

Evaluation, Laboratory Testing, Construction Documentation, and Field Testing/Monitoring of the US 52 Overflow Bridge over the Mississippi River

**Final Report
July 2020**



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EXECUTIVE SUMMARY

The objectives of this project were to validate design assumptions and evaluate the performance of the structural components and construction approaches provided in the design documents for a pretensioned prestressed concrete beam (PPCB)-supported partial-depth precast deck system with precast overhang panels.

The researchers conducted laboratory tests to evaluate the deck system and overhang panels in order to: 1) evaluate the sufficiency of the proposed polyethylene bearing between the precast deck panels and the supporting girders, 2) verify the sufficiency and constructability of using flexible polyethylene foam as a sealing strip in conjunction with leveling bolts to create an adjustable haunch, and 3) evaluate the composite action between the cast-in-place (CIP) concrete deck and precast panel. The laboratory specimen details and construction work were carefully documented.

Test results indicated the following:

- The high-density polyethylene foam has sufficient stiffness and strength to support the precast deck panels and the construction load during concrete deck placement.
- The leveling bolt with normal polyethylene foam works fine to support the deck panel and resist the lateral concrete load.
- The gluing work of the polyethylene foam needs additional attention to create a strong connection between the girder/deck concrete surface and the polyethylene foam.

The load test results indicated that both types of interior panels have an ultimate load capacity of about 240 to 250 kips when loaded under a point load until punching shear failure, which is not likely to happen during a bridge service life. The composite action between the CIP and precast deck concrete is functional through the load application process, and no debonding or sliding was found at the horizontal interface between the CIP and precast concrete.

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CHAPTER 1. INTRODUCTION

1.1 Background and Problem Statement

The US 52 overflow bridge over the Mississippi River was planned to be replaced with a pretensioned prestressed concrete beam (PPCB) bridge in 2017. The structural systems planned for the new bridge included integral abutments, a drilled shaft-supported concrete pier cap system, and a PPCB and concrete deck system.

The 376 ft long and 40 ft wide bridge was longer and wider than the bridge it was to replace. The additional width conveniently allowed for the construction of the drilled shaft foundations of the new bridge in the river overflow channel while keeping the existing bridge open to normal traffic.

To further minimize construction time and traffic interruption, multiple innovative accelerated bridge construction (ABC) techniques were proposed for use in this replacement project. The following construction options and their corresponding design plans were provided:

1. **Base design:** For all options, a base design which included partial-depth precast prestressed deck panels and a double-panel precast overhang system to facilitate construction of the bridge deck and cast-in-place (CIP) pier caps were used. Partial-depth precast deck panels have been used in Iowa on low-volume roadways in the past. In those cases, the partial-depth panels span between the precast, prestressed concrete (PPC) beams, and the overhang is formed and cast conventionally with the CIP slab. To accelerate the construction, precast overhang panels have been developed based on a design previously used by the Texas Department of Transportation (TxDOT). The overhang panels are a combination of partial-depth and full-depth components. The full-depth area is under the barrier rail to properly develop the anchorage of the barrier rail reinforcement.
2. **Deck options: The contractor was able to choose between the following deck options.**
 - a. **AccelBridge deck system:** The patented full-depth precast deck system is used in this design with CIP pier caps. The necessary compression between precast deck panels are provided by “deck jacking,” which eliminates the need for conventional post tensioning.
 - b. **Partial-depth precast deck**
3. **Pier cap options: The contractor was able to choose between the following pier cap options.**
 - a. **CIP pier cap**
 - b. **A precast pier cap**

The design of the deck system, either the partial-depth panels with the precast overhang or the full-depth precast AccelBridge deck system option, is unique and had not been used in Iowa in the past. The precast/post-tensioned pier cap is also an innovative ABC approach that had not been used in Iowa before.

To further understand/validate the design assumptions and identify potential constructability issues, the original research tasks were proposed to study the behavior of the PPCB-supported deck system and the precast/post-tensioned pier cap system. The research plan was intentionally developed with multiple options so the research team could adapt the research based on the specific bridge system selected by the contractor from the above options.

Although the bridge was first designed with a partial-depth precast deck panel and a propped cantilever precast overhang deck panel, after initiation of the research project, the contractor submitted a value engineering (VE) proposal to change the bridge system to a conventional prestressed concrete beam with a CIP deck. To incorporate this change into the research project, the contractor was interviewed to gain additional information on the VE proposal, the details of which are included in Chapter 3.

After discussion with the technical advisory committee (TAC), the original research plan was adjusted and the scope of the laboratory tests were to include the partial-depth precast deck panel system with a propped cantilever precast overhang deck panel. The revised research objectives and research plan are presented in Sections 1.2 and 1.3.

1.2 Objectives

The objectives of this project were to validate design assumptions and evaluate the performance of the structural design details for a PPCB-supported partial-depth precast deck panel system with propped cantilever precast overhang deck panels.

1.3 Research Plan

To meet the objectives of the project, three main work tasks were identified. All of the tasks were performed in very close communication with the TAC.

Task 1: Literature Review. Given the bridge construction approach was let with various structural design details that the contractor could choose from to accomplish the construction within the critical closure, the literature review was conducted with respect to all possible options including: 1) PPCB-supported partial-depth precast deck system with precast overhang panels, 2) PPCB-supported AccelBridge deck system, and 3) precast/post-tensioned pier cap system.

Task 2: Field Information Collection. The field information was collected via two means: construction documentation and interviewing the contractor. During construction of the bridge, a webcam was placed at the site to document the entire bridge construction process. This step was designed to provide live updates of the construction activities and store images at regular intervals. In addition to the webcam, the contractor was interviewed to understand the major reasons resulting in their final decision to change the deck system.

Task 3: Laboratory Testing. The experimental tests were performed on the PPCB-supported partial-depth precast deck system and precast overhang panels. Two small-scale superstructures that included all the critical components of interest were designed, constructed, and tested. The specific interests of conducting these experiments were identified as follows:

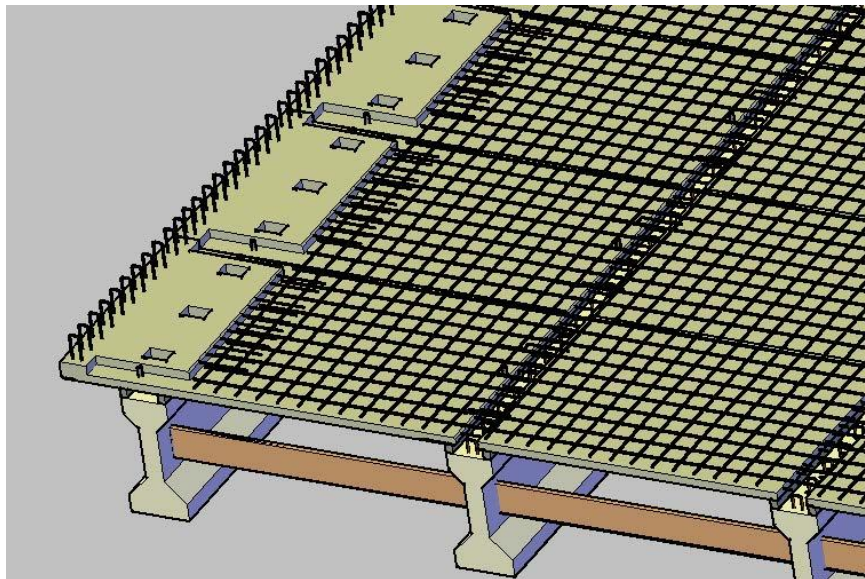
- Verify the sufficiency of the proposed polyethylene bearing between the precast deck panels and the supporting girders
- Verify the sufficiency of using flexible polyethylene foam to create an adjustable haunch form, its strength to resist the lateral pressure from CIP concrete, and the constructability of using the sealing strip in conjunction with leveling bolts
- Validate composite action between the CIP concrete deck and the precast panel by investigating the behavior of the deck panels under vertical and horizontal loads

CHAPTER 2. LITERATURE REVIEW

As the bridge construction approach was not finalized at the beginning of the research project, the literature review was conducted with respect to all possible options: 1) PPCB-supported partial-depth precast deck system, 2) PPCB-supported full-depth precast deck overhang, 3) PPCB-supported AccelBridge deck system, and 4) precast/post-tensioned pier cap system. This chapter includes background information on these methods, including previous research conducted and associated construction practices.

2.1 PPCB-Supported Partial-Depth Precast Deck Panel System

The first use of partial-depth precast concrete dates back to the 1950s in Illinois. Traditional partial-depth precast concrete deck panels are thin (typically 3.5 in. thick) prestressed concrete panels that span between girders (see Figure 1).



Ley et al. 2010, Oklahoma Transportation Center

Figure 1. Precast partial-depth deck and precast deck overhang

This system avoids the use of stay-in-place forms for a CIP concrete deck. The panels are pretensioned with strands located at mid-depth running in the transverse direction. After placing the prestressed panels, the top layer of reinforcing steel is placed and the CIP concrete portion of the deck is cast on top of the panels (Goldberg 1987).

Hieber et al. (2005) summarized the use of four commonly used precast concrete systems for rapid bridge construction outside of Washington, including precast partial-depth deck systems. The report presents four key issues associated with the use of partial-depth deck panels: 1) bearing of the partial-depth panels on the supporting girders, 2) extension of prestressing strands out of the precast panel, 3) development of prestressing within panels, and 4) composite action between CIP concrete and precast panels. To provide sufficient composite action with the CIP

concrete, the use of a roughened surface on top of the precast panels was commonly suggested (AASHTO 1998, Fagundo et al. 1985, and Goldberg 1987). The Washington State DOT (WSDOT 1998) recommends that panels be raked in the direction parallel to the strands to minimize the reduction of the section modulus. Klingner and Bieschke (1988) performed tests on precast panels with and without U-bars and found that their presence did not significantly affect the structural performance of the bridge deck.

Ley et al. (2010) investigated the performance of welded-wire reinforcing to replace the tied reinforcing bars in the partial-depth panels to improve the economy, constructability, and construction speed of bridge decks. The experimental tests were conducted on bridge decks constructed with either tied reinforcing or welded-wire mats. The results indicated that the wire mat helped to resist cracking and allowed an owner greater construction tolerances for the reinforcement placement and improved crack control, and, thus, long-term durability.

Spraggs et al. (2012) conducted a field investigation of spalling in several partial-depth precast, prestressed concrete bridge decks using nondestructive testing. Ground-penetrating radar (GPR) was used to assess the condition of the concrete and identify areas of delamination at the interface between the precast, prestressed concrete panels and CIP concrete topping. Core samples and visual inspection were used to interpret and validate the GPR data.

The results indicated that spalling observed in the precast, prestressed concrete panels is the result of corrosion associated with the penetration of water and chlorides through reflective cracking in the CIP concrete topping to the interface between the CIP concrete topping and the precast, prestressed concrete panels, and then through the precast, prestressed concrete panels to the prestressing tendons located near the panel joints.

Visual inspection results from the bottom surface of the bridge deck indicated an uneven distribution of deterioration in the bridge deck panels. GPR results indicate that panels located under the area with increased negative moment reinforcement (that is, girder interior supports) were less deteriorated; therefore, concrete integrity increased with increased CIP concrete topping reinforcement in the bridge longitudinal direction because such reinforcement serves as transverse crack control and thus delays the onset of spalling.

GPR showed that most deterioration in the CIP concrete topping occurred near the area of reflective cracking and not over the middle of the panels.

2.2 PPCB-Supported Precast Overhang

Although the precast partial-depth deck removes the need for formwork for most of the deck construction work, falsework and formwork are still required to construct the deck overhang. To further reduce field labor cost and bridge construction time, different types of precast deck overhangs have been proposed.

For example, Mander et al. (2010) introduced a precast deck overhang system (as shown previously in Figure 1) and conducted a series of laboratory tests to investigate the deformation behavior up to factored design load limits. Two small-scale specimens, each consisting of three supporting girders, two overhang panels, and two precast, prestressed interior deck panels were fabricated and tested. The specimens were loaded near the edge to examine the collapse capacity and the associated failure modes—and particularly the influence of panel-to-panel connections that exist, transverse to the bridge deck axis.

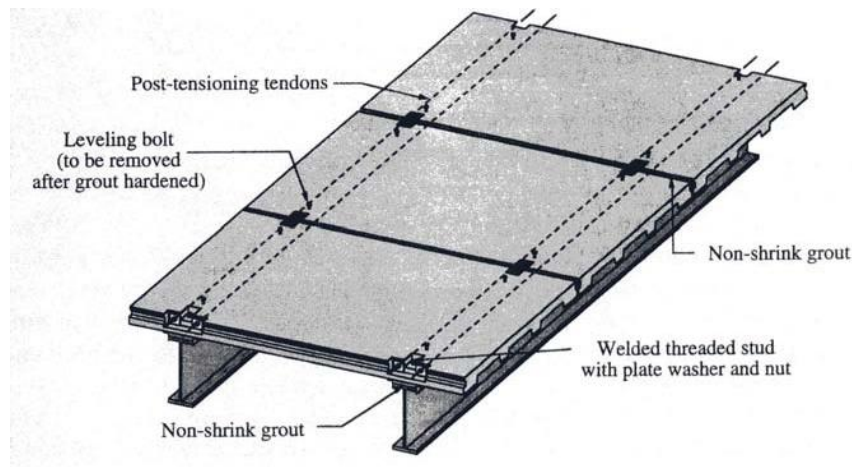
Comparative tests were also conducted with a conventional CIP overhang system. When compared to the conventional CIP overhang behavior, the experimental results showed that the precast, full-depth overhang introduces different behavior modes, largely due to the influence of the partial-depth panel-to-panel connection, which reduced the capacity by 13%.

