of post-tensioning force per rod, however, did lower strains below the HPC cracking strain until Service Level II at each instrumentation row.

Ultimate Capacity Test Results

Upon completion of static testing for the modified detail, the post-tensioning rods were removed and the transverse module-to-module connection detail was tested to ultimate moment capacity. Figure 4.49 shows the moment-displacement curve for the specimen during testing. Strain data for the embedded gages along reinforcement rows 1, 2, and 3 (Figure 4.50 – Figure 4.55) were analyzed in combination with qualitative observations to determine the failure mechanism for the transverse module-to-module connection detail.

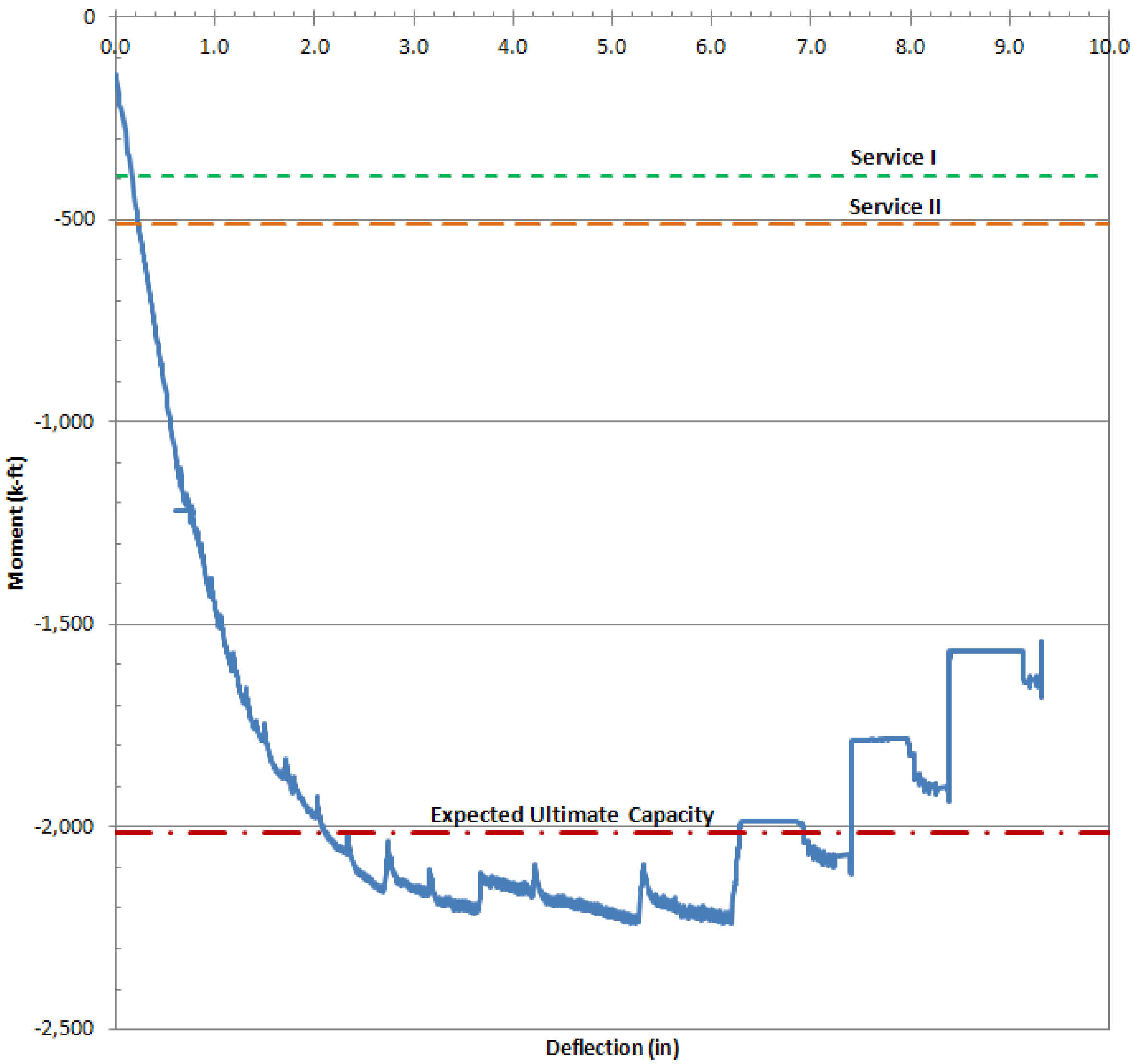

Figure 4.49. Ultimate Capacity Moment vs. Deflection

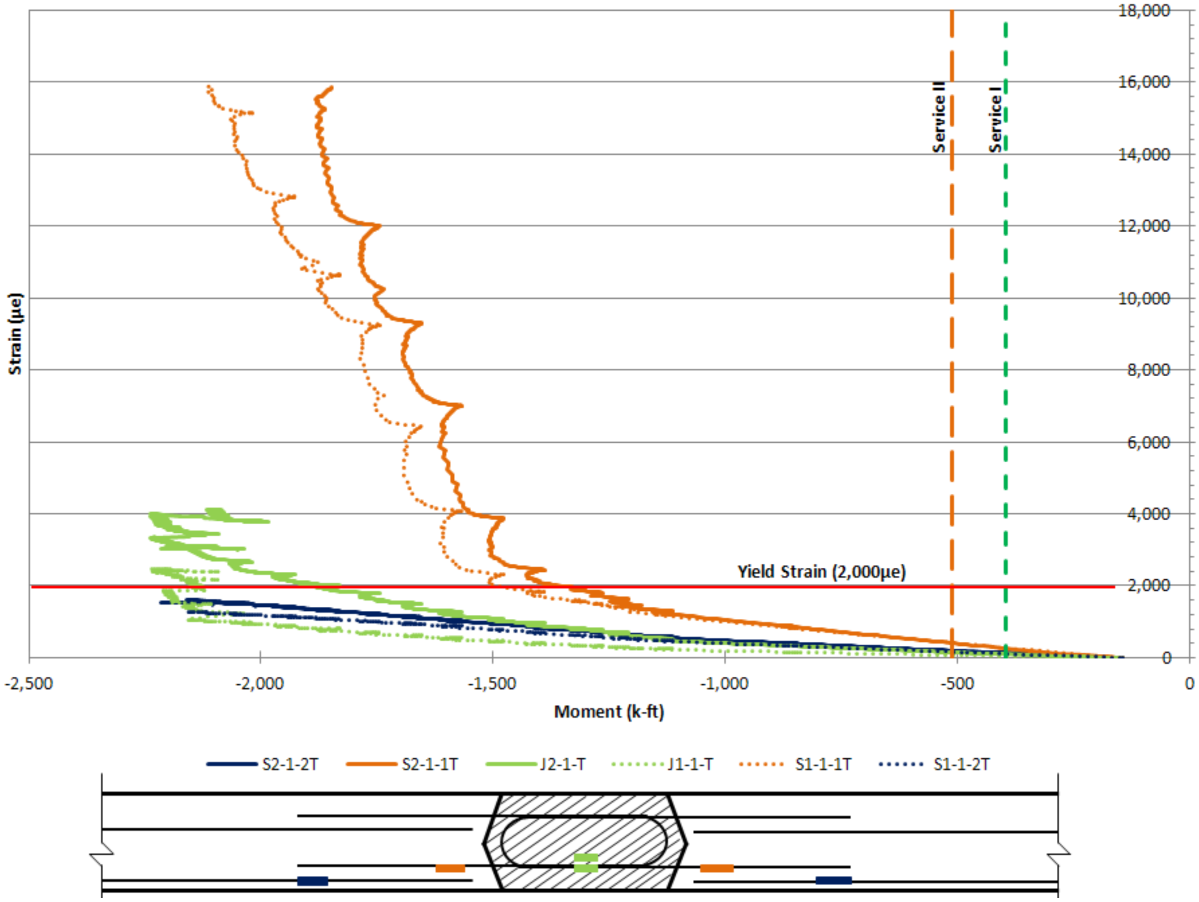

Figure 4.50. Row 1, Top-of-Deck Embedded Strain Gages (Ultimate)

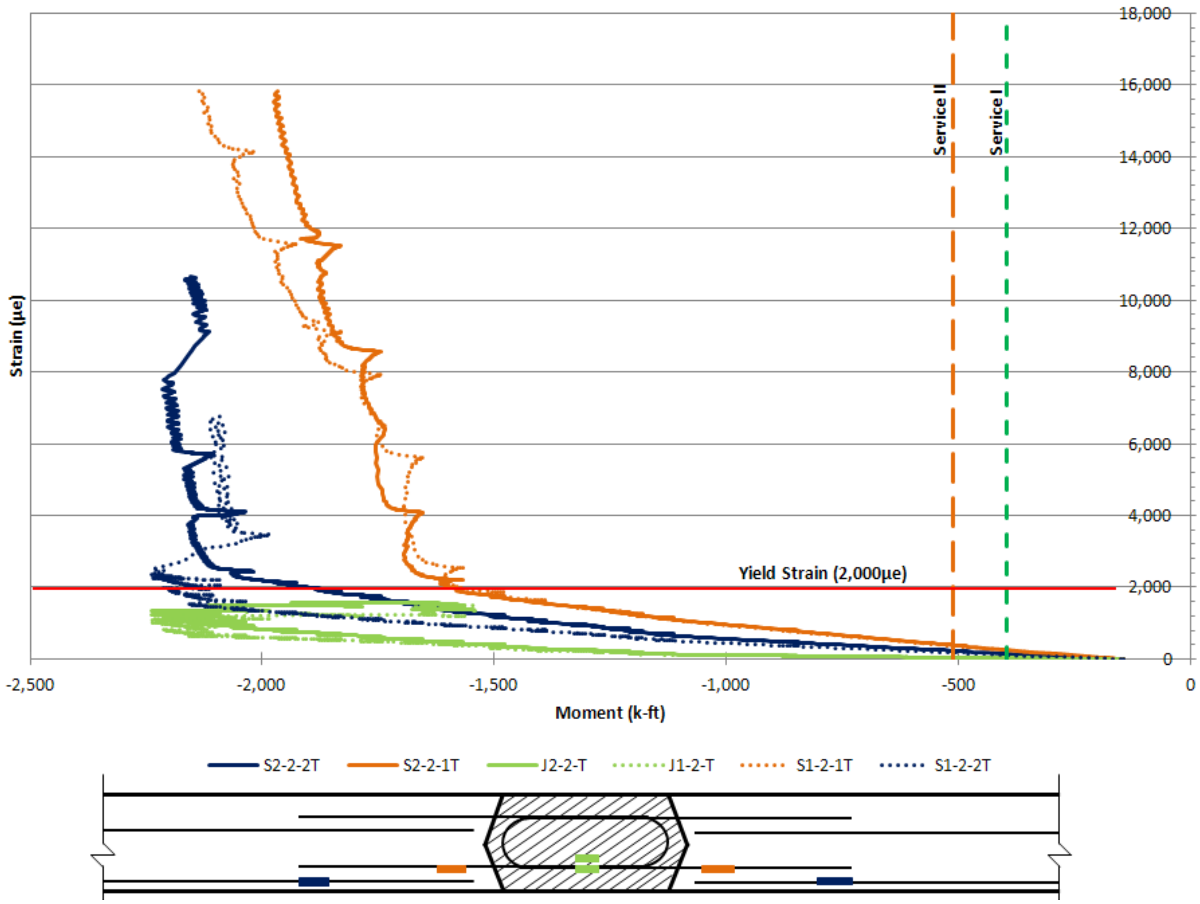

Figure 4.51. Row 2, Top-of-Deck Embedded Strain Gages (Ultimate)

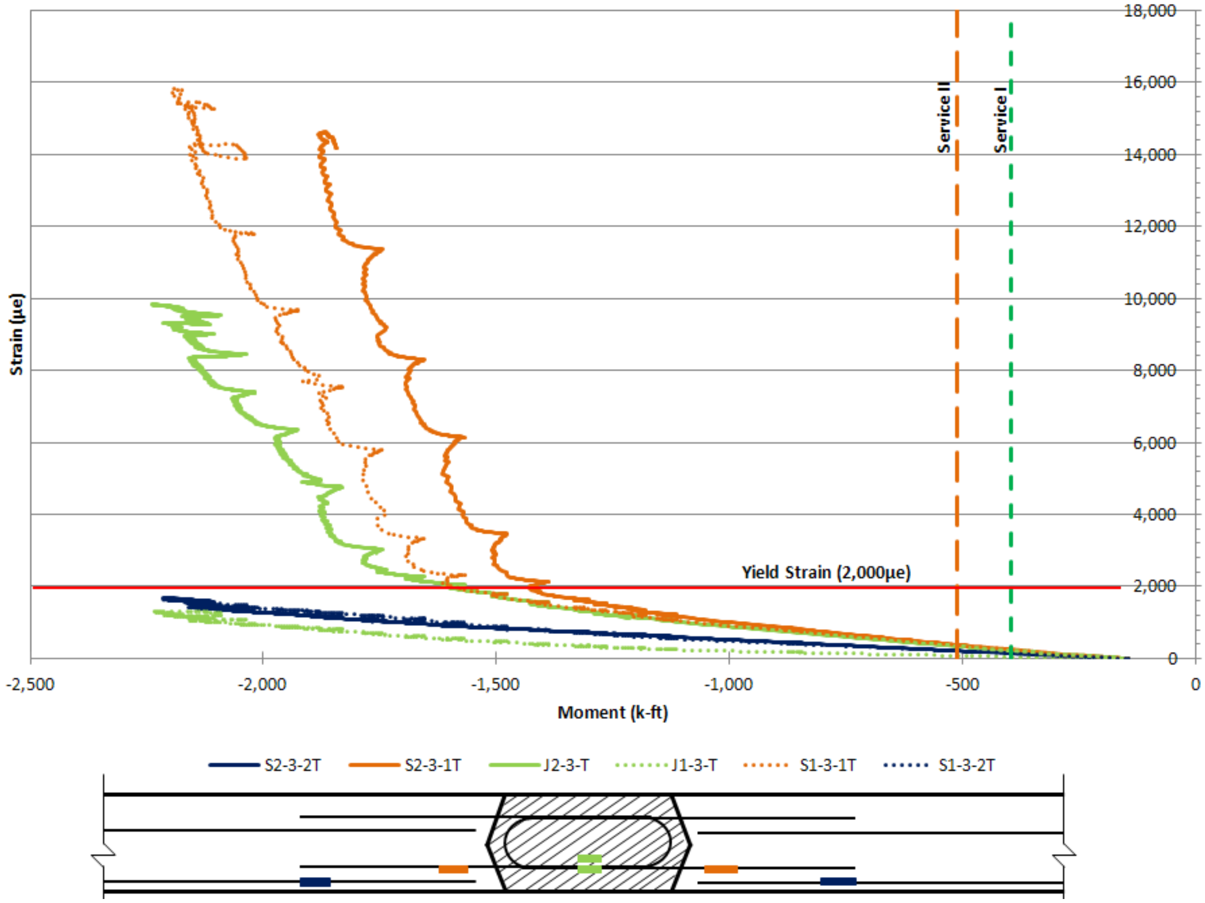

Figure 4.52. Row 3, Top-of-Deck Embedded Strain Gages (Ultimate)

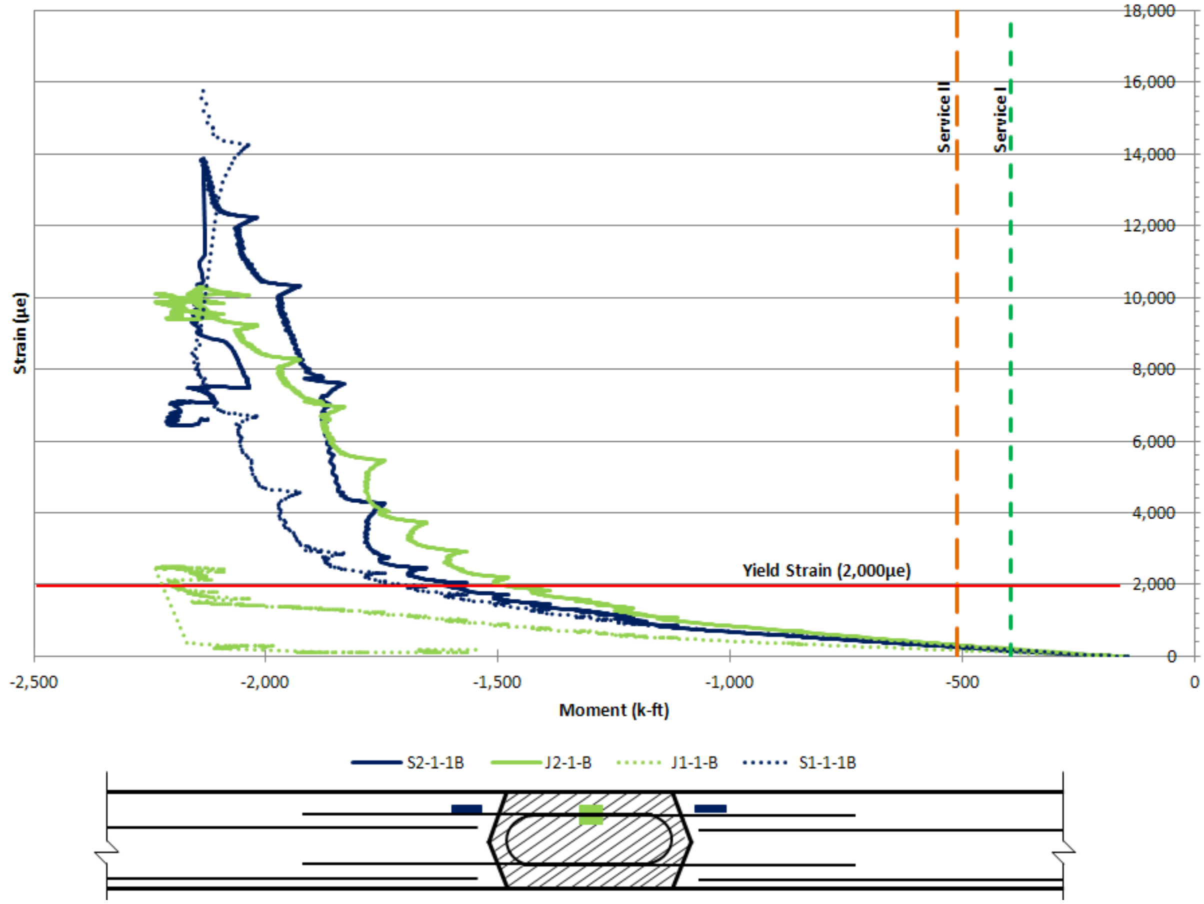

Figure 4.53. Row 1, Bottom-of-Deck Embedded Strain Gages (Ultimate)

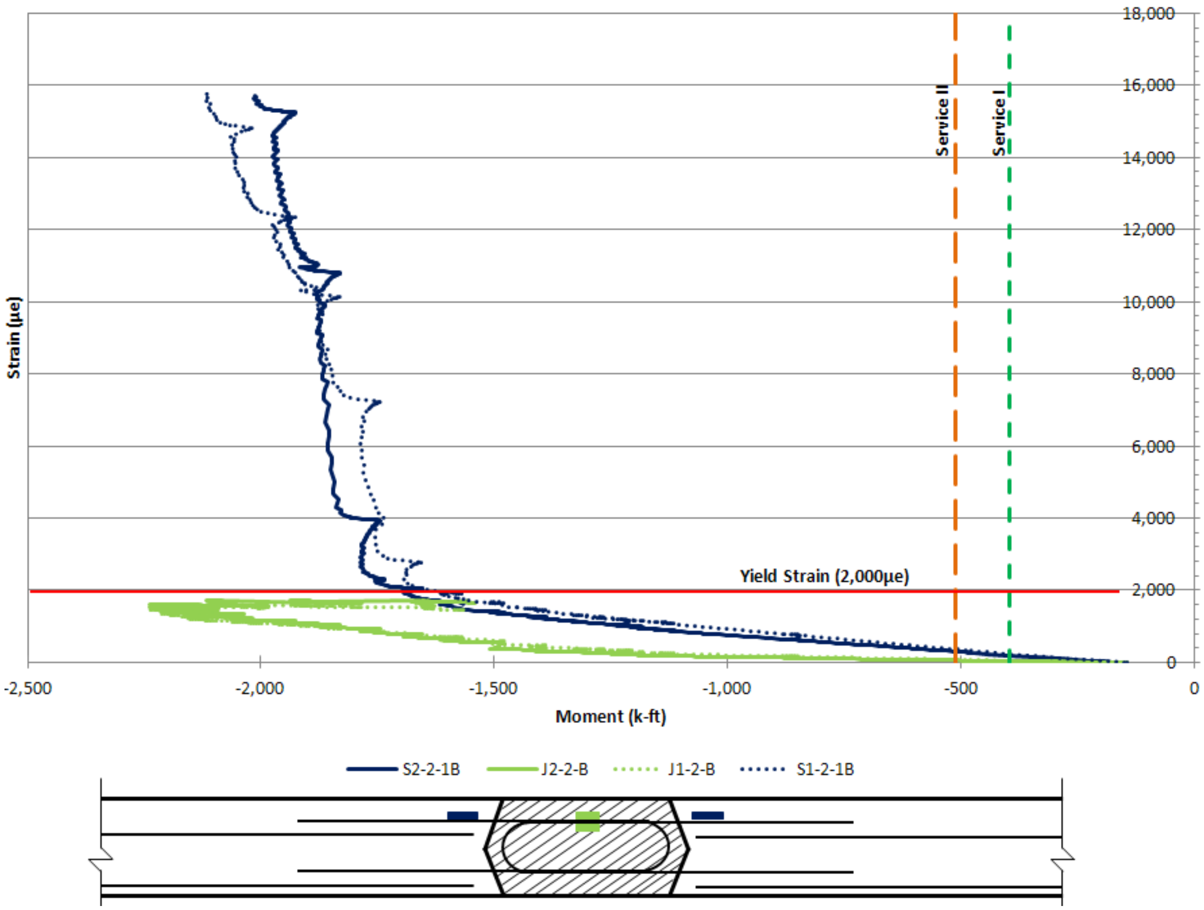

Figure 4.54. Row 2, Bottom-of-Deck Embedded Strain Gages (Ultimate)