Trejo et al. (2008) investigated the structural capacity of the precast overhang system and the corresponding deck joints. The results indicated that the precast overhang system could provide a system with comparable structural performance to a bridge deck system using the conventional reinforced overhang details. Furthermore, it appeared that this system can provide improvements in safety, constructability, and economy over the conventional overhang system.

Mander et al. (2011) developed a modified yield-line theory to estimate the overhang capacity of full-depth concrete bridge deck slabs. This theory accounts for the development length of the mild steel reinforcing to reach yield strength. Failure of the full-depth panels is influenced by the presence of the partial-depth transverse panel-to-panel seam. When applying a load on the edge of the seam, the loaded panel fails under flexure, while the seam fails in shear. Through the use of the modified yield-line theory, coupled with a panel-to-panel shear interaction, analytical predictions are accurate within 1% to 6% of experimental results for critical cases.

2.3 PPCB-Supported Full-Depth Precast Deck System with Jacking (Accelbridge Deck System)

Traditional full-depth precast deck systems are one of the most widely used ABC techniques due to their shorter construction times, high-quality components, and relatively simple construction procedures. These systems typically consist of precast concrete panels, approximately 8 in. thick and 10 ft long, placed adjacent to one another on bridge girders (as shown in Figure 2).



Yamane et al. 1998, 2020 Precast/Prestressed Concrete Institute

Figure 2. Precast full-depth deck

The panels are typically connected to the girders using shear pocket connectors, which enable the panels to develop composite action with the girders. Sometimes, the pre-tensioning force is used in either transverse or longitudinal directions. Hieber et al. (2005) indicated that the post-tensioning places the transverse joints between panels into compression, improving durability and promoting monolithic behavior. The transverse closure pours that are required at the panel joints can consist of ultra-high-performance concrete (UHPC), portland cement concrete (PCC), high-strength grout and match cast.

Issa et al. (1998) conducted a finite element (FE) analysis to determine the amount of post-tensioning needed in the longitudinal direction to secure the tightness of the adjacent transverse joints and keep them in compression. The models were created based on a few selected simply supported and continuous bridges. Results of the study indicated that the minimum prestress level is 200 psi (1.4 MPa) for simply supported bridges, while a prestress level of 450 psi (3.1 MPa) is needed over the interior supports for continuous bridges.

The AccelBridge system is a patented, full-depth precast deck system invented by He. In this system, longitudinal compression between deck panels is provided by jacking the deck against the girders, and, therefore, the need for expensive post-tensioning or closure pours is avoided. The major advantages of AccelBridge encompass durability and easy and low-cost construction.

He et al. (2020) documents rehabilitation work on four separate bridges using the AccelBridge full-depth precast deck system. The work on a bridge typically involved two weekend closures. The first weekend was for rehabilitation work on abutments and sleeper slabs on the approach. The second weekend included demolition and replacement of the existing bridge deck. Decks of all four bridges were replaced successfully, each within the weekend closure. The use of match-cast joints and compressing the precast deck with girder jacking aims to make AccelBridge installation straightforward for contractors.

The results indicated that the precast deck was a simple and practical ABC method. Using only conventional materials and straightforward details (match-cast joint) significantly reduces the risk in terms of schedule and quality. Compared with other full-depth precast deck systems, such as post-tensioning and UHPC joints, the AccelBridge system reduces field work, is quicker to install, and is more cost-effective. In addition, there is no reinforcing steel bar or posttensioning across the panel joints, which enhances durability given there is nothing to corrode.

2.4 Precast/Post-Tentioned Pier Cap

The use of precast/post-tensioned pier caps appears to be novel. A preliminary literature search did not find any previous applications of this construction approach. However, the Florida DOT (FDOT) introduced the use of post-tensioning on straddle bents (Corven Engineering, Inc. 2002), as shown in Figure 3.

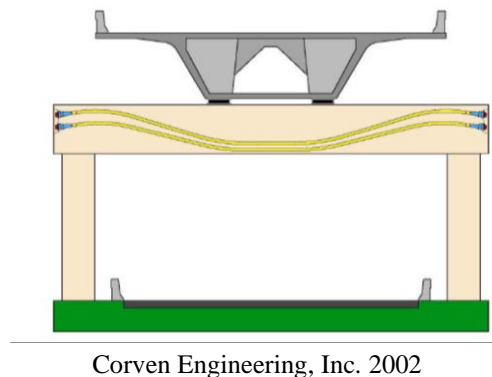


Figure 3. Post-tensioned straddle bents

In a typical straddle bent, tendons drape to a prescribed profile that may be similar to the drape in a beam on simple supports, or it may rise over the columns where a monolithic connection is made to transfer moments into the columns and provide frame action. The columns may be reinforced or post-tensioned, depending upon the magnitude of the forces and moments induced in the frame. Tendons in straddle bents are internal and grouted during construction. However, it is possible to apply external tendons of a similar type to repair or rehabilitate a damaged structure.

CHAPTER 3. FIELD INFORMATION COLLECTION

The field information was collected in two phases: construction documentation and interviewing the contractor. This portion of the project was adjusted based on changes in construction methods used for the US 52 bridge project.

3.1 Field Construction Documentation

At the initial phase of the research work, a webcam was placed at the project site to document the entire bridge construction process. This step was designed to provide live views of the construction activities and store images at regular intervals. Figure 4 and Figure 5 show the camera setup and the site view provided by the camera, respectively.



Figure 4. Camera setup



Figure 5. US 52 overflow bridge over the Mississippi River

Given the contractor changed the structural system from a partial-depth precast deck panel system to a conventional CIP deck, the images captured during construction were not used as a part of this research project report.

3.2 Interviews of Bidding Contractors

Since the final structural system used a conventional CIP deck, the researchers interviewed the contractor about their intentions in making this change. A questionnaire with three questions was developed and sent to two bidders: one non-winning bidder and the contractor that won the project. These questions were designed to reveal the reasons for the requested change of the structural system and the advantages/disadvantages of each option. The findings from these interviews are included in this section, with detailed responses included in the Appendix.

In general, the contractors preferred to use the precast partial-depth deck system over the AccelBridge deck system considering two important factors: experience and cost. First, due to limited experience with the AccelBridge deck system, the full-depth panels appeared very cumbersome and difficult to construct. Second, the cost to fabricate the AccelBridge deck was high and the labor cost of installing the deck system was about the same as that for the partial-depth precast deck. The partial-depth precast deck is a less expensive alternative to the AccelBridge deck according to the contractors interviewed.

Both contractors agreed that the precast partial-depth deck system does not necessarily reduce construction time over the conventional CIP deck, since the construction of the pier diaphragms and abutment diaphragms still take considerable time. Still, the decision to change the project to a conventional CIP deck was due to the associated construction timeline. The CIP deck was

preferred due to greater control over the construction speed via assigning more workers to the effort to get expedited results. One of the bidders suggested that partial-depth precast panels would not expedite the superstructure construction unless the field work of pier and abutment diaphragms can be eliminated or accelerated.

CHAPTER 4. LABORATORY TESTING

Aligning with the results from the literature review, field information collection, and communication with the TAC, the laboratory tests were decided to be performed on the partial-depth deck system. The pier cap was excluded from the laboratory study.

Given the precast double-panel overhang system is new, its ability to carry design loads at different construction stages, as well as factored American Association of State Transportation and Highway Officials (AASHTO) loads under service conditions and extreme event conditions, needed to be laboratory validated. In addition, the constructability of using leveling bolts and flexible sealing strips as haunch forms, and the composite action of the overhang system, were of interest.

Previous experience shows that a solid and uniform bearing between the partial-depth panels and the girder needs to be provided to avoid cracks in the CIP deck and to ensure composite action in the deck system. Laboratory tests were therefore proposed to validate the bearing and composite action, as well as the strength of the precast partial-depth panels. Specifically, the objectives of the laboratory testing were as follows:

1. Verify the sufficiency of the proposed polyethylene bearing between the precast deck panels and the supporting girders
2. Verify the sufficiency of using high-density polyethylene foam to create an adjustable haunch form, its strength to resist the lateral pressure from CIP concrete, and the constructability of using the high-density polyethylene foam in conjunction with leveling bolts
3. Validate composite action between the CIP concrete deck and precast panels by investigating the behavior of the deck panels under vertical and horizontal loads

To achieve the laboratory testing objectives, two specimens were constructed. The specimens had identical geometry except that one had a barrier and the other did not. The one with the barrier was loaded with horizontal loading and the one without the barrier was loaded subject to vertical loading.

The design and construction details for the specimens are presented in Section 4.1 and 4.2, respectively. The experimental setup and loading information are included in Section 4.3. Section 4.4 and 4.5 present the instrumentation plan and test results, respectively.

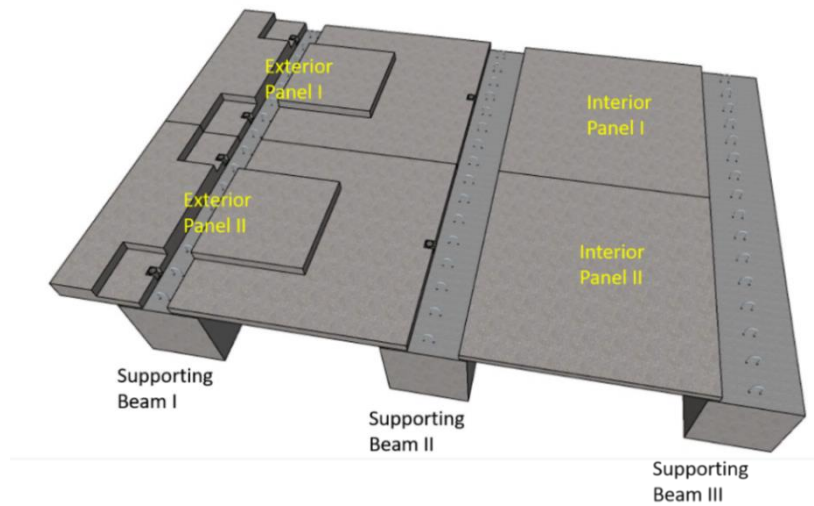
4.1 Specimen Design

The laboratory specimens were designed based on construction drawings developed by Parsons. A portion of the bridge superstructure of about 16 ft in the traffic direction and 21 ft in the transverse direction was built in the laboratory. Both specimens consisted of three supporting beams, two precast prestressed interior panels, two exterior double-panel overhangs, and the top layer of CIP concrete, as detailed in Table 1.

Table 1. Specimen components

Specimen	Components	Quantity
Specimen I (with barrier)	Pre-stressed interior panel	2
	Exterior panels with barrier	2
	Supporting beam	3
	CIP deck and barrier	1
	CIP barrier	1
Specimen II (without barrier)	Pre-stressed interior panel	2
	Exterior panels	2
	Supporting beam	3
	CIP deck	1

Figure 6 shows the three-dimensional (3D) view of the specimen without a barrier or the CIP concrete deck.

**Figure 6. 3D view of test specimen without CIP deck**

A barrier was constructed on one specimen with horizontal loading to transfer the load to the deck panel. The trapezoid barrier on this specimen was simplified as a rectangular shape as shown in Figure 7 to create a flat vertical surface for the application of the horizontal loading.

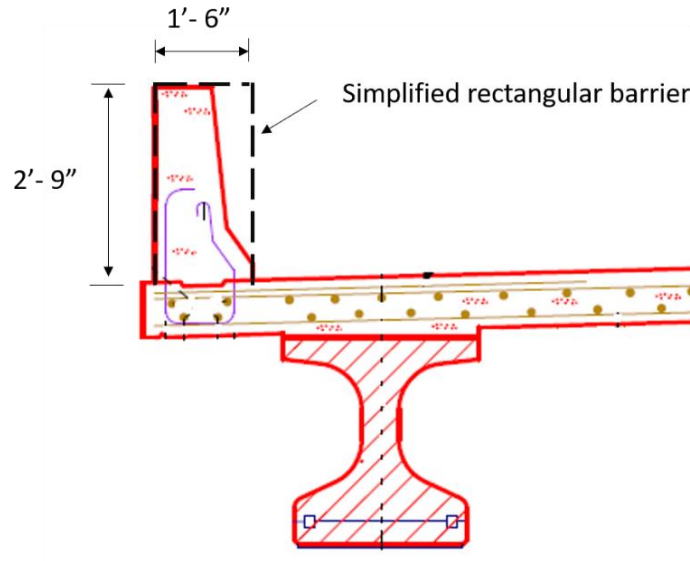


Figure 7. Simplification of barrier

Given the beams were only used as support for the deck panels, and the I shape would not contribute to the structural behavior of the deck system, the beams were simplified to a rectangular shape and designed without longitudinal prestressing force. Figure 8 shows the simplification of the supporting beam.

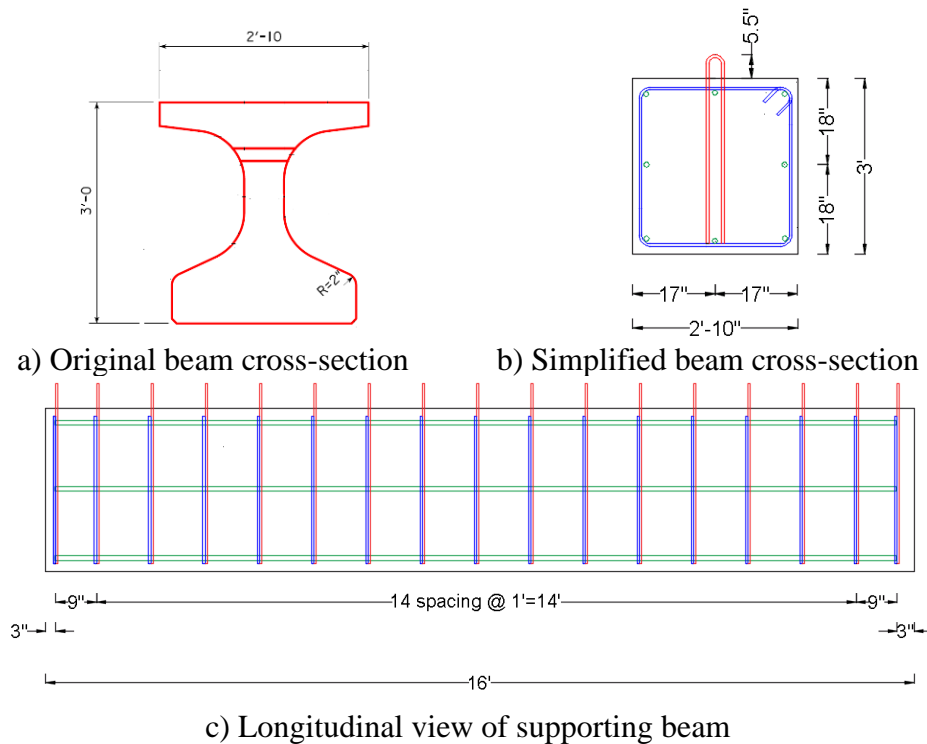


Figure 8. Design of supporting beam

The interior and exterior panels were designed according to the design drawings shown in Figure 9 and Figure 10.

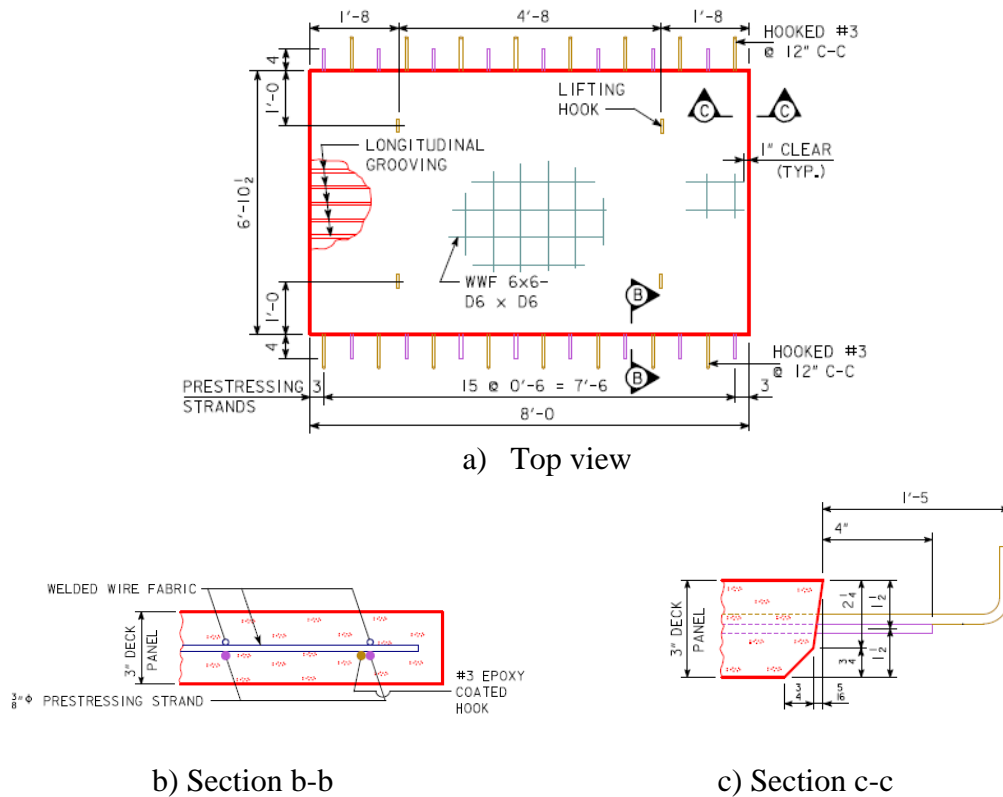
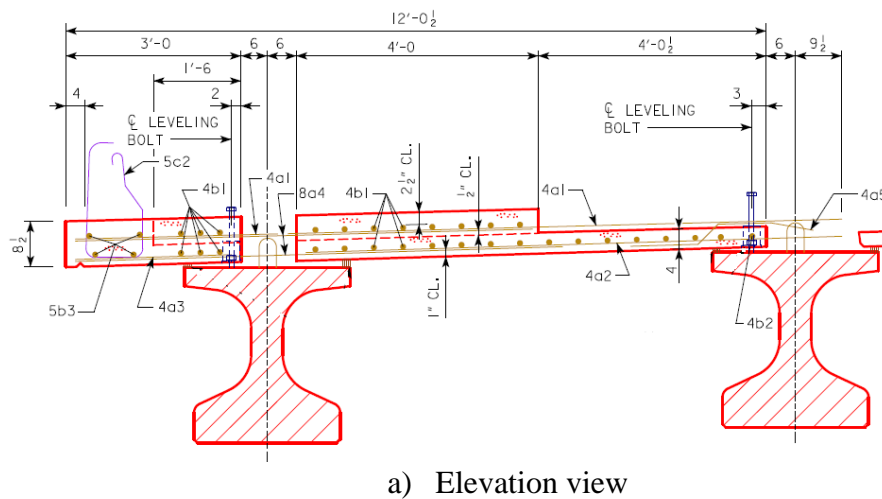
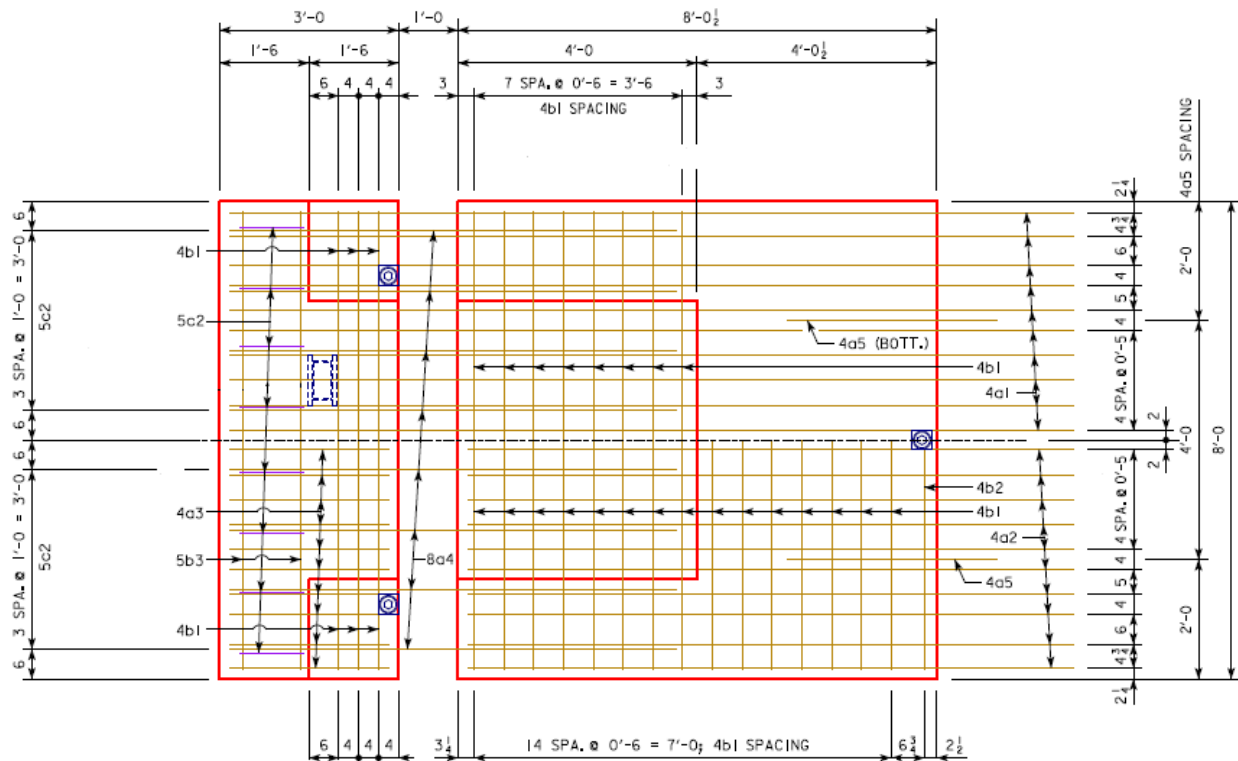


Figure 9. Design of precast prestressed interior deck panel





b) Top view

Figure 10. Design of precast double-panel overhang (exterior panel)

According to the design drawings, these precast deck panels are designed to support the dead load of the panel, reinforcement steel, CIP concrete, and 50 lb/ft² of construction load. The panel and CIP slab, acting as a composite section, is designed for HL-93 loading plus 20 lb/ft² of roadway for a future wearing surface. The top surface of the precast deck panels shall be given a suitable texture with a wire broom or comb having a single row of tines, with the desired grooving in the longitudinal direction (parallel to the centerline of bridge roadway).

The prestressing strands in the interior deck panel are the 1/2 in. diameter grade 270 low-relaxation strands with an initial tension of 16,100 lb/strand (70% of the guaranteed ultimate tensile strength). No. 3 reinforcing bars, with 1 ft spacing in both directions, was considered an allowable substitution for the WWF 6×6-d6 × d6. The panel concrete adhered to a minimum 28-day strength of 6,000 psi and a minimum release strength of 4,500 psi. The deck panels were at least 28 days old before the CIP slab was placed.

Figure 11 shows the plan view of the tested specimen without the barrier, and Figure 12 shows the reinforcing details of the tested specimen with the barrier.

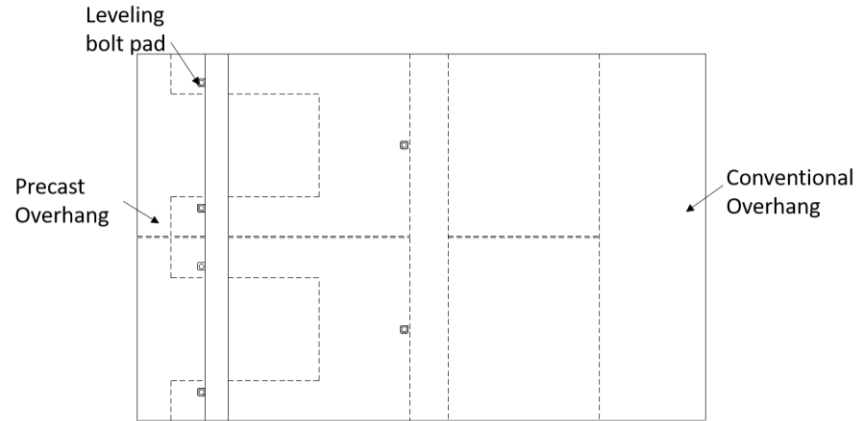


Figure 11. Plan view of the tested specimens (without barrier)

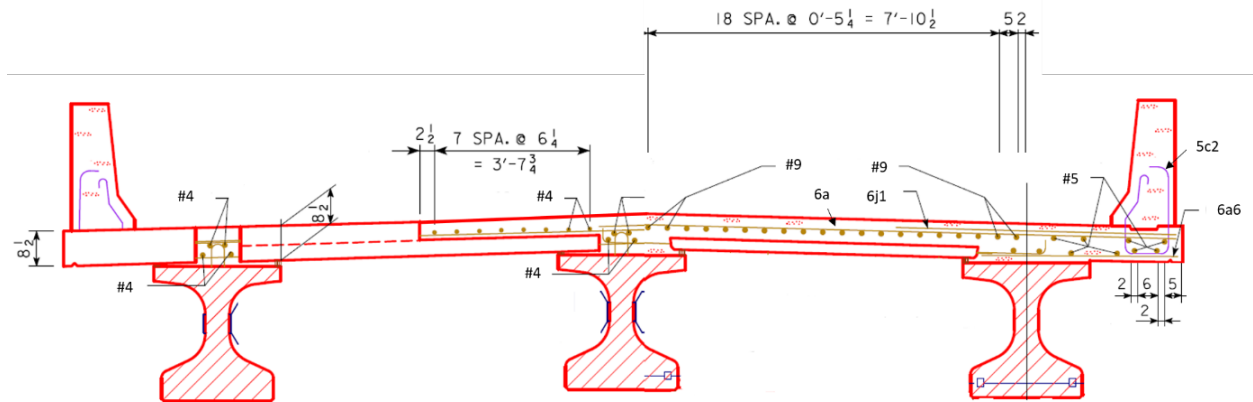


Figure 12. Reinforcing details of the tested specimens (with barrier)

The tested specimen was designed to include both a precast double-panel overhang and the CIP deck overhang. This design allows for a comparison of structural performance of the overhangs.

4.2 Specimen Construction

Figure 13 shows the construction of the precast prestressed interior deck panel.



Figure 13. Construction of precast prestressed interior deck panel

Before construction of the interior panel, two dead walls were cast and tied down on the floor in the laboratory to provide the reaction force for the application of the pre-tensioning load. According to the bridge design drawings, each strand was pretensioned to 16,100 lb/strand, which is 70% of its ultimate tensile strength.

Figure 14 shows the construction of the precast double-panel overhang (exterior panel).



Figure 14. Construction of precast double-panel overhang (exterior panel)

Complex formwork was designed and built for the specific geometry of the double-panel deck overhang. The steel pieces for the level bolt were embedded into the precast concrete. According to the specifications, the top surface of the bottom precast deck layer on both interior and exterior panels was roughened with a wire broom to strengthen the bond between the CIP and precast deck concrete.

Figure 15 shows the construction of the rectangular supporting beam.



Figure 15. Construction of supporting beam

Three vertical polyvinyl chloride (PVC) ducts were embedded into each beam with a spacing of 4 ft to allow for tightening of the beams to the floor during testing.

Figure 16 shows the bearing details of the specimen with the barrier.