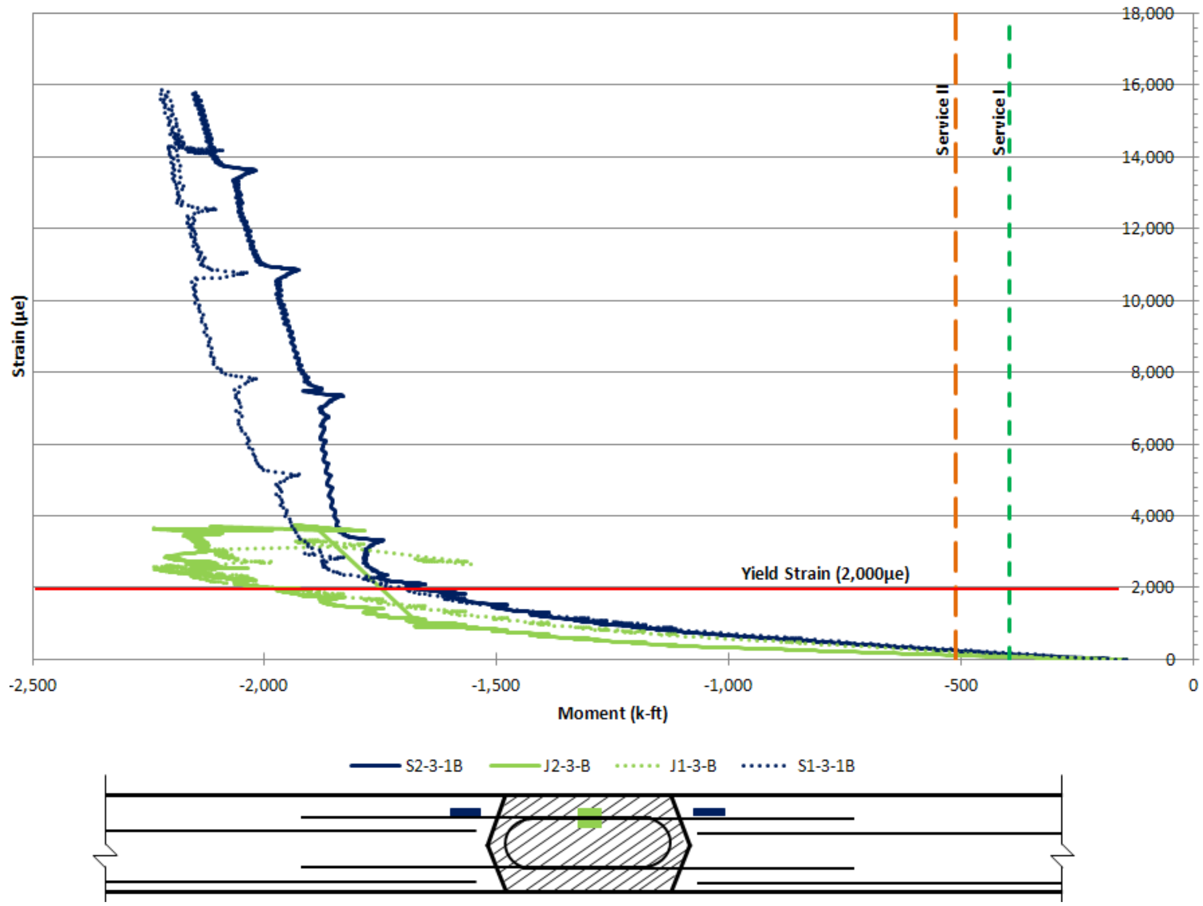

Figure 4.55. Row 3, Bottom-of-Deck Embedded Strain Gages (Ultimate)

Embedded strain gage data immediately adjacent to the joint interface in the top-of-deck reinforcement entered the inelastic range, suggesting yielding (2,000µe) at approximately 1,500 to 1,600 kip-ft of applied moment (Figure 4.50 – Figure 4.52). All gages embedded on the top-of-deck reinforcement at these locations behaved in this manner. This corresponded to the specimen entering into inelastic deformation around the same applied moment in Figure 4.49.

Similarly, bottom-of-deck embedded strain gage data immediately adjacent to the joint interface indicated yielding at approximately 1,600 to 1,800 kip-ft moment (Figure 4.53 – Figure 4.55).

With increased loading, the opening at the interface between the HPC deck and the UHPC joint widened. Cracks from service level testing propagated and widened throughout the precast deck (Figure 4.56). At 1,660 kip-ft, 2 large cracks, 1 in each deck module, spanning the entire width of the specimen became apparent approximately 1.5 in from joint interface on the bottom of the deck surface. As the specimen was pushed well beyond service level moments, reinforcement in the HPC deck near the UHPC interface began to yield. Eventually, the moment-displacement curve entered into the nonlinear region, and correspondingly, strains in reinforcement near the joint began to deform plastically (Figure 4.50 – Figure 4.55).

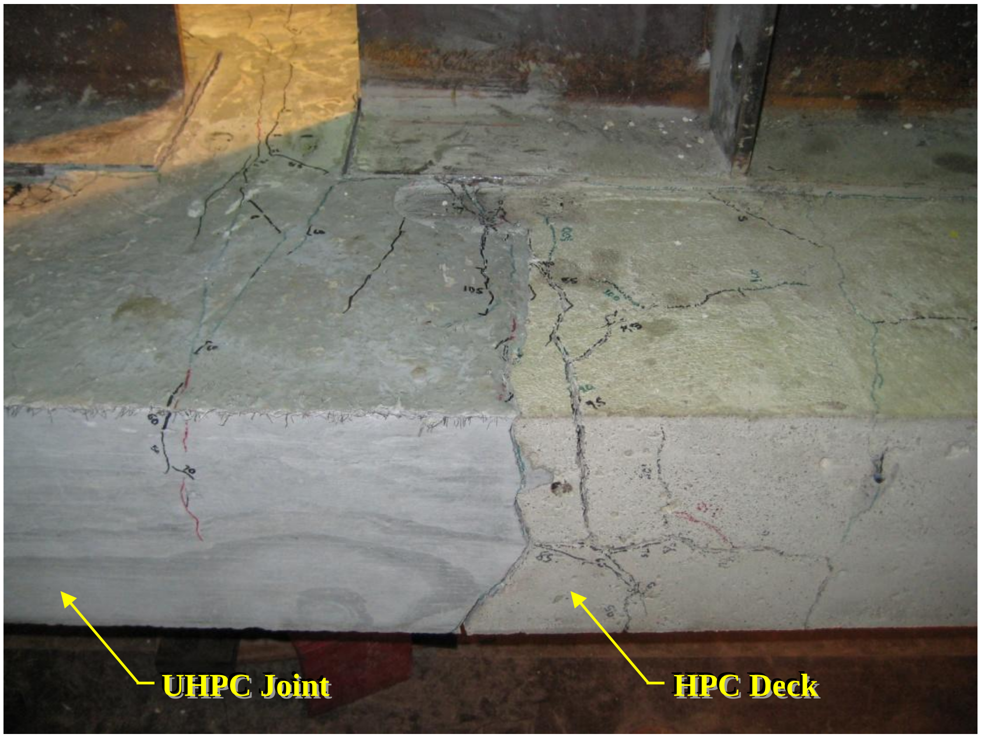

Figure 4.56. Interface Opening and Crack Propagation

Throughout ultimate moment capacity testing, the W30X99 girders appeared to be slowly pulling away from the joint. All of the cracking in the UHPC joint and HPC deck could be seen accumulated locally where the girder appeared to pull away (Figure 4.57).

Figure 4.57. Girder-Deck Interface

The two large cracks parallel to the joint interface continued to widen, and eventually the UHPC suffered tensile rupture near the shear studs located in the joint (Figure 4.58). Cracking in the precast deck exposed the outermost reinforcement hairpins that entered into the joint allowing for pullout (Figure 4.60). Load application continued, and the specimen reached a peak moment of 2,239 kip-ft before successive fractures of multiple hairpin reinforcement bars acted as the ultimate mode of failure for the transverse connection.

Figure 4.58. UHPC Rupture (Top and Bottom-of-Deck)

Figure 4.59. Bottom-of-Deck at Failure

Figure 4.60. Exterior Hairpin Reinforcement (Opposite Sides)

Deformation in the bottom flange of the W30X99 girders was identified as the test progressed and the specimen underwent large deflections (Figure 4.61). In addition, during the ultimate moment capacity testing, the slip critical bolted connections went into bearing and caused local deformation in the flange holes. Upon failure, the total deformation of the specimen at the centerline of the UHPC joint was 9.3 in (Figure 4.62).

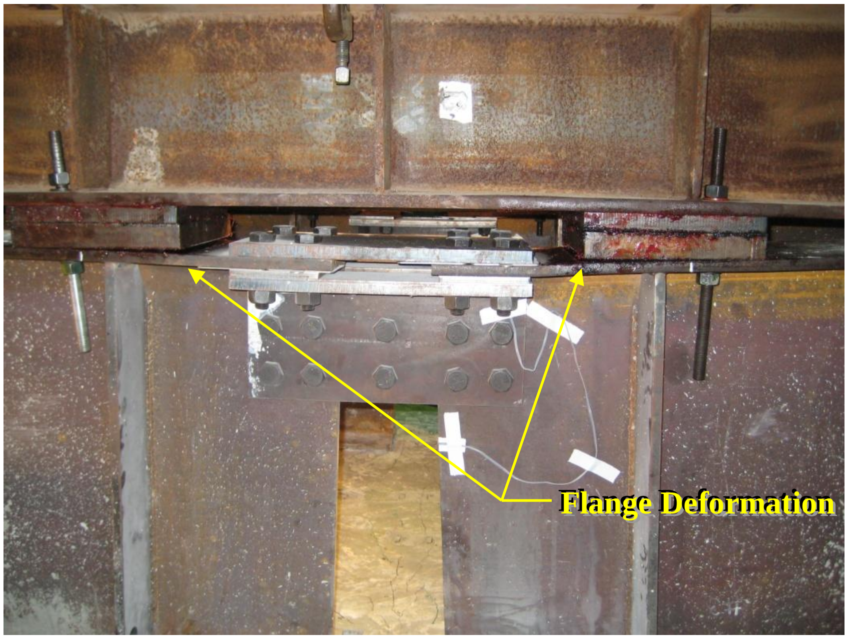

Figure 4.61. Bottom Flange Deformation

Figure 4.62. Overall Specimen Deflection

CHAPTER 5. CONCLUSIONS

This chapter presents conclusions for the abrasion, constructability, and strength and serviceability tests. The different test conclusions summarize the important issues and recommendations from the qualitative and quantitative testing data.

UHPC ABRASION

The objectives for the abrasion testing were to determine the early age grindability of the UHPC material in an effort to assist the accelerated scheduling of the contractor on the demonstration project. Several conclusions and recommendations for the joint material were made.

- Assuming a curing temperature of 70°F, the UHPC will reach the 10,000 psi compressive strength required for grinding at approximately 2 days.
- Assuming a 70°F curing temperature, the material reaches the 14,000 psi compressive strength threshold required to open the bridge to traffic after 4 days. Thus, the contractor will have roughly two days to grind the UHPC joints for the bridge deck surface prior to bridge reopening.
- Specimens with a formed surface finish exhibited less abrasion resistance than cut surfaces because the steel fibers in the UHPC were lying parallel with the surface and tended to pull off easily. Fiber alignment was attributed to material flow on the bottom surface of the mold.
- Specimens with a rough surface finish generally included small entrapped air bubbles which also allowed for easy removal of the UHPC material.
- If the demonstration bridge's field cast joints have a formed surface finish due to a plywood top form, the abrasion resistance in the field is expected to most nearly resemble that of the formed surface specimen abrasion resistance results.
- If the field cast joints have an unfinished top surface, the abrasion resistance in the field is expected to most nearly resemble that of the rough surface specimen abrasion resistance results.

- For the formed surface finish, abrasion resistance of the UHPC at 10,000 psi compressive strength will likely be about 40% lower than the abrasion resistance roughly 2 days later when the UHPC reaches 14,000 psi compressive strength.
- For the rougher, unfinished surface, abrasion resistance of the UHPC at 10,000 psi compressive strength will likely be about 27% lower than the abrasion resistance when the UHPC reaches 14,000 psi compressive strength roughly 2 days later.

JOINT CONSTRUCTABILITY

The completed construction and casting of the intersecting deck joint mock-up specimen helped formulate a proposed UHPC placement plan.

- Ambient temperatures at the time of batching are very important to the flow characteristics of the UHPC.
- When ambient temperatures were 65°F, the temperature of the UHPC upon discharge from the mixer ranged from 82 to 85°F. Within this range, adequate flow characteristics to achieve good consolidation and flow around corners were observed.
- At ambient temperatures of 75.5°F, the temperature of freshly mixed UHPC reached 100°F and the flow characteristics were inadequate for placement and consolidation.
- Consequent replacement of water with ice by mass in the batch reduced the temperature of freshly mixed UHPC to 60°F and once again allowed for acceptable flow characteristics of the UHPC.
- At satisfactory discharge temperatures, the acceptable flow characteristics created no significant voids around steel reinforcing bars at the intersection of longitudinal and transverse deck joints.
- The UHPC should be placed from areas of lowest elevation to highest while applying top forms as the deck joints are filled and a small chimney should be constructed at the highest elevation to provide hydrostatic head in the UHPC and aid material consolidation.
- Full depth stay-in-place acrylic bulkheads create a possible infiltration plane for water and chemical access to the embedded steel joint reinforcement and should be avoided if possible.

- To maintain controlled sequential placement of the UHPC and avoid infiltration planes, a partial-height removable acrylic bulkhead should be used in the longitudinal joint at locations where the UHPC material will likely be in compression.

TRANSVERSE JOINT STRENGTH AND SERVICEABILITY

Testing of the transverse module-to-module connection over the pier identified the likely cracking moment and determined the ultimate capacity of the section. Many results and recommendations were made from the testing regimen regarding serviceability of the deck over the connection.

Service Level Static Testing

- During both static and fatigue testing, surface mounted strain gages spanning the interface between the prefabricated deck modules and the UHPC joint indicated early debonding and significant opening at the interface.
- Visual observation of the interface at and below service level load conditions confirmed the early debonding and opening of the HPC/UHPC interface.
- In addition to debonding at the joint interface, embedded strain gages near the interface registered strains above the HPC cracking strain level (110με) at approximately half of Service Level I moment conditions, suggesting cracking is likely to occur in the prefabricated deck modules.

Service Level Fatigue Testing

- Visual inspection at the onset of the 1,000,000 cycle service level fatigue testing confirmed cracking the in the precast HPC deck near the joint interface.
- Strain accrual during fatigue testing suggested propagation of existing cracks in the specimen. Visual inspection throughout the fatigue testing confirmed propagation of existing cracks and formation of new, full-depth cracks in the prefabricated deck modules within 10 ft of the joint.

Connection Retrofit Testing

- To mitigate the serious durability concerns at the transverse module-to-module connection over the pier, a modified detail which would not compromise the accelerated construction aspect of the project was devised and implemented. The modified detail post-tensioned the deck in this region and to minimize tensile stresses in the concrete through service level conditions.

- Static service level testing when 60 kip post-tensioning force was applied in each of the 4 rods for the modified module connection detail reduced the tensile strain across the interface, but not sufficiently to reduce strains below HPC cracking levels prior to Service Level I conditions.
- Static service level testing indicated that the application of 70 kips of post-tensioning force negated the tensile strain across the interface entirely until after Service Level I conditions were reached.
- Strains measured with surface mounted strain gages did not exceed the HPC cracking strain until after Service Level I conditions were reached and strains measured with embedded strain gages throughout the specimen did not exceed the HPC cracking strain until after Service Level II conditions were reached with 70 kips per rod of post-tensioning force.
- The 70 kip post-tensioning force per rod was recommended for application in the demonstration bridge in order to reduce the likelihood of deck cracking over the piers and increase deck durability at the transverse joint interface.

Ultimate Capacity Testing

- The overall specimen moment vs. deflection plot indicated inelastic deformation of the specimen around 1,500 to 1,600 kip-feet of applied moment.
- Top of deck reinforcement began yielding at approximately 1,500 to 1,600 kip-feet of applied moment, corresponding to the inelastic deformation of the entire specimen.
- Bottom of deck reinforcement suggested yielding between approximately 1,600 and 1,800 kip-feet of applied moment.
- The W30X99 girders slowly pulled away from the joint. UHPC tensile rupture occurred near the shear studs located in the joint and connected with two large cracks in the HPC deck parallel to the joint interface that had formed and widened as load increased.

- Spalling at the edges in the precast deck exposed the exterior module hairpins and allowed for rebar pullout. Successive fracture of multiple hairpin reinforcement bars entering the transverse joint was the ultimate mode of failure for the connection.
- The actual ultimate moment capacity of the transverse module-to-module connection (2,239 kip-ft) was determined to be approximately 10% greater than the expected ultimate moment capacity (2,016 kip-ft).

The durability of full-depth deck joints between prefabricated panels has been a major concern for many years. Unless post-tensioned, these joints may allow penetration of water and chemicals leading to corrosion. Post-tensioning of bridge decks, however, has traditionally been a time-consuming field operation ill suited to ABC. UHPC was investigated as possible solution to this problem because its high bond strength to reinforcing bars allows narrow joints, its relatively high bond strength to precast concrete may negate the need for post-tensioning, and its low permeability enhances long-term durability. Three suites of laboratory tests were conducted to evaluate the UHPC deck joints used in the demonstration bridge. Abrasion testing was completed to assess the abrasion resistance of the cast-in-place deck joints with respect to anticipated grinding operations, a constructability test was carried out to assess the placement procedure and feasibility of the longitudinal and transverse UHPC joint intersection detail, and strength and serviceability testing was completed to quantify the cracking moment and ultimate moment capacity of the transverse module-to-module connection detail over the bridge pier.

Abrasion and maturity testing of the UHPC material indicated that when cured at 70°F, the compressive strength thresholds required for grinding (10 ksi) and opening the bridge for traffic (14 ksi) were reached at two and four days after placement of the UHPC, respectively. Thus, the contractor would have two days before grinding could commence and two days to complete grinding prior to reopening the bridge to traffic. Abrasion resistance increased by roughly 40% for the UHPC material from two to four days, emphasizing the advantages in time and equipment to grinding the joints as early as possible.

A UHPC placement procedure tailored to the demonstration bridge was developed from the findings of the constructability testing for the longitudinal and transverse joint intersection detail. Partial-height, removable bulkheads were recommended in order to control the placement of the UHPC in the deck joints. In addition, the sensitivity of the UHPC mix design to ambient air temperatures was identified while batching for the laboratory tests. Provided that the UHPC's sensitivity to ambient temperature effects were accounted for, the UHPC exhibited excellent flow characteristics and consolidation during placement of the intersecting deck joints. In addition, the accelerated rate of compressive strength gain and higher cracking strain level relative to regular concrete proved useful for application in this ABC project.

While the UHPC displayed several superior material characteristics with respect to the durability and strength of the deck joints themselves, the direct tensile bond strength between the UHPC and the precast, high performance concrete (HPC) deck observed during the strength and serviceability testing was a concern. Testing revealed that the interface between the transverse UHPC joint and HPC deck underwent early debonding and significant opening well below service level moment conditions. This raised concerns as to the durability of the module-to-module transverse joint connection for the demonstration bridge. Consequently, a post-tensioned retrofit detail was developed and tested in an effort to eliminate opening at the interface and cracking in the HPC deck immediately adjacent to the transverse joint over the pier. With adequate post-tensioning force, the retrofit detail successfully limited strains levels to below the cracking threshold of the HPC.