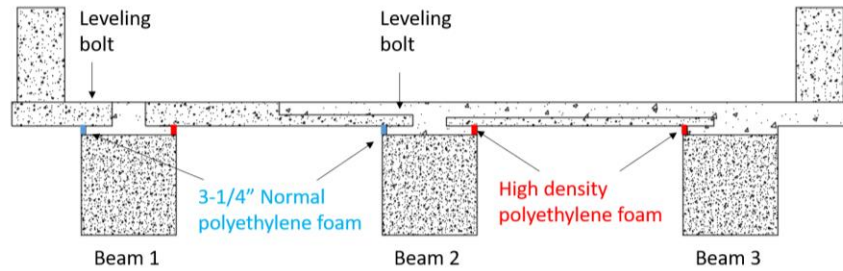


Figure 16. Bearing details

Note that the specimen was designed with the same bearing details. For the left edge of Beam 1 and 2, the normal polyethylene foam was used and incorporated the leveling bolt. Normal polyethylene foam was used as the seal strips to resist the lateral pressure from the fresh concrete. The leveling bolt was used to adjust the level of the precast deck panel and to control the transverse slab slope in the field.

Although the deck transverse slope was not included in this test, the leveling bolt was used to create a haunch height of 3-1/4 in. This is the maximum haunch height given in the bridge drawing, which is in a range of 1-1/2 to 3-1/4 in. Using the maximum value generates the greatest lateral concrete pressure during placement of the CIP concrete and enabled the research team to evaluate the sufficiency of the normal polyethylene foam to resist the pressure from the fresh concrete.

The high-density polyethylene foam was installed over the right edge of Beam 1 and 2 and the left edge of Beam 3. The high-density polyethylene foam used here served two functions: 1) provide enough strength to support the precast deck panel, CIP deck, and construction weight and 2) provide enough stiffness to keep the desired haunch height without experiencing large deformation when subjected to the deck and construction loads. Figure 17-a shows the polyethylene foam after it was installed on the specimen.



a) High density polyethylene foam

b) Coupler used to anchor the precast deck

Figure 17. Precast component assembly details

After placement of the precast exterior and the interior decks, couplers were used to anchor the extension reinforcement of the precast deck to the shear studs of the supporting beam, as shown in Figure 17-b. While one coupler was used for each precast deck panel to prevent potential deck slippage during the placement of the CIP concrete, it was found that the precast deck panels are very stable and no slippage occurred.

Prior to concrete placement, the exposed surfaces were blown via an oil-free air blast to remove all dust and debris. Figure 18 shows the construction of the CIP concrete and barrier.

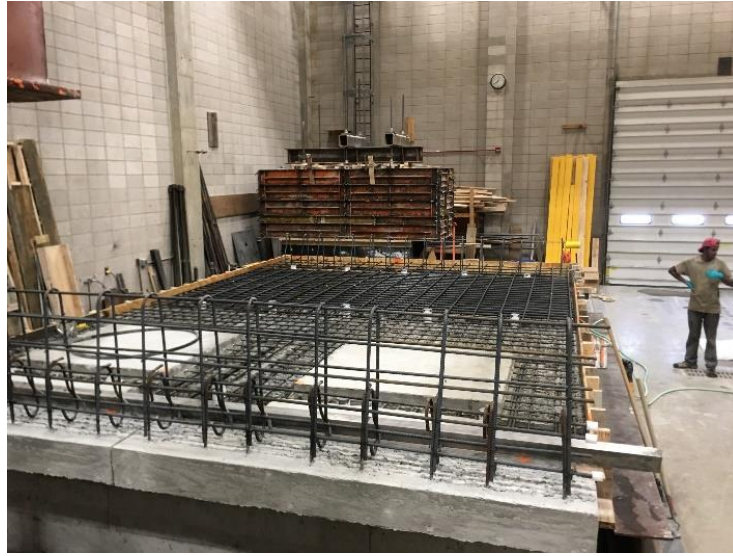


Figure 18. Construction of CIP concrete and barrier

Both high-density polyethylene foam and normal polyethylene foam were glued to the top of the supporting beams. However, during placement of the CIP concrete on Specimen I (with the barrier), the high lateral pressure of the fresh concrete broke the high-density polyethylene foam over Beam 2, as shown in Figure 19.



Figure 19. High-density polyethylene foam punch-out

A timber board was used to temporarily provide additional bracing. The glue-work on the second specimen was conducted with additional attention. In addition to that, before the gluing work, the top surface near the edge was ground to create a flat surface for the application of the glue material.

In general, the leveling bolt with normal polyethylene foam provided the needed support for the deck panel to resist the lateral concrete load. However, special attention is needed when gluing the polyethylene foam to ensure a strong bond is created between the concrete and the polyethylene foam.

Due to the large amount of concrete placement work, each structural component was cast separately. The 4 × 8 in. cylinders were cast with each concrete placement to obtain concrete compressive strength on the test dates. Table 2 shows the compressive strength for each structural element.

Table 2. Material properties

Specimen	Component	Compressive strength (ksi)
Specimen I – with barrier	Exterior panel I	9.13
	Exterior panel II	7.61
	Interior panel I	10.64
	Interior panel II	10.64
	CIP deck	6.02
	Barrier I (Precast deck overhang side)	6.87
	Barrier I (CIP deck overhang side)	6.87
Specimen II – without barrier	Exterior panel I	7.03
	Exterior panel II	7.70
	Interior panel I	10.14
	Interior panel II	10.14
	CIP deck	5.1

Each test was performed after the all of the structural elements were 28 days old. According to the bridge design drawings, the compressive strength of the interior prestressed panels should be at least 6 ksi at 28 days. Because of this, high-strength concrete was used and resulted in a compressive strength of about 10 ksi on the test dates. The other precast and CIP elements had compressive strengths of 5.1 to 7.7 ksi, aside from one exterior panel with a high strength of 9.13 ksi. The compressive strengths of all elements satisfied the specified requirements.

4.3 Experiment Setup and Loading Configuration

4.3.1 Specimen Subjected to Horizontal Loading

The two specimens were subjected to different types of loading. Specimen I – with barrier was horizontally loaded. The objective of this test was to investigate the structural behavior of the exterior double-panel deck when a vehicle collides with the barrier. The load was applied on the vertical surface of the barrier, as shown in Figure 20.

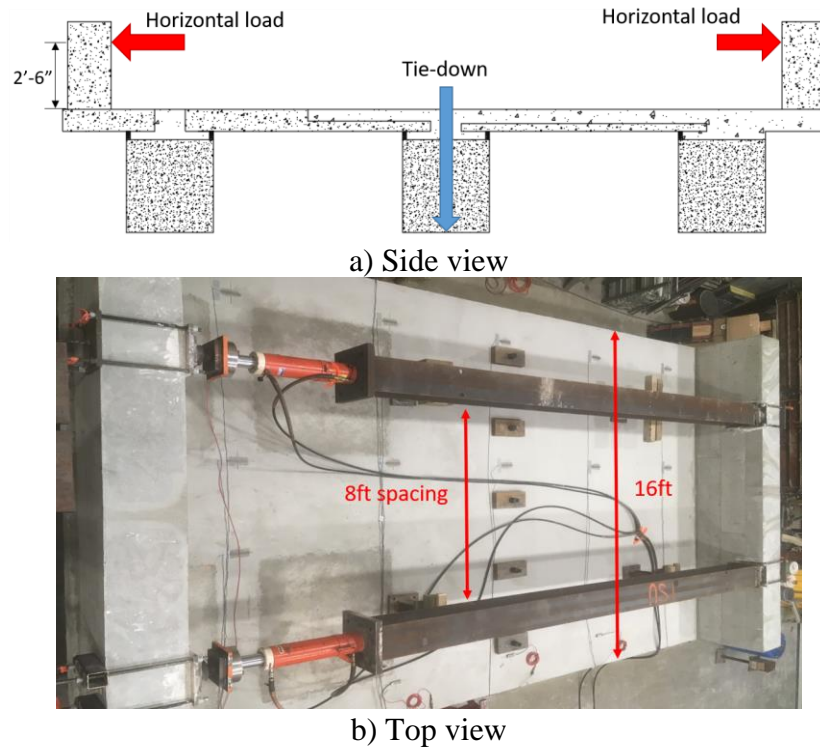


Figure 20. Application of horizontal load on Specimen I – with barrier

The specimen was loaded until failure to measure the ultimate capacity. During the test, the middle supporting beam was tied to the floor to prevent rigid motion of the specimen. The load was applied at two points on each side of the barrier, as shown in Figure 20. The total length of the specimen in the traffic direction was 16 ft and the spacing of the two load application locations was about 8 ft.

At each load application location in the longitudinal direction, one actuator was used to generate the loading. One side of the actuator was set to push against one of the barriers and the other side of the actuator was set against a steel pile. The other side of the steel pile was set to pull the other barrier (see Figure 20-b). This load application method enabled the loads acting on each barrier to be equal.

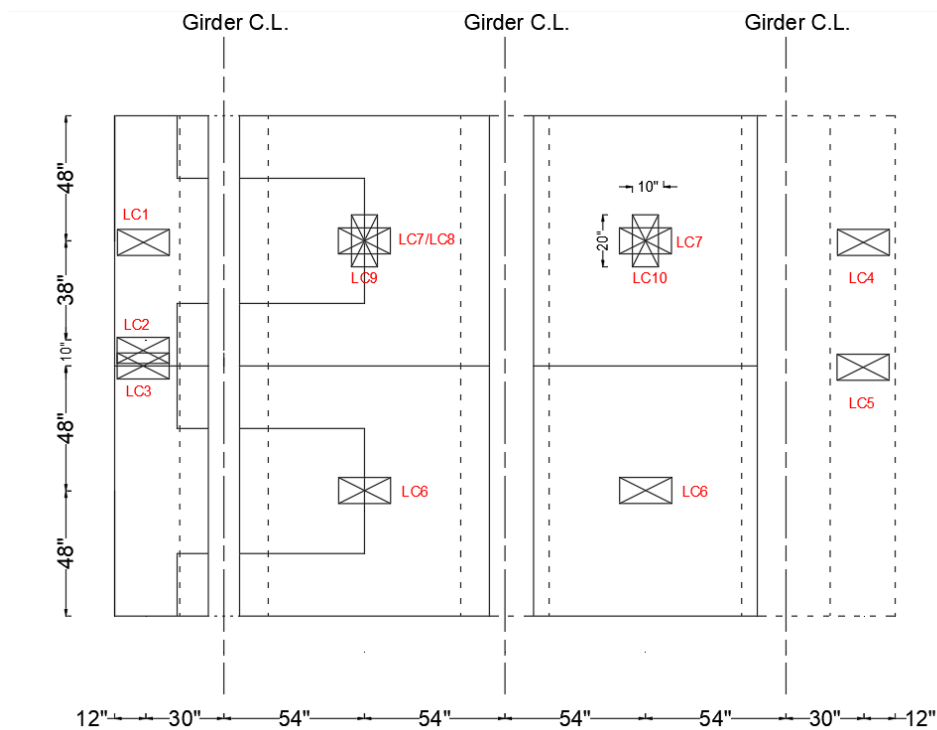
4.3.2 Specimen Subjected to Vertical Loading

The specimen without a barrier was vertically loaded. The objective of this test was to evaluate the deck composite performance when subjected to the vertical construction and vehicle loads. To achieve the objective, various load cases (LCs) were conducted.

Table 3 shows the loading magnitude for each LC, with the loading location for all LCs shown in Figure 21.

Table 3. Load cases on the specimen subject to vertical loading

LC No.	Magnitude	Load until Failure
1	16	---
2	16	---
3	16	---
4	16	---
5	16	---
6	32	---
7	180	---
8	200	---
9	250	Yes
10	240	Yes

**Figure 21. Load cases on the specimen subject to vertical loading**

In total, 10 load cases were performed. LC1 to LC5 were applied on the deck overhang to simulate the construction load. LC6 and LC7 were applied to the deck between the girders to simulate truck loading during the bridge service life. From LC1 to LC6, each load point was loaded to 16 kips, which generates a transverse negative moment equivalent to that induced by a

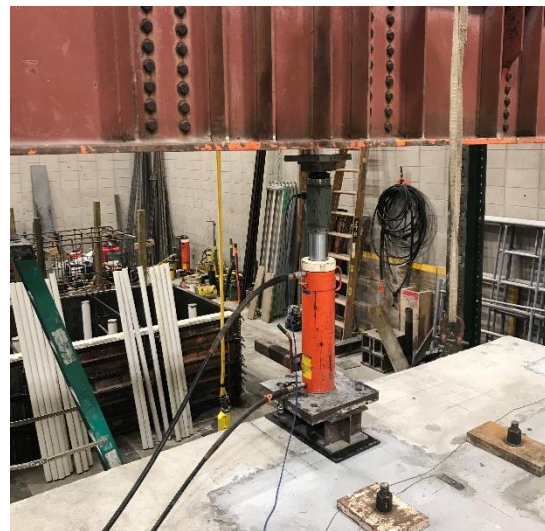
HL-93 design truck based on the AASHTO Load and Resistance Factor Design (LRFD) Bridge Design Specifications (2012).

LC7 was designed to break the specimen; however, the actuator reached its capacity (90 kips) during the test. It was then decided to load each bay separately until failure. LC8 and LC9 were performed over the precast double-panel (see previous Figure 21).

LC8 loaded the specimen to 200 kips, at which the actuator was at capacity, and the specimen was still unbroken. LC9 was then performed with a larger actuator. Note that for LC1 through LC8, the load pad was placed with the short edge parallel to the traffic direction. In LC9 and LC10, the loading pad was turned 90° to generate a greater bending moment to break the specimen. The ultimate loading in LC9 was about 250 kips. LC10 was conducted over the prestress interior panel with an ultimate load of about 240 kips. The results from each LC are presented in Section 4.5. During this test, the specimen was tied to the ground utilizing the post-tensioning rods through the plastic ducts pre-installed into the supporting beams. The whole specimen was restrained at nine locations (three on each beam), as shown in Figure 22.