Due to the interfacial bond issues observed over the course of this testing, further investigation into the direct tensile bond strength between the UHPC and HPC is recommended. This testing would better evaluate the durability of the longitudinal and transverse UHPC deck joints present in the ABC demonstration bridge and help to determine the long-term viability of this UHPC deck joint detail as a solution in future ABC projects.

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APPENDICES

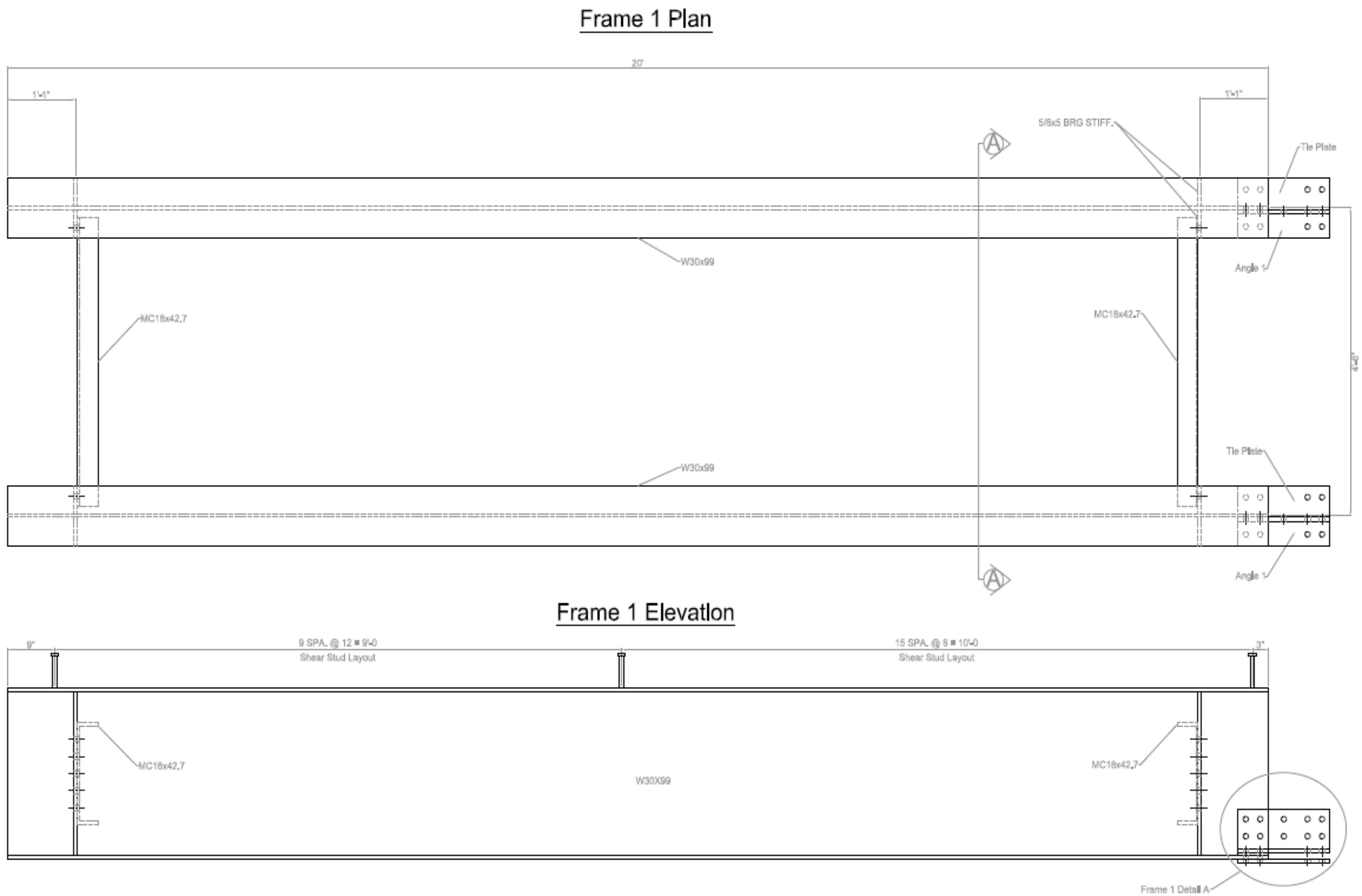


Figure A.1. Steel Frame 1

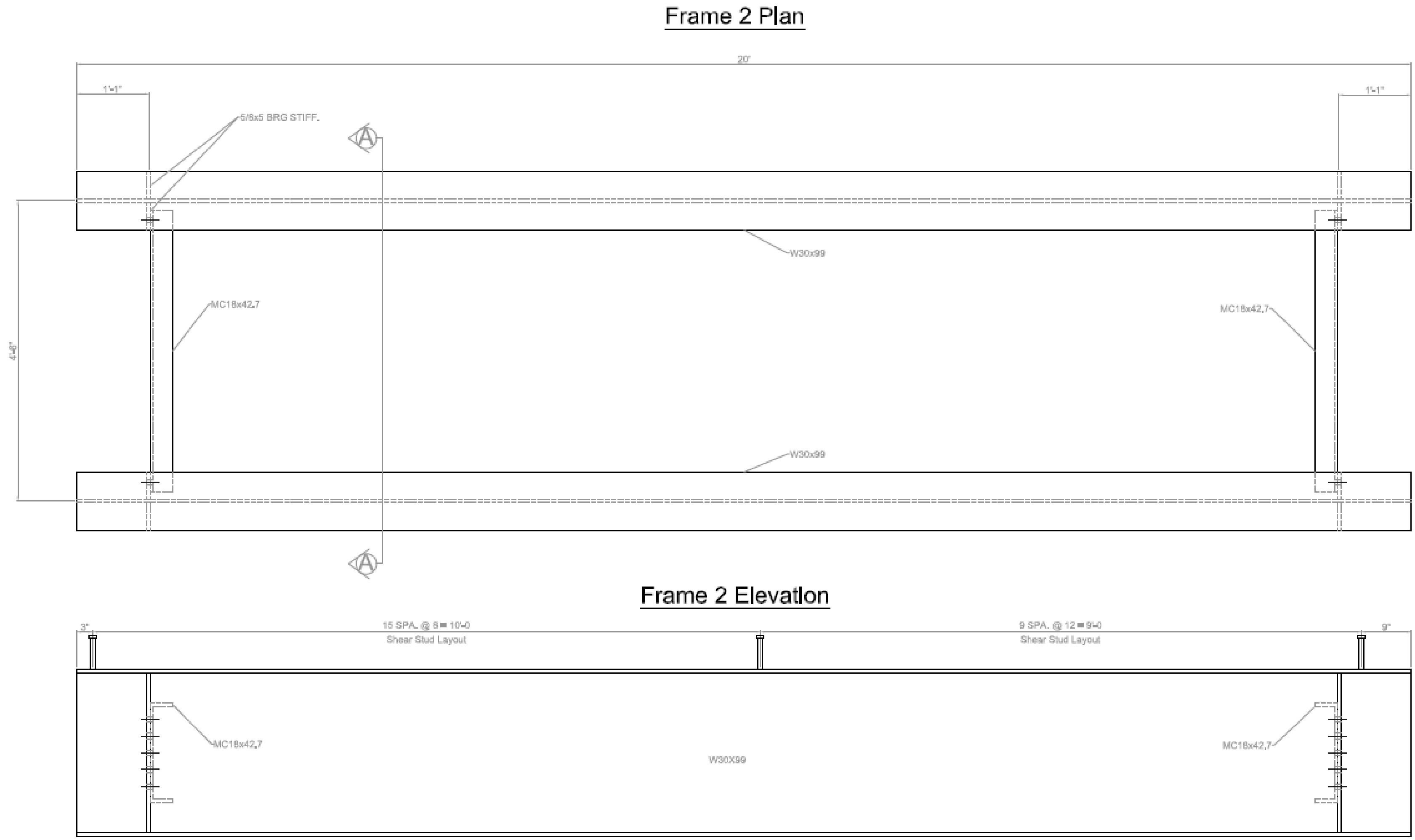
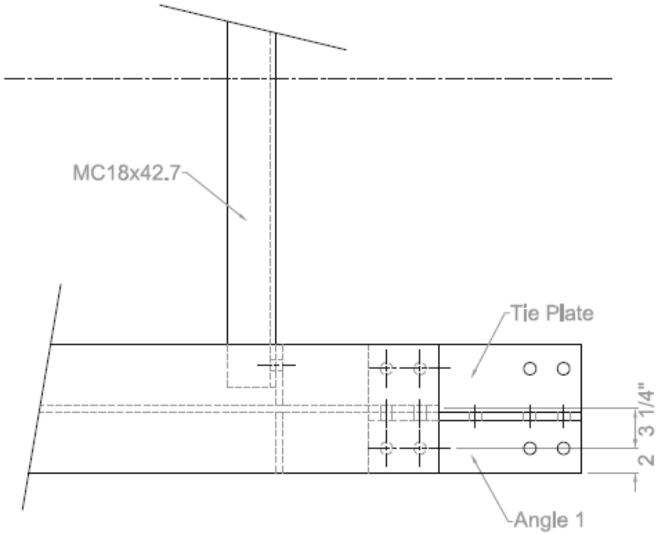


Figure A.2. Steel Frame 2



GENERAL NOTES:

1. STRUCTURAL STEEL SHALL BE ASTM A709 GRADE 50W

2. ALL BOLTS SHALL BE HIGH-STRENGTH ASTM A325 TYPE III BOLTS

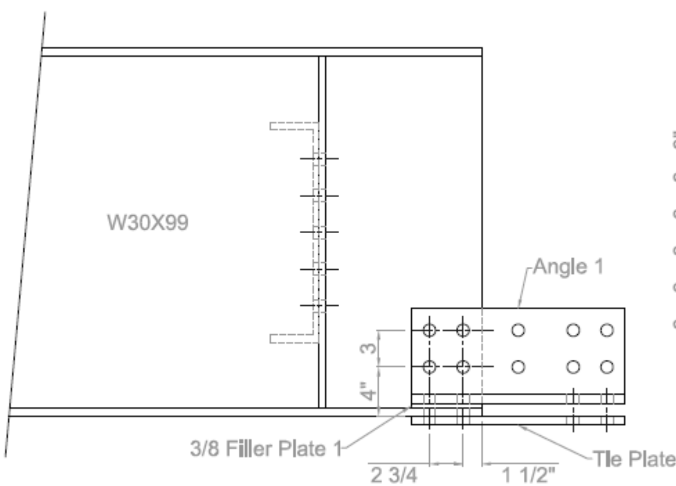
3. ALL NUTS SHALL BE ASTM A563 HEAVY HEX NUT GRADE DH3

4. ALL WASHERS SHALL BE ASTM F436 TYPE III WASHERS

5. ALL OPEN HOLES ARE TO BE 15/16" DIAMETER AND ALL BOLTS ARE TO BE 7/8" DIAMETER.
   BOLT LENGTHS:
   - 3 1/2" LONG THROUGH WEB
   - 4" LONG THROUGH FLANGE
   - 2 1/2" LONG THROUGH DIAPHRAGM

6. AWS PREQUALIFIED WELDED JOINTS, AND UNLESS OTHERWISE NOTED THE DESIGN JOINT DETAILS ARE FOR MANUAL SHIELDED METAL-ARC WELDING.

Frame 1 Detail A



Section A-A: Frame 1 & 2

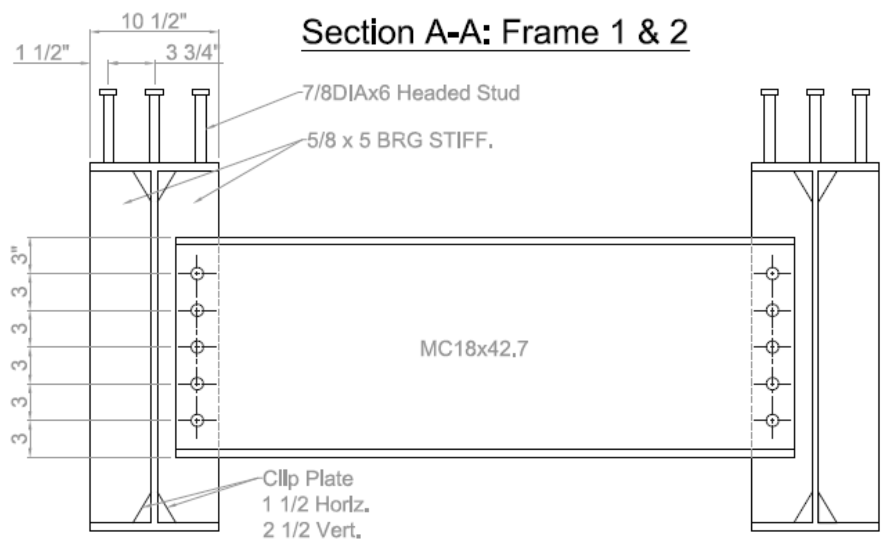
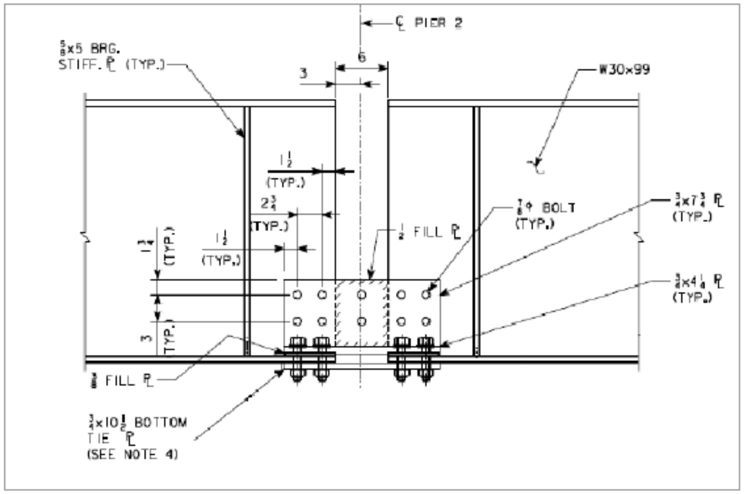
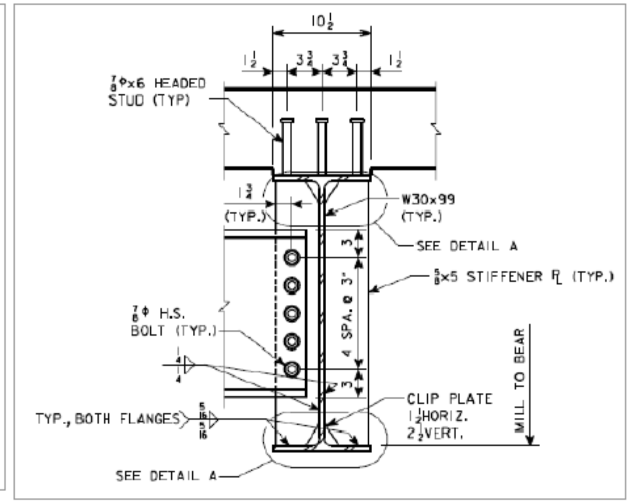


Figure A.3. Frame Details 1

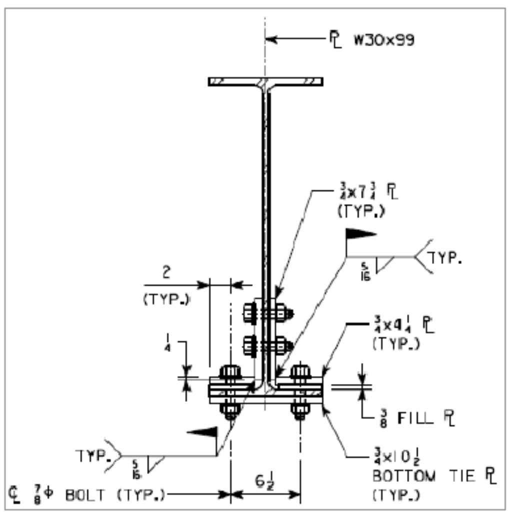
Connected Frame Elevation



Diaphragm Connection Detail



Angle Section Detail



Stiffener Weld Termination Detail

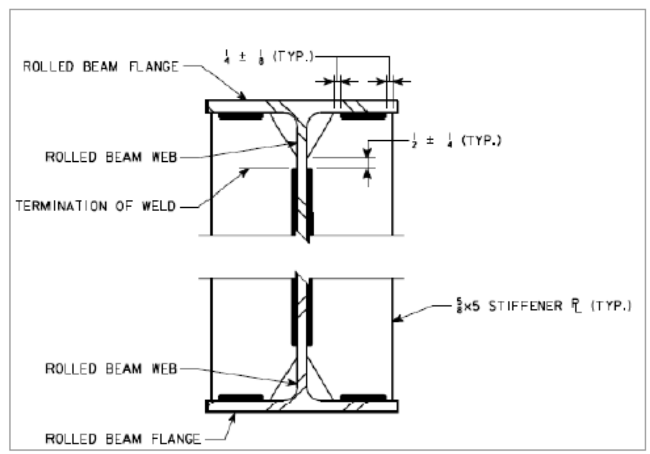


Figure A.4. Frame Details 2

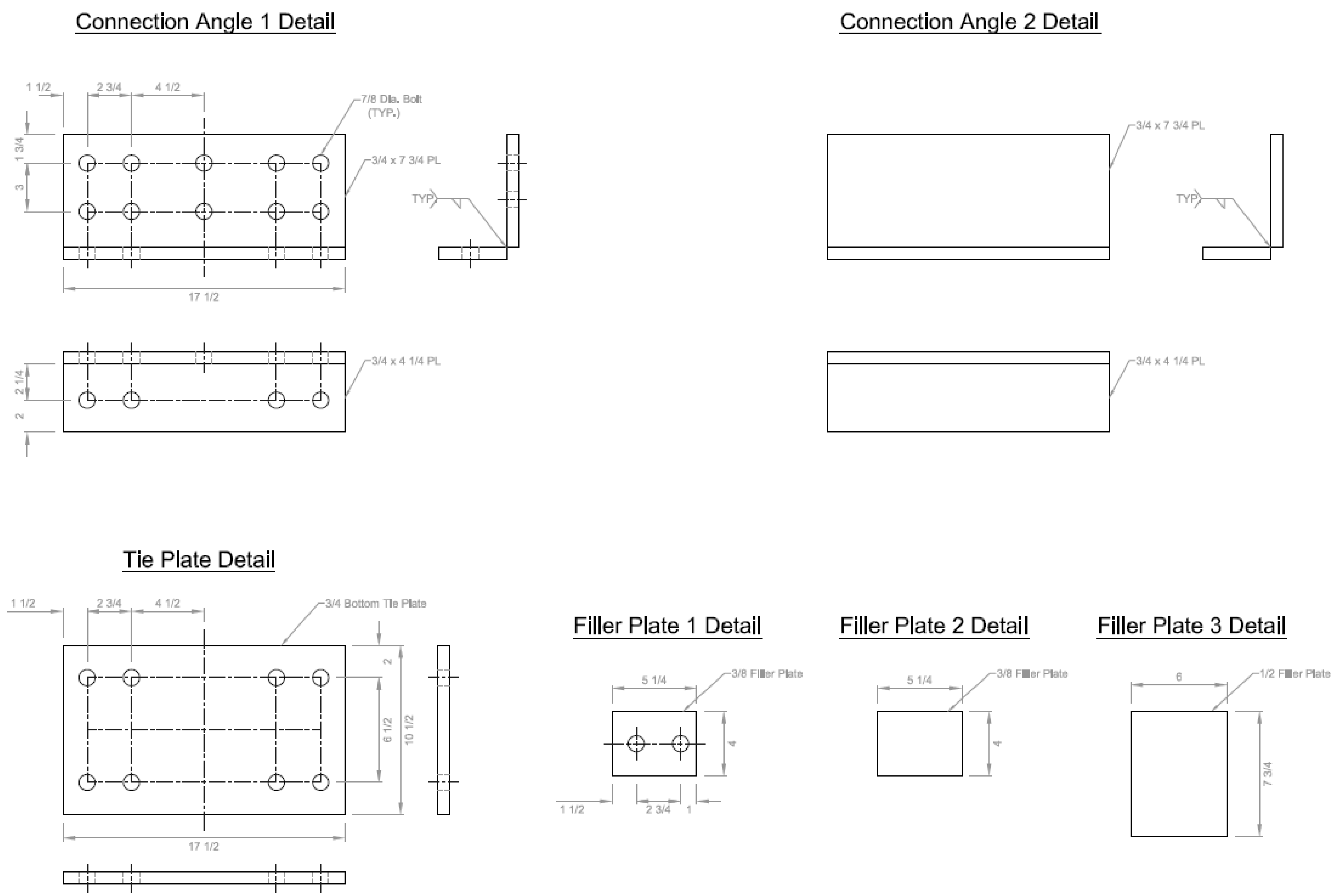


Figure A.5. Connection Details

<u>Connection Fabrication Notes:</u>

"**Frame 1**" shall have all holes drilled in the webbing and flanges for "Angle 1" and "Tie Plates" to be attached to.