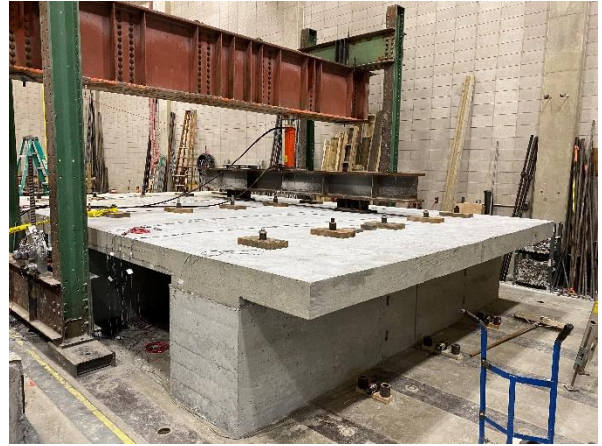
a) LC1



b) LC2



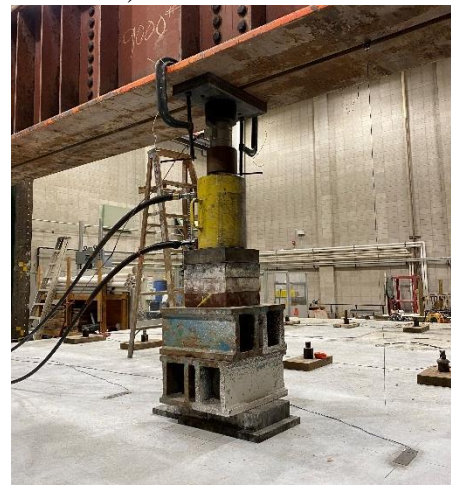
c) LC6



d) LC7



e) LC9

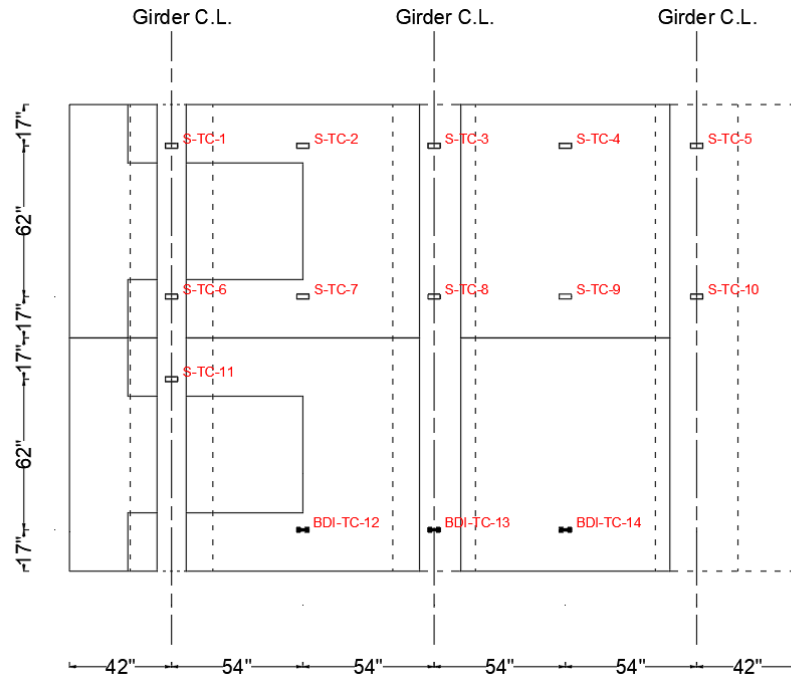


f) LC10

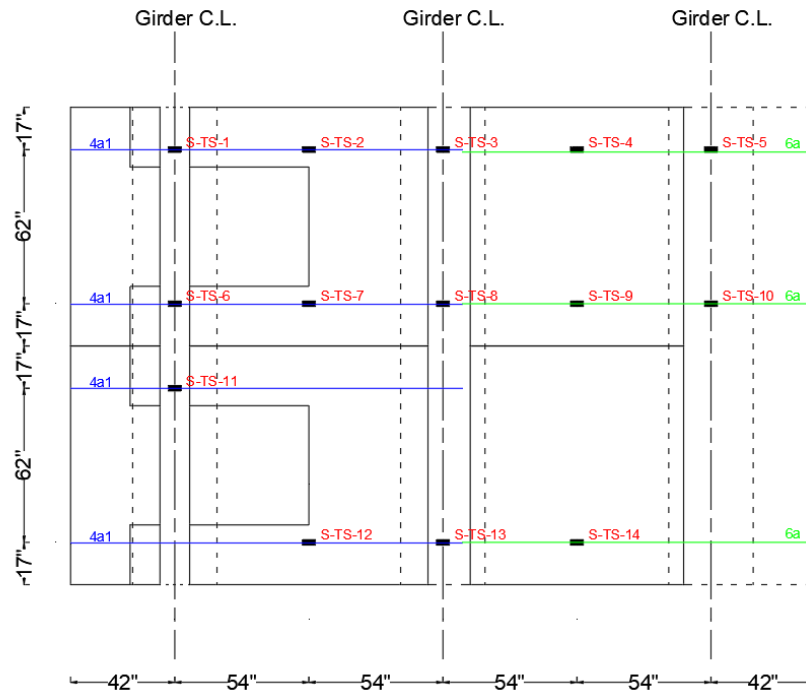
Figure 22. Loading frame setup during vertical load test

4.4 Instrumentation Plan

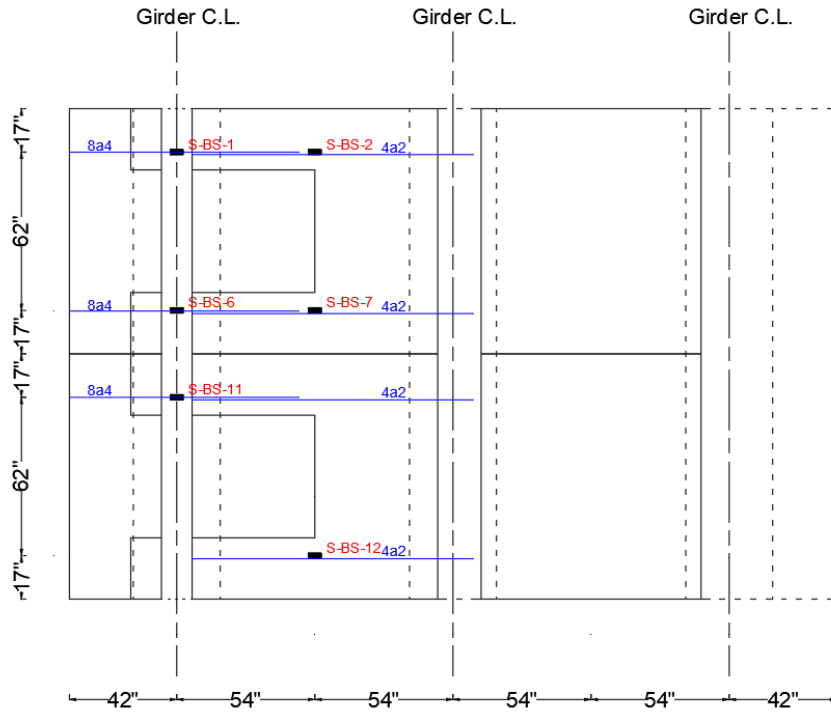
The instrumentation plan was designed to capture the composite action between the precast and CIP deck system when subjected to vertical and horizontal loading. Hence, multiple strain gauges were placed at the same location, but at different levels in or on the exterior surface of the deck to measure the strain variance within the height of the deck. In general, the gauges were installed at four layers: top surface of the concrete deck, top layer of reinforcing steel bar in the deck, bottom layer of reinforcing steel bar in the deck, and the bottom surface of the concrete deck. The instrumentation plan for each layer is shown in Figure 23.



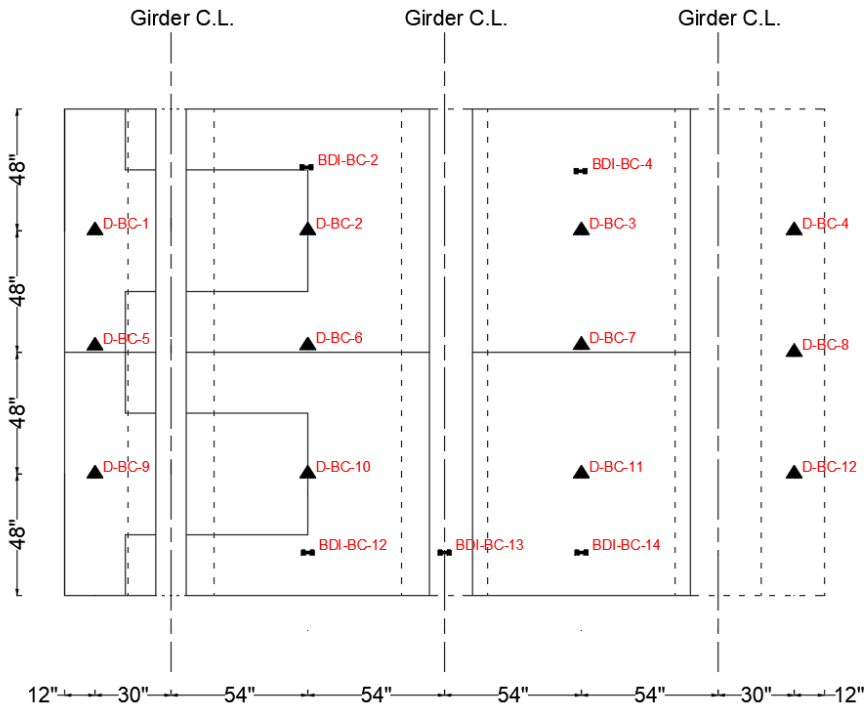
a) Top surface of concrete



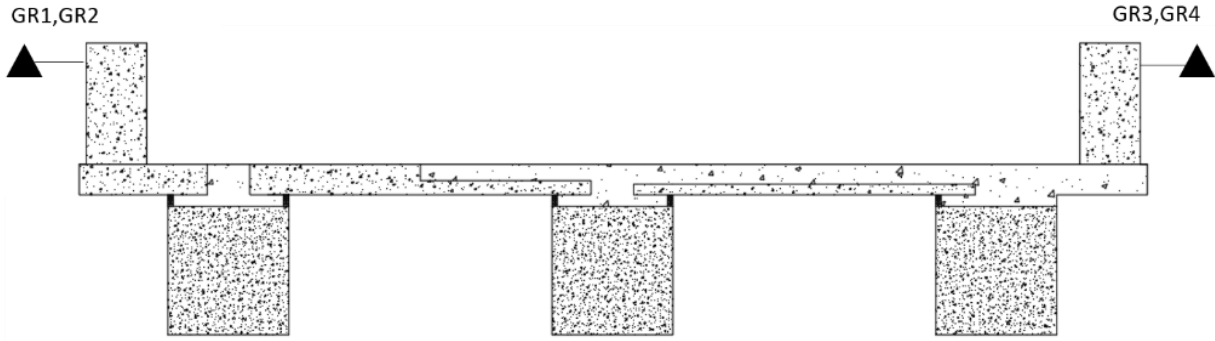
b) Top layer steel rebar



c) Bottom layer steel rebar



d) Bottom surface of concrete



e) Lateral displacement measurement (Specimen I only)

Figure 23. Instrumentation plan

The notations shown in Figure 23 indicate the following: S for a foil strain gauge, BDI for a BDI strain gauge, D for a displacement transducer, TC for the top surface of the concrete deck, TS for the top layer of the steel rebar in the deck, BS for the bottom layer of the steel rebar in the deck, and BC for the bottom surface of the concrete deck.

Both specimens followed the same instrumentation plan. Table 4 summarizes the gauges that were installed on each specimen.

Table 4. Instrumentation plan for each specimen

Location	5 mm Rebar Foil Strain Gauge	90 mm Concrete Foil Strain Gauge	BDI	Displacement Transducers
Deck Top Surface		11	3	
Top Layer Rebar in Deck	14			
Bottom Layer Rebar in Deck	6			
Deck Bottom Surface			5	12+4 (Specimen I only)
Sum	20	11	8	12

In total, 20 5-mm foil strain gauges were installed on the top and bottom layer of reinforcing steel bar, 11 90-mm foil strain gauge were attached on the top surface of the concrete deck, another 8 BDI strain gauges were used to measure the external strain, and 12 displacement transducer were placed at the bottom of the concrete to measure the vertical displacement at various locations. For Specimen I with a barrier, four additional displacement transducers were installed to measure transverse movement at each loading point (see previous Figure 23-e).

4.5 Test Results: Specimen Subjected to Horizontal Loading

Horizontal loading was applied to the barrier to simulate behavior when the bridge experiences a vehicle crash. In this study, only the horizontal load was gradually increased until it reached the ultimate stage, and no dynamic behavior was included. This horizontal load on the barrier generated a negative moment in the deck over the exterior girder. The composite action of the deck over the exterior girder and in the overhang is of interest.

4.5.1 Deck Cracking

The horizontal load was gradually applied to each barrier through two actuators, as shown in the previous Figure 20-b. The loading was paused when each actuator reached 15 kips and 50 kips to document crack propagation of the specimen. At each pause, the cracks on the top and bottom surfaces of the deck were marked and photographed.

Figure 24 shows the crack propagation near the barrier and on top of the deck.

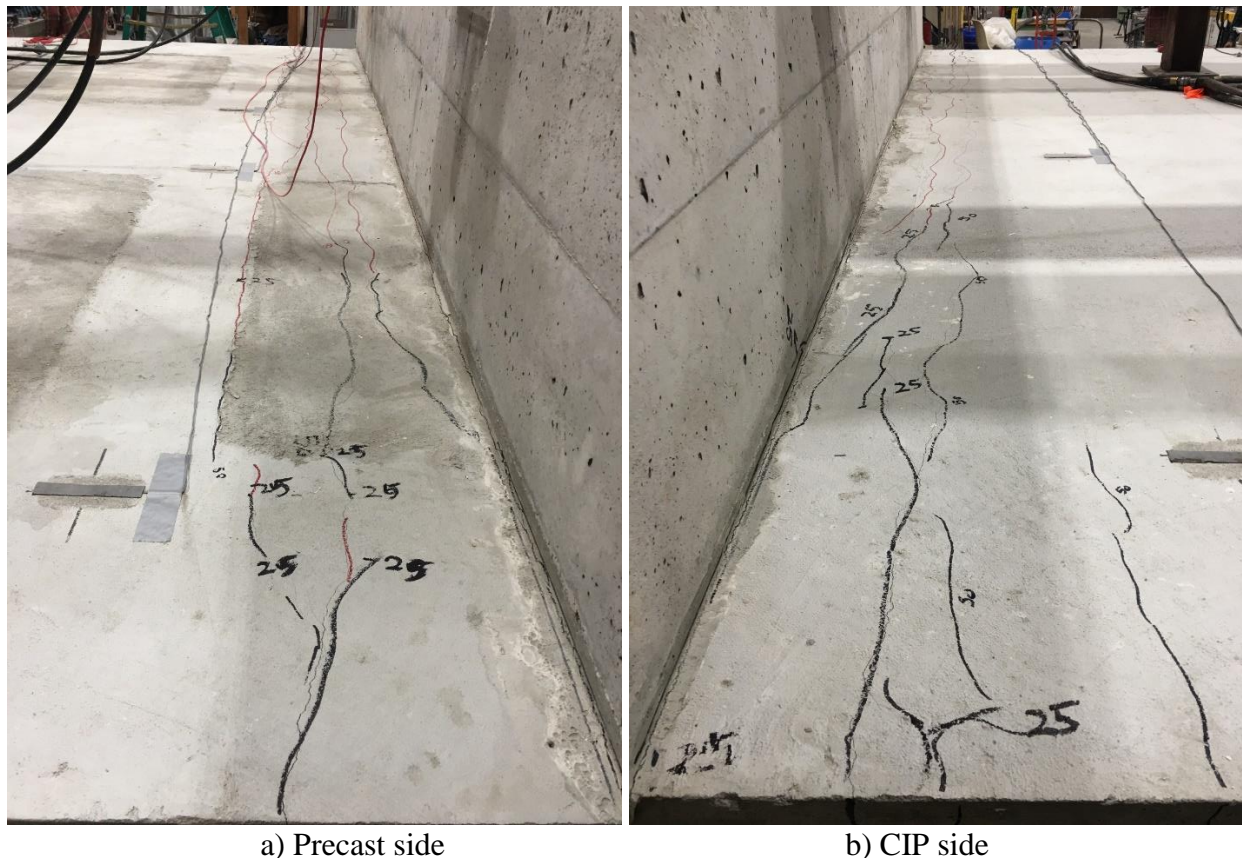


Figure 24. Cracks on the top surface (Specimen I)

It was found that when the load reached 25 kips, both the CIP and precast sides already experienced some level of cracking. On the precast side, the cracking included debonding between the precast deck panel and the CIP concrete.

Figure 25 shows the cracking at the side of the deck near/underneath the barrier.



Figure 25. Cracks under the barrier (Specimen I)

It was found that, on the precast side, the debonding occurred at the vertical interface between the precast and CIP deck when the load reached 25 kips. When the load was increased to 50 kips, the cracking continued in the horizontal direction, as shown in Figure 25-a. On the CIP side, three vertical cracks were found in the deck over the exterior beam when the load reached 25 kips, and one vertical crack was found underneath the barrier when the load reached 50 kips.

Figure 26 shows the failure on the precast side underneath the barrier after the ultimate load of 55 kips per actuator.

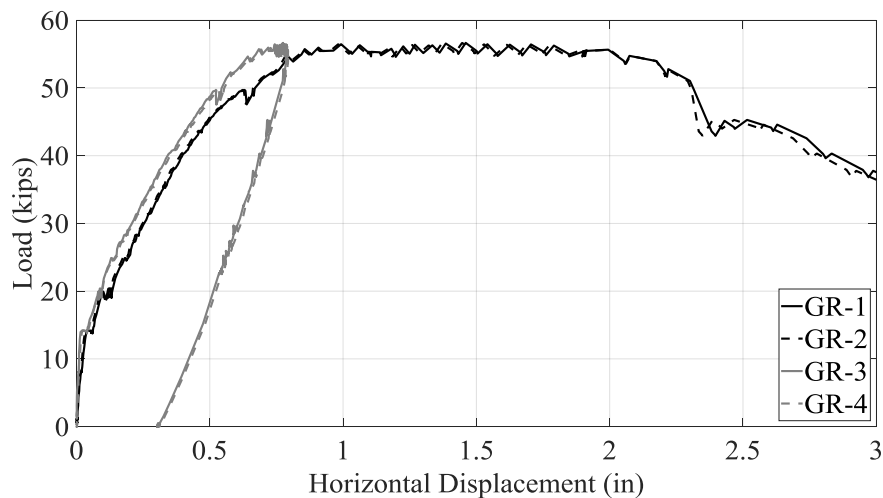


Figure 26. Precast deck concrete crash (Specimen I)

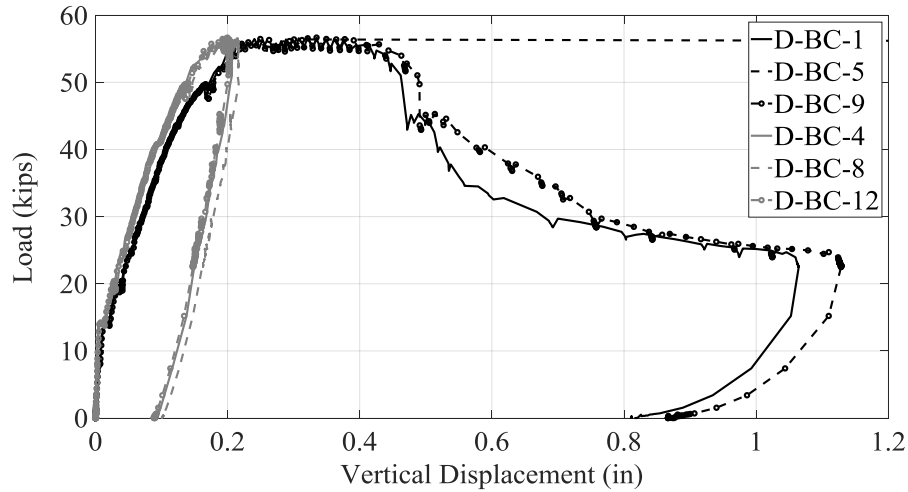
It was found that when the load surpassed 55 kips, most of the new cracks occurred in the deck underneath the barrier.

4.5.2 Test Data

In this section, the measured strain and displacement data are plotted with the loading change in each actuator. For example, Figure 27-a shows the load versus horizontal displacement at each loading point (see the previous Figure 23 for the gauge locations).



a) Horizontal displacement collected at the load application points



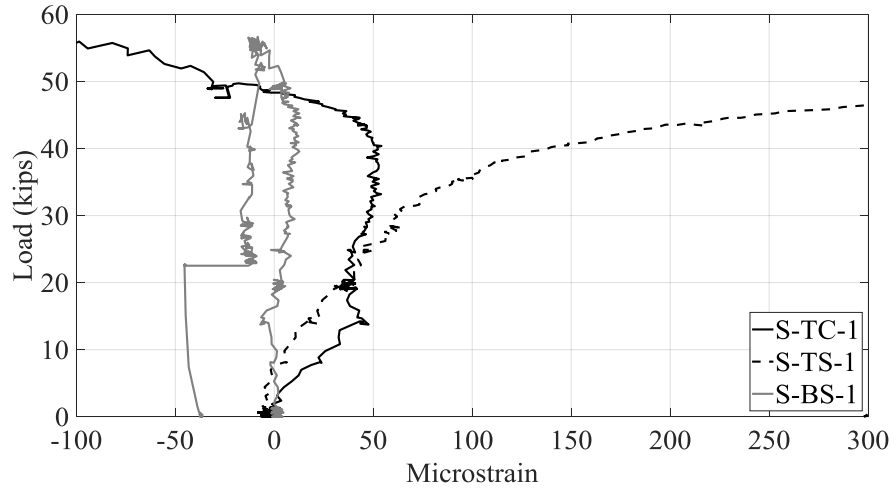
b) Vertical displacement at the bottom surface

Figure 27. Displacement data from Specimen I test

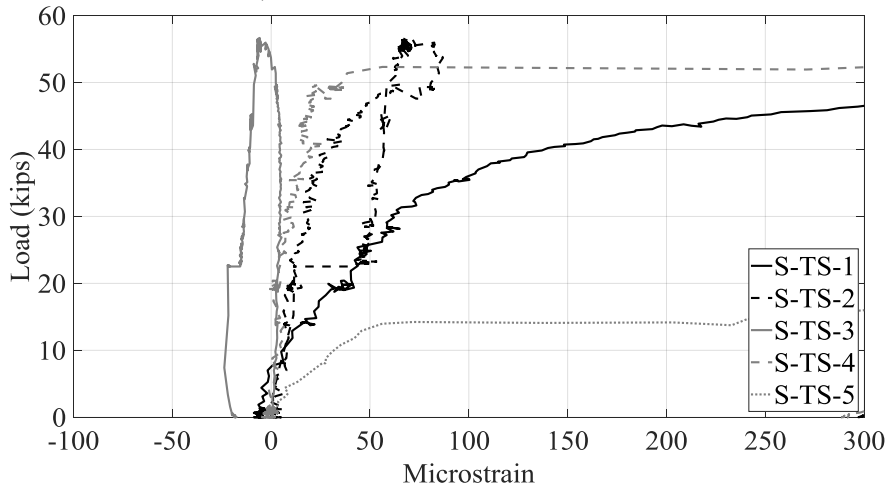
The results indicated that the load versus displacement curves at each loading point on the same barrier are very similar, and no significant twisting behavior was found. Comparing the data from the precast side (black lines) and CIP side (gray lines), it was found that the CIP side has greater stiffness than the precast side throughout the duration of loading. This result was validated by the vertical displacement data shown in Figure 27-b, such that the vertical displacement measured from the precast side (black lines) was higher than that of the CIP side (gray lines).

The setup of the loading frame ensured that both barriers were subjected to the same magnitude of load, and, thus, the ultimate loading occurs when any portion of the barrier fails first. Based on the results from the inspection during the test and the analysis of the test data, it could be concluded that the ultimate load of 55 kips failed on the precast deck side. The ultimate capacity of the CIP deck may be higher than 55 kips, but could not be obtained from this test.

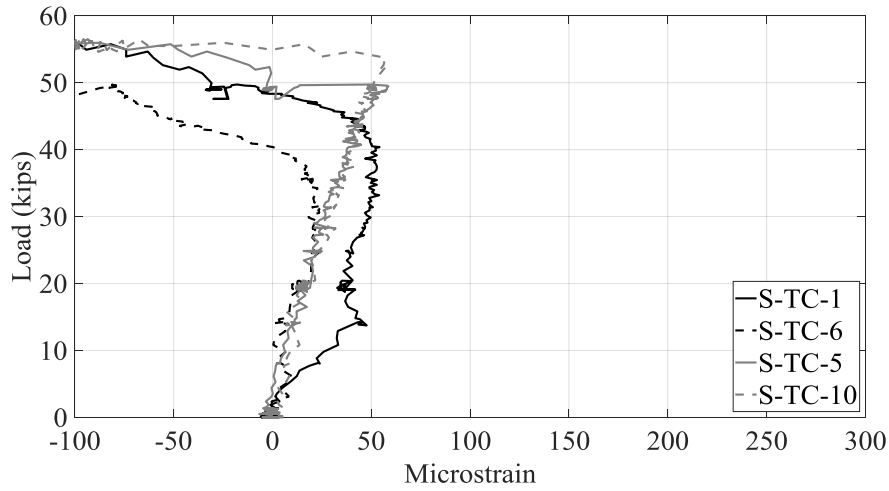
Figure 28 plots the strain data versus loading at various locations (see the previous Figure 23 for the gauge locations).



a) Strain collected at location-1



b) Selected strain data on the top layer reinforcement



c) Selected strain data on the top surface of concrete deck

Figure 28. Strain data from Specimen I test

Figure 28-a shows the strain at different levels from location-1. The results satisfy the structural principle that the highest tensile strain occurs at the top surface and compression occurs at the lower level when subjected to a negative moment.

Figure 28-b plots the strain in the top layer reinforcing steel bar at multiple locations. It was found that S-TS-5 shows a sudden strain increase when the load reached about 15 kips. This indicates that the first crack occurs in the concrete at this load level. This finding could also be validated by the strain data shown in Figure 28-c, which plots the strain on the top surface of the deck at various locations. The strain from S-TC-1, initially, linearly increased; however, a yielding point occurred when the load reached about 5 kips. These results indicate that the first cracks occurred when the load reached 15 kips, and after that the reinforcing steel bar starts to carry more load.

4.6 Test Results: Specimen Subjected to Vertical Loading

Specimen II was subjected to vertical loading at various locations via 10 LCs. As with the first specimen, the first five LCs (LC1 through LC5) were loaded on the precast/CIP deck overhang to simulate construction loading. LC6 and LC7 were designed to simulate truck loading. LC8 and LC9 were loaded on the precast, non-prestressed deck panel with the intention to break the deck and obtain the ultimate capacity. LC10 was loaded over the precast prestressed interior panel to investigate the deck ultimate capacity. The load case locations were previously presented in Figure 21.