"**Frame 2**" shall be fabricated without holes drilled in the webbing and flanges (as detailed).

Two (2) "**Angle 1**" pieces shall be fabricated with holes as detailed and be bolted to "Frame 1" on both W30x99 beams.

Two (2) "**Angle 2**" pieces shall be fabricated without holes as detailed.

Two (2) "**Tie Plates**" shall be fabricated with holes as detailed and attached to "Frame 1" on both W30x99 beams.

Two (2) "**Filler Plate 1**" pieces shall be fabricated with holes as detailed and attached between "Angle 1" pieces and W30x99 flanges (See "Frame 1 Detail A").

Six (6) "**Filler Plate 2**" pieces shall be fabricated without holes as detailed.

Two (2) "**Filler Plate 3**" pieces shall be fabricated without holes as detailed.

<u>Final Deliverables:</u>

One (1) - Frame 1 *(Includes: two (2) Angle 1 pieces, two (2) Filler Plate 1 pieces and two (2) Tie Plates)*

One (1) - Frame 2

Two (2) - Angle 2's

Six (6) - Filler Plate 2's

Two (2) - Filler Plate 3's

Twenty (20) - 7/8" Diameter, 3 ½" Long, High Strength ASTM A325 Type III Bolts (Total)

Sixteen (16) - 7/8" Diameter, 4" Long, High Strength ASTM A325 Type III Bolts (Total)

Forty (40) - 7/8" Diameter, 2 ½" Long, High Strength ASTM A325 Type III Bolts (Total)

Seventy-Six (76) - ASTM A563 Heavy Hex Nuts Grade DH3 (Total)

Seventy-Six (76) - ASTM F436 Type III Washers (Total)