4.6.1 Prior to the Load Application

Prior to load application, a few hairline shrinkage cracks were found on top of the CIP concrete deck, as shown in Figure 29.

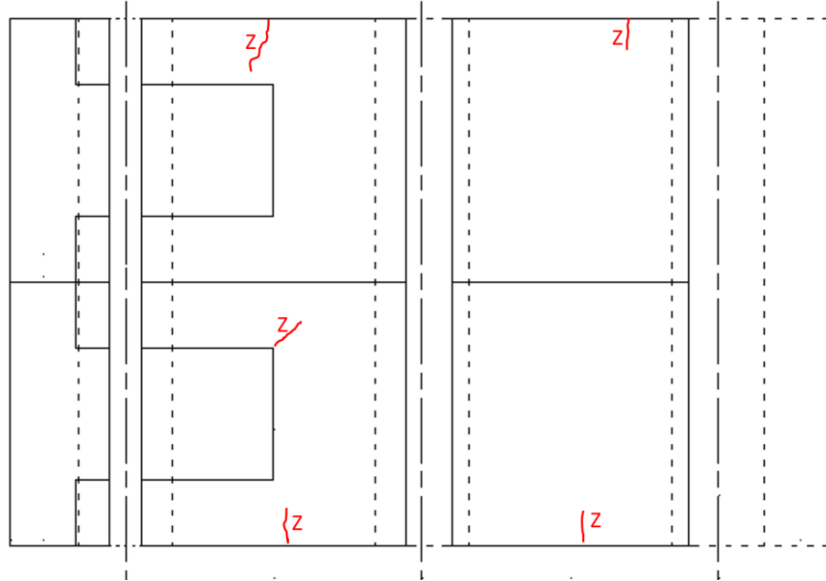


Figure 29. Crack map on the top surface of deck before load application

4.6.2 LC1 to LC5

LC1 through LC5 were designed to simulate loading during construction activities. The deck overhang was loaded up to 16 kips at various locations. These low load levels were chosen with the intention not to generate significant cracking of the specimens so as not to influence the structural behavior during subsequent LCs.

4.6.2.1 Crack Map

During LC1 through LC5, only one crack was generated, due to the load in LC1, as shown in Figure 30.

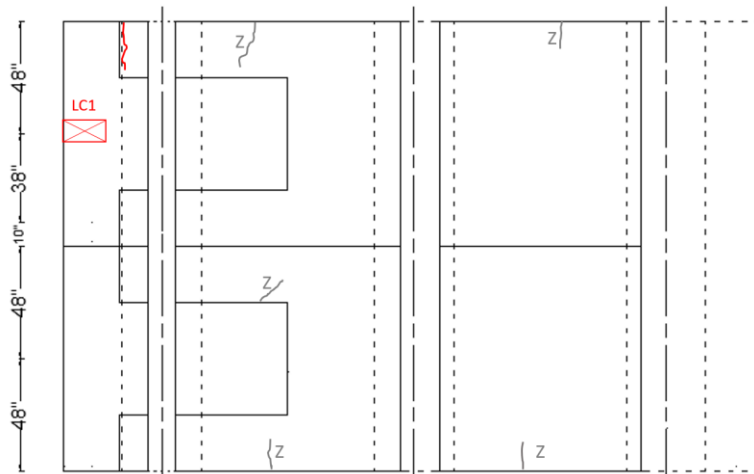


Figure 30. Crack map on the top surface of deck after LC1

This crack occurred at the interface between the precast and CIP concrete, which was likely induced by the negative moment generated by the vertical loading on the deck overhang and low bond strength at the interface.

4.6.2.2 Test Data

Although one crack was found during the application of LC1, all the strain and displacement gauges that were initially designed to measure the composite action at the critical locations did not show any significant structural response in any of the five LCs. The displacement and strain data collected from LC1 through LC5 are not presented in this report given no significant readings were measured.

4.6.3 LC6 and LC7

LC6 and LC7 were designed to simulate truck loading via two load points. LC6 was loaded to 32 kips (16 kips per loading point), and LC7 was loaded to 180 kips (90 kips per loading point). LC6, with a maximum loading of 16 kips per loading point, was designed to investigate the deck composite behavior under design loads. The loading of 16 kips per loading point generated a negative moment over the girder, which is equivalent to the design negative moment with an 8 ft girder spacing based on AASHTO bridge design specifications (2012).

The initial design of LC7 was to break the specimen; however, it could not be broken as the load exceeded the capacity of the actuator. It was then decided to individually load each point in LC7 in the subsequent LCs to investigate the deck behavior at the ultimate stage.

The location of the loading points and the orientation of the loading pad was shown in the previous Figure 21. The loading pad was placed with the long edge perpendicular to the traffic direction to simulate the truck wheel footprint.

4.6.3.1 Crack Map

Figure 31 and Figure 32 show the cracking on the bottom surface of the deck after application of LC6 and LC7, respectively.

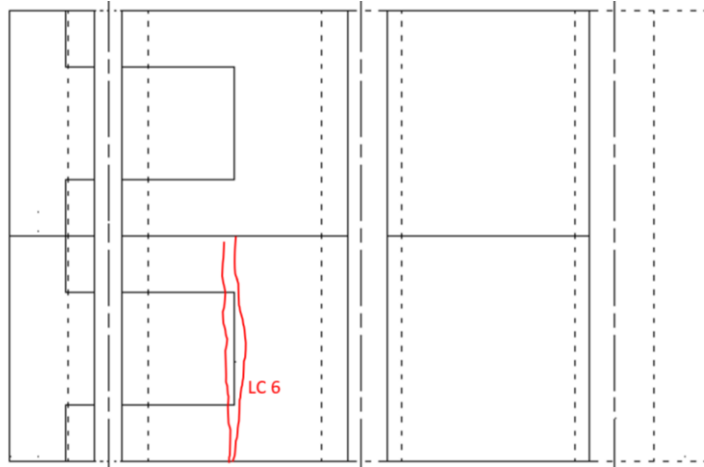


Figure 31. Crack map on the bottom after LC6

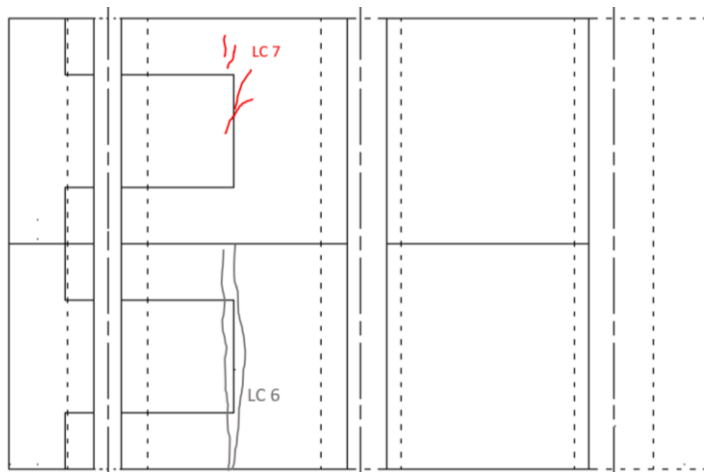


Figure 32. Crack map on the bottom surface of deck after LC7

LC6 and LC7 generated cracks on the bottom surface of only the non-prestressed interior deck panels, as shown in Figure 33 and Figure 34.



Figure 33. Crack on the bottom surface of deck after LC6

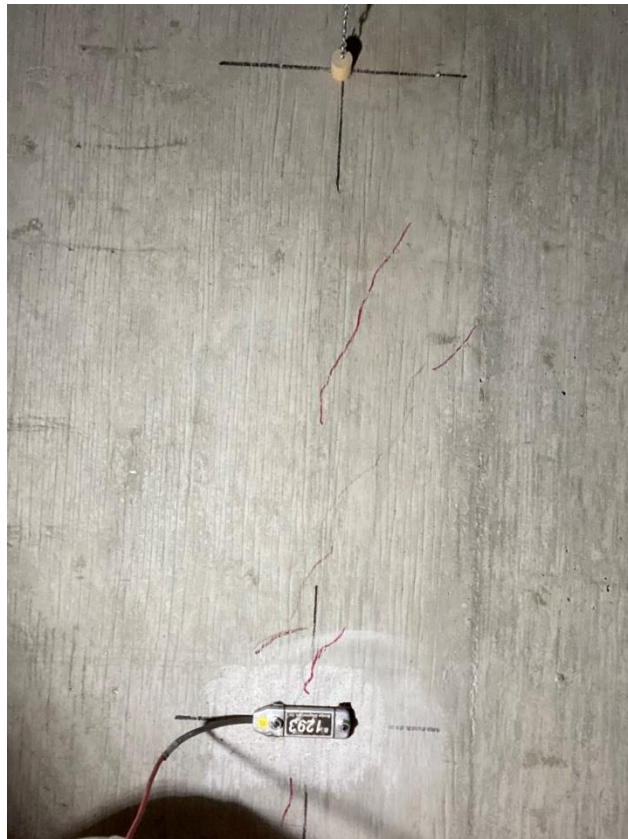
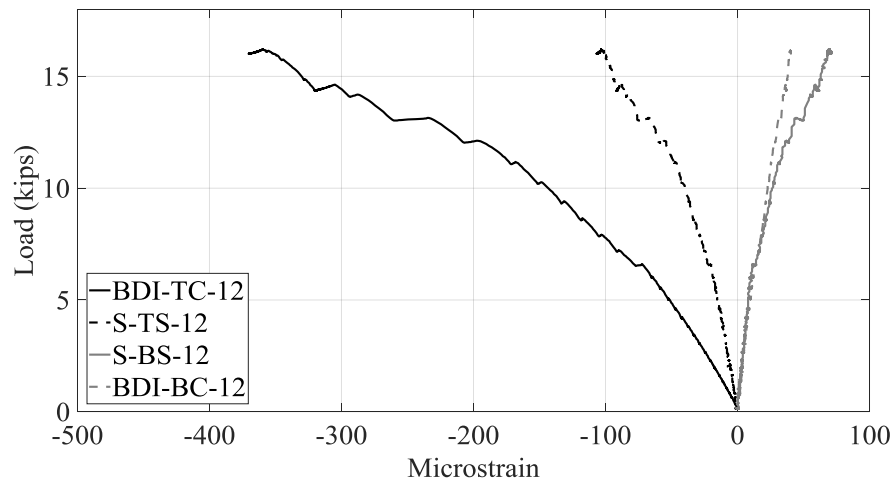


Figure 34. Crack on the bottom surface of deck after LC7

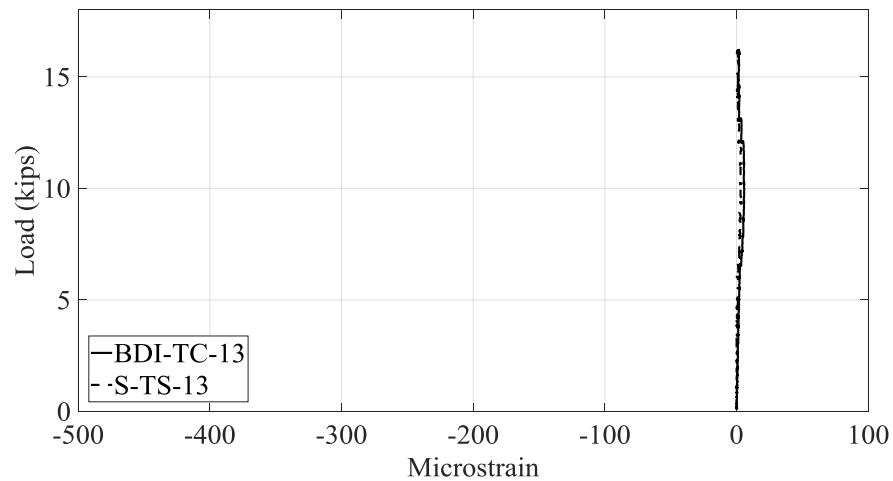
These cracks are parallel to the traffic direction and were induced by the transverse moment. No cracks were found on the prestressed interior deck panel given the pre-stressing force compresses the precast deck panel and resists the tensile stress development in the lower level of the deck.