Figure A.6. Fabrication Notes

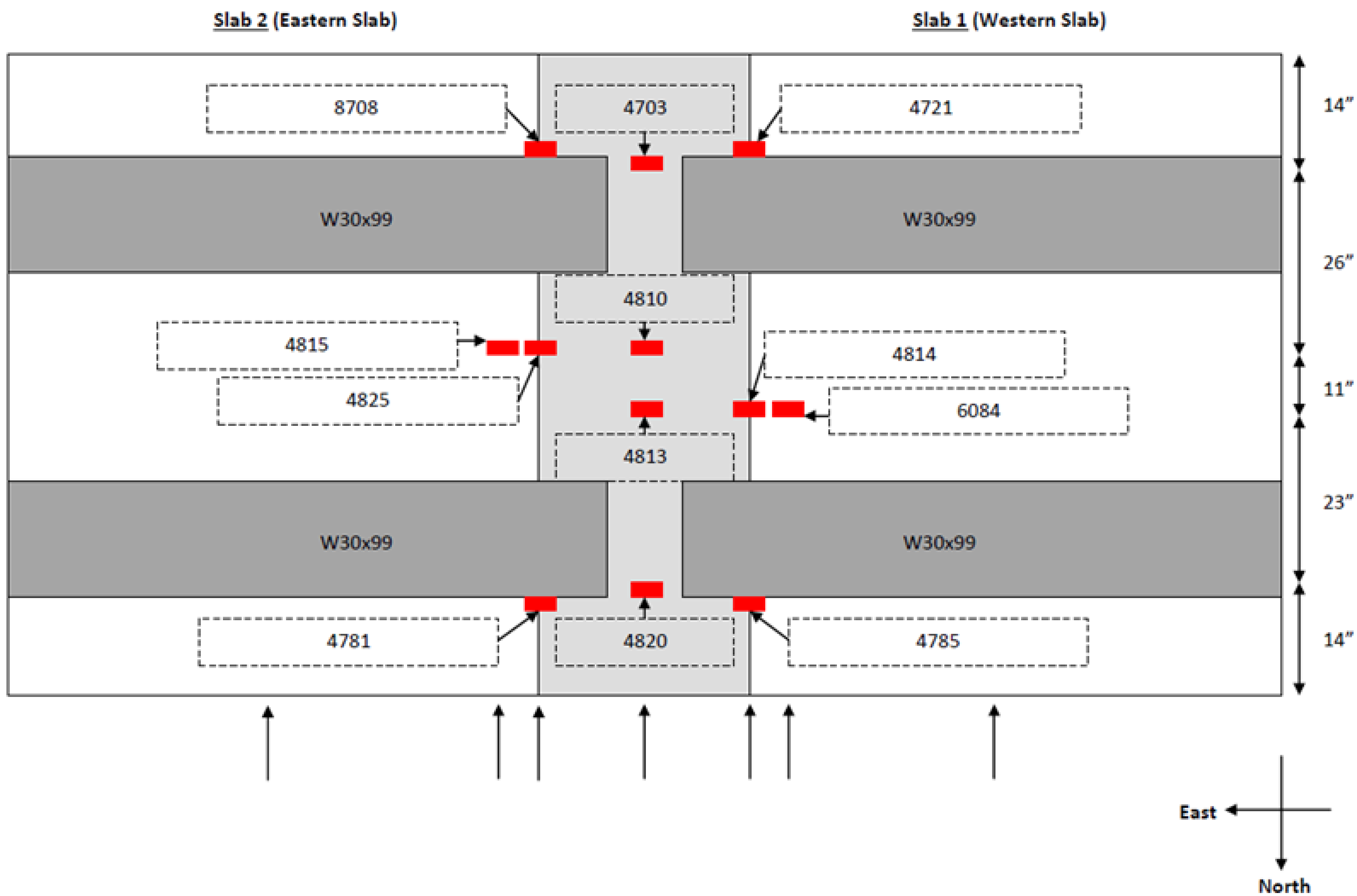


Figure B.7. Bottom-of-Deck Surface Mounted Strain Gages

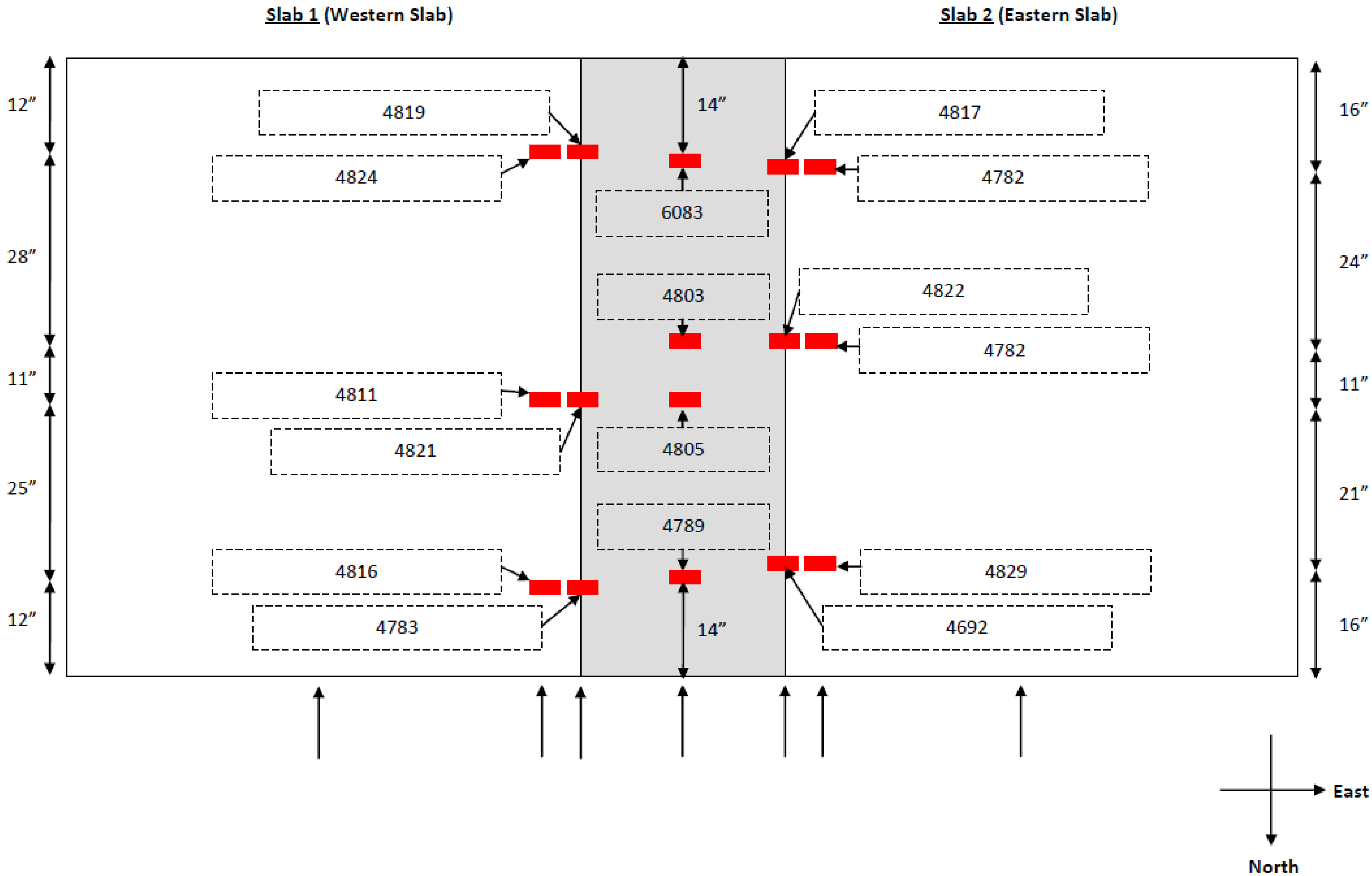


Figure B.8. Top-of-Deck Surface Mounted Strain Gages

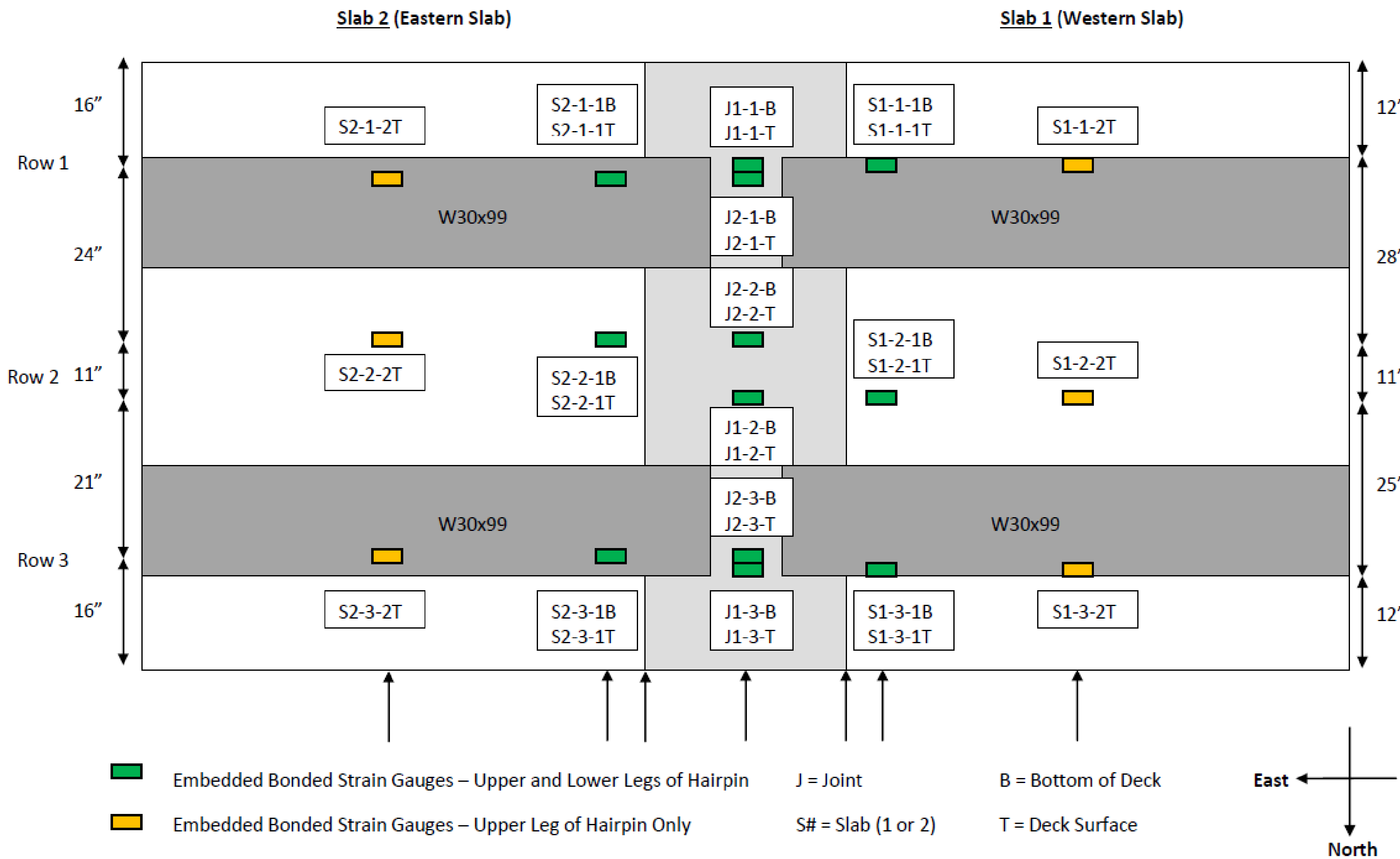


Figure B.9. Embedded Strain Gages

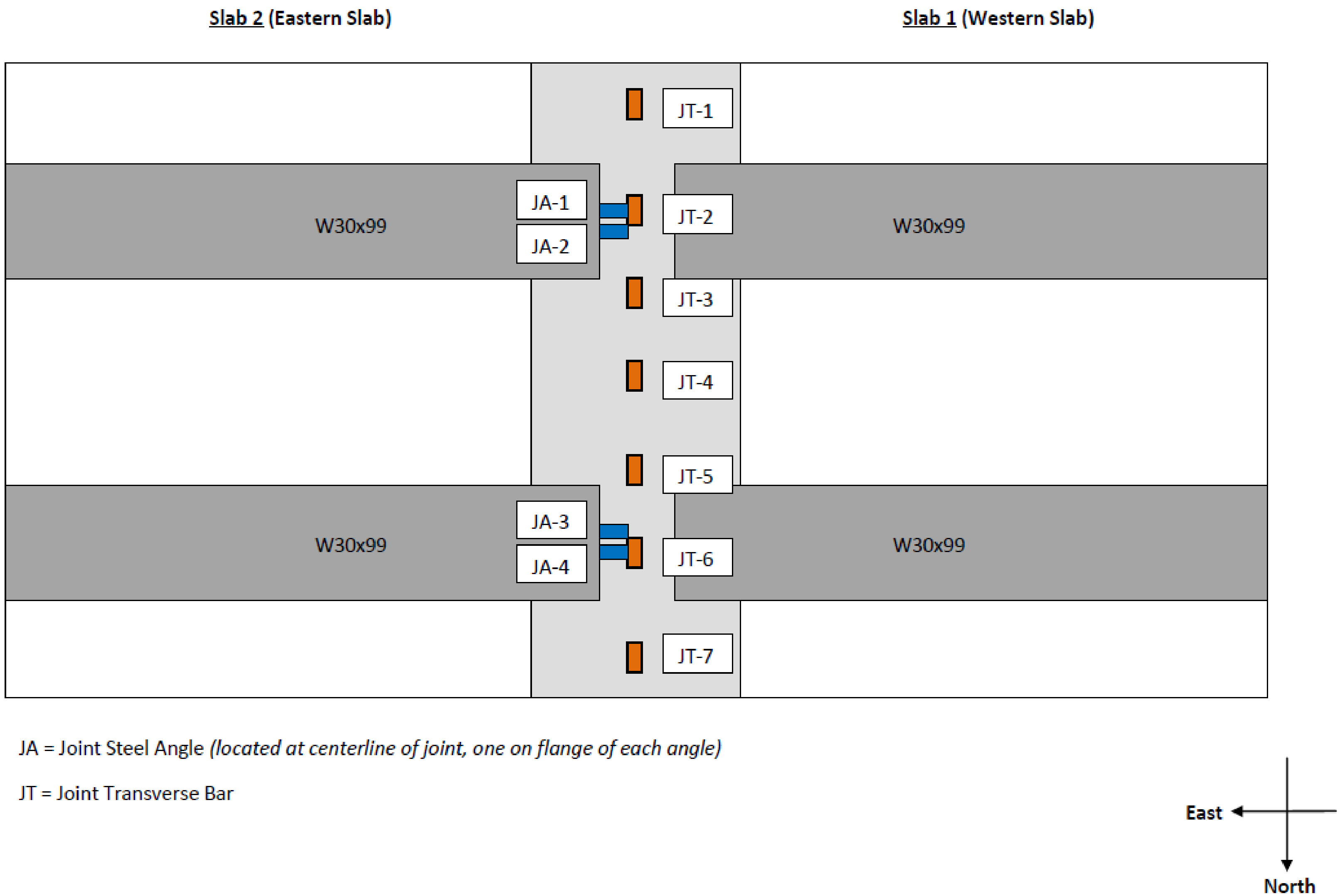


Figure B.10. Embedded Transverse Bar Gages & Steel Angle Gages

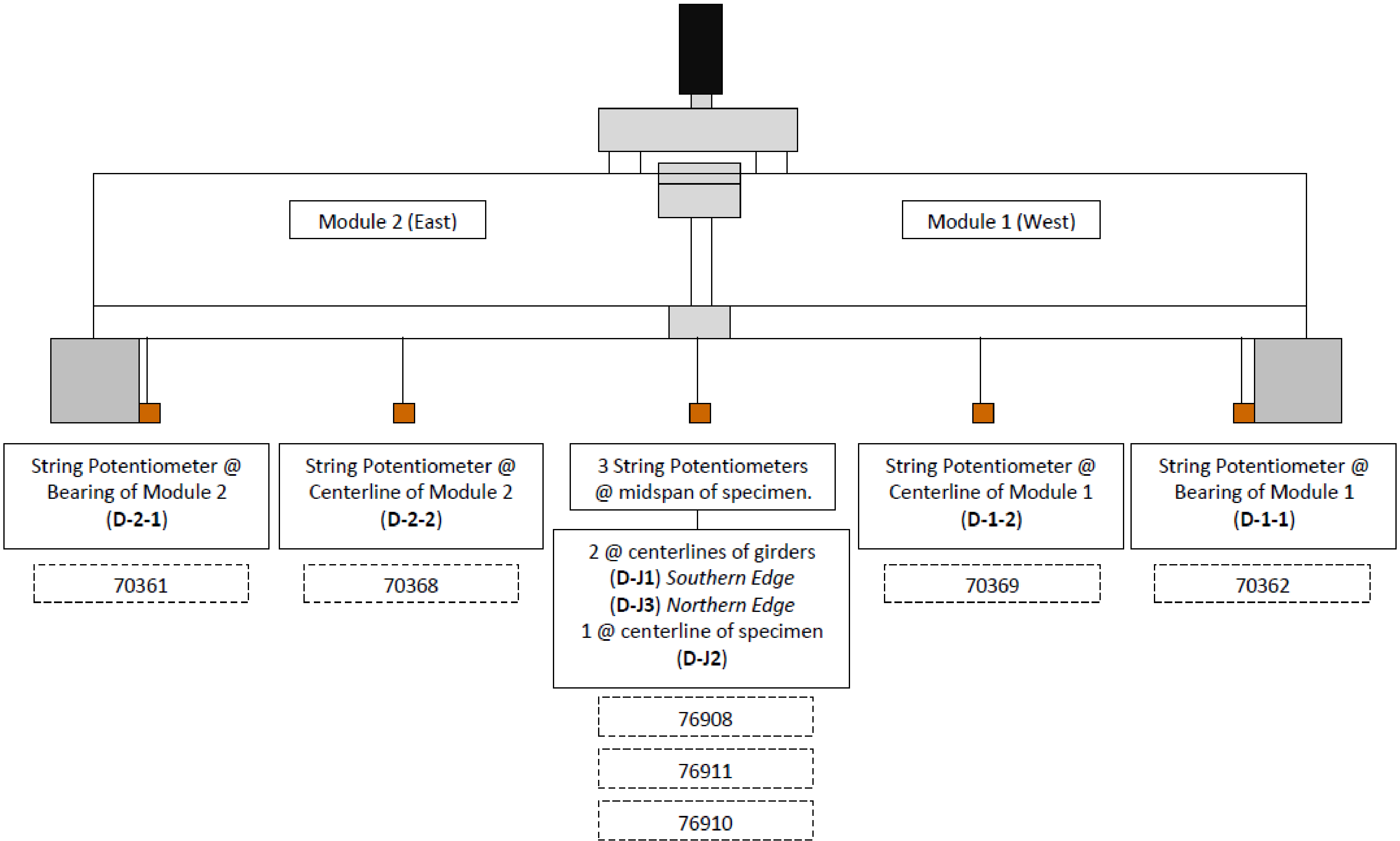


Figure B.11. String Potentiometers