4.6.3.2 Test Data

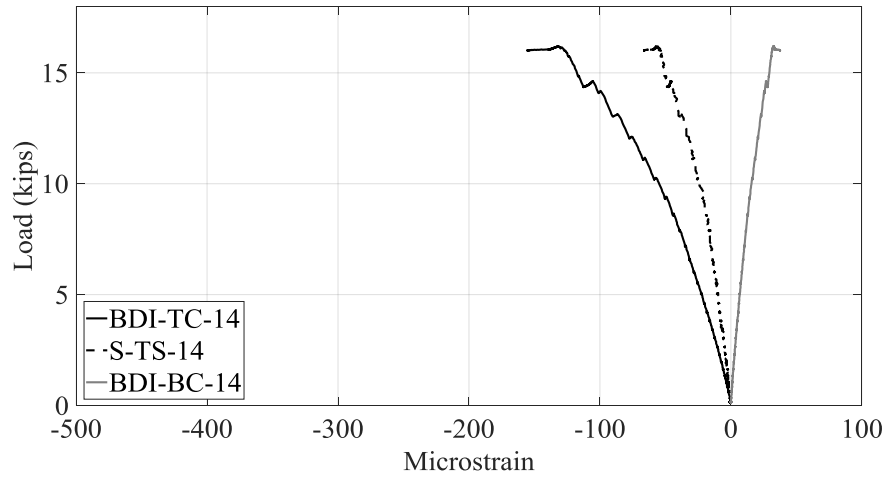
Figure 35 and Figure 36 plot the strain and displacement data from selected gauges for LC6 and LC7, respectively. Note that the displacement data for LC6 are not presented given the results were nearly zero. The strain data in Figure 35-a, -c and Figure 36-a, -b, and -c indicated that a positive moment was generated at the center of the deck between the girders.



a) Strain data at Location-12



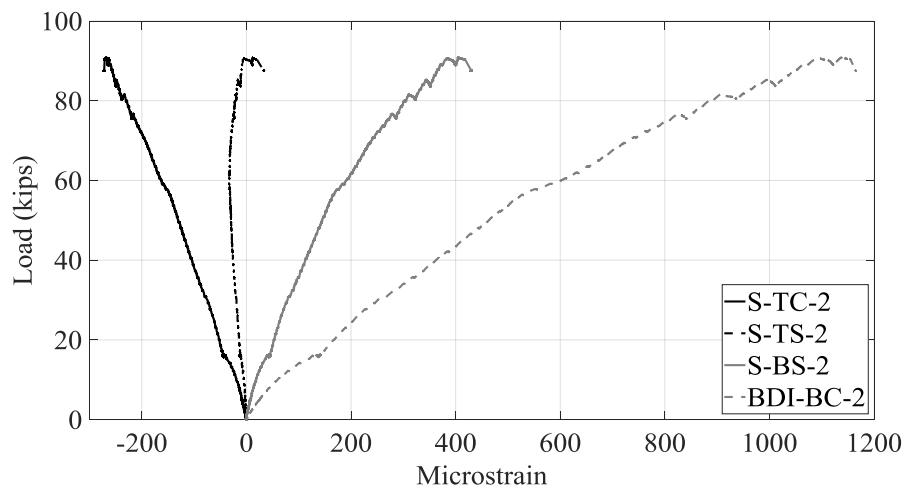
b) Strain data at Location-13



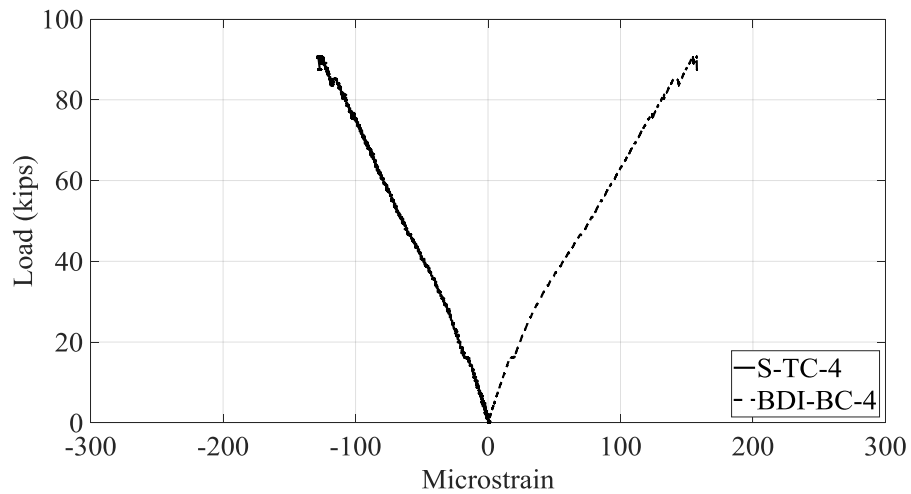
c) Stain data at Location-14

Figure 35. Strain and displacement data from LC6

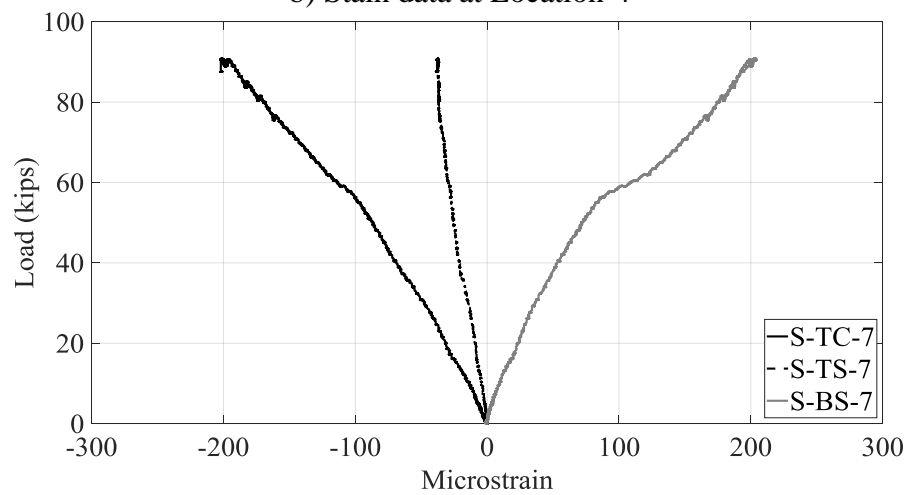
The large strain measured from the strain gauge at the bottom of the deck surface (BDI-BC-2 in Figure 36-a) indicated that the gauge was over one or more cracks.



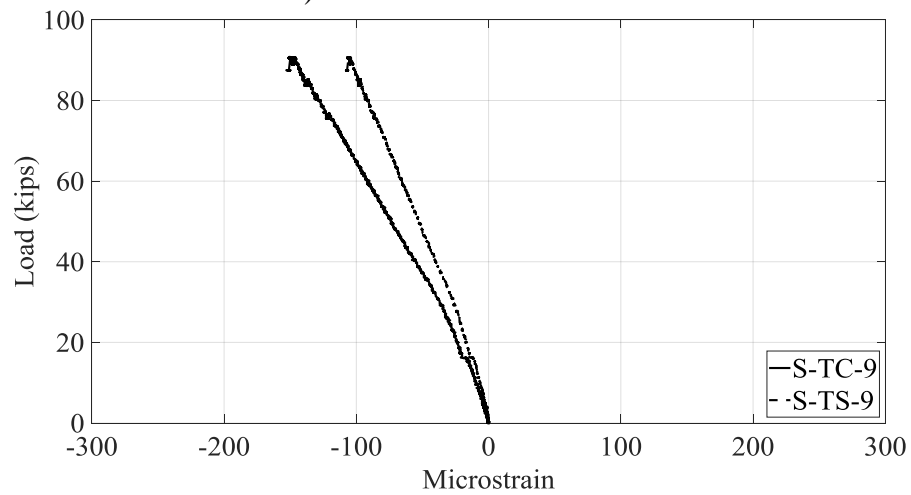
a) Stain data at Location-2



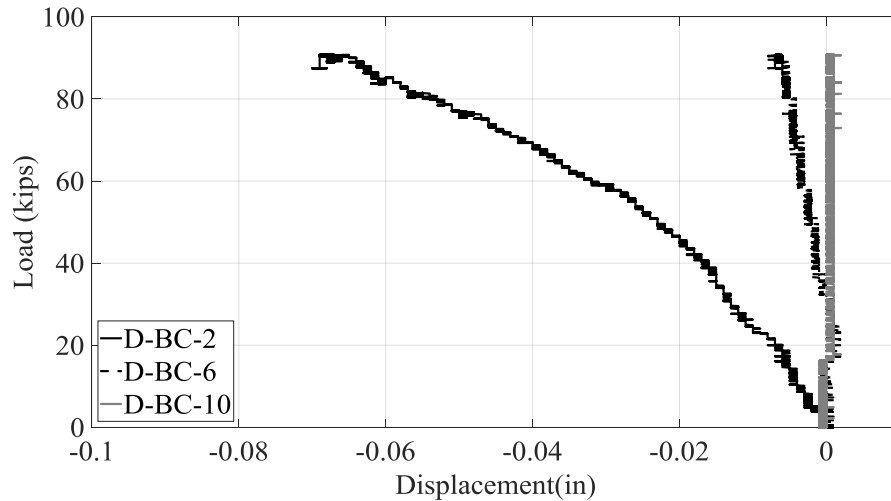
b) Stain data at Location-4



c) Stain data at Location-7



d) Stain data at Location-9



e) Selected displacement data (I)

Figure 36. Strain and displacement data from LC7

This agrees with the inspection results as a few cracks were found near gauge BDI-BC-2, with one crack propagating underneath this gauge. Figure 35-b confirms that a negative moment developed over the girder when the strain on the top surface and in the top layer reinforcing steel bar are in compression.

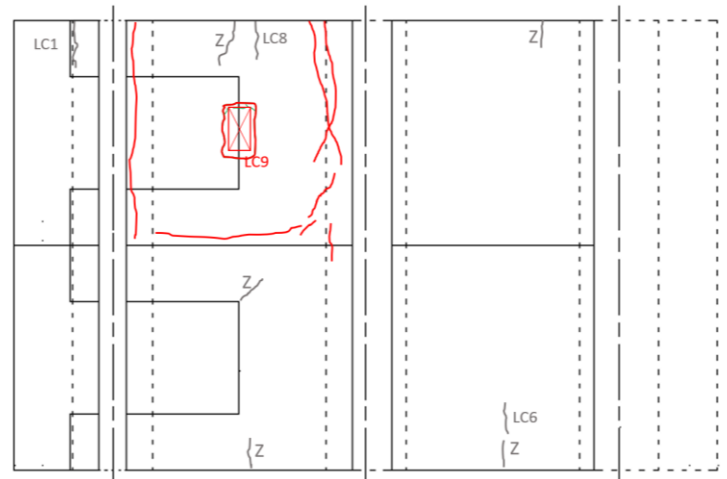
The displacement in Figure 35-d and -e indicated that, when the load is applied at the center of a non-prestressed interior panel, a large displacement occurs on that panel. However, the displacement at the center of the adjacent panel is nearly zero. This indicates that the majority of the load was carried by the single precast panel. Unfortunately, the three displacement transducers (D-BC-3, D-BC-7, and D-BC-11) under the prestressed interior deck panel lost connection during LC6 and LC7 so that data were not available.

4.6.4 LC8 and LC9

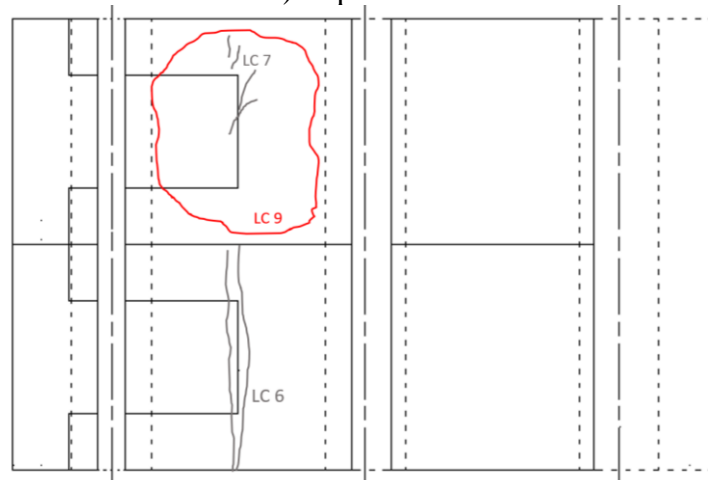
LC8 and LC9 were conducted to break the precast, non-prestressed deck panel. LC8 included a loading pad perpendicular to the traffic direction and loaded to 200 kips, which was the load limit of the actuator. However, the specimen was not yet loaded to capacity. LC9 was then conducted with a larger capacity actuator and loaded with the loading pad turned 90° (parallel to the traffic direction). (See the previous Figure 21 for more detailed loading information.) The specimen reached ultimate capacity when loading reached about 250 kips.

4.6.4.1 Crack Maps

The cracks on the top and bottom surface were mapped after the completion of LC9, as shown in Figure 37.



a) Top surface



b) Bottom surface

Figure 37. Crack maps after LC9

A few longitudinal cracks (parallel to the traffic direction) were generated on top of the deck near the edge of the girder. These cracks were induced by the negative transverse moment over the girder.

After the removal of the loading pad, punching shear failure was found, as shown in Figure 38-a.



a) Top of the deck after removal of load pad



b) Side of the precast non-prestressed deck panel



c) Spalling of deck concrete at the bottom of the deck

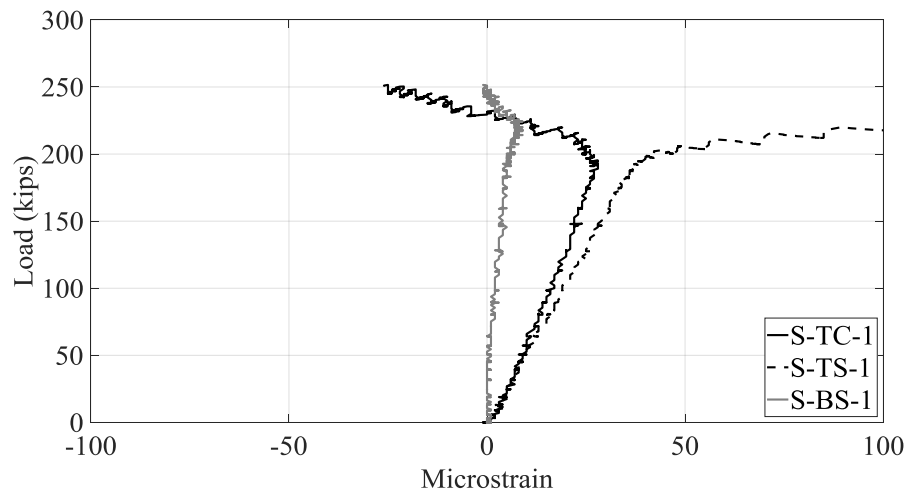
Figure 38. Crack situation on the deck after LC9

After the test, a few vertical cracks were found at the side of the deck in the middle and near the bottom surface (shown in Figure 38-b). These cracks were likely induced by the positive moment in the middle of the deck. At the bottom surface of the deck, concrete spalling occurred when ultimate loading was reached (Figure 38-c). The spalling at the bottom surface significantly reduced the cross-sectional area of the deck.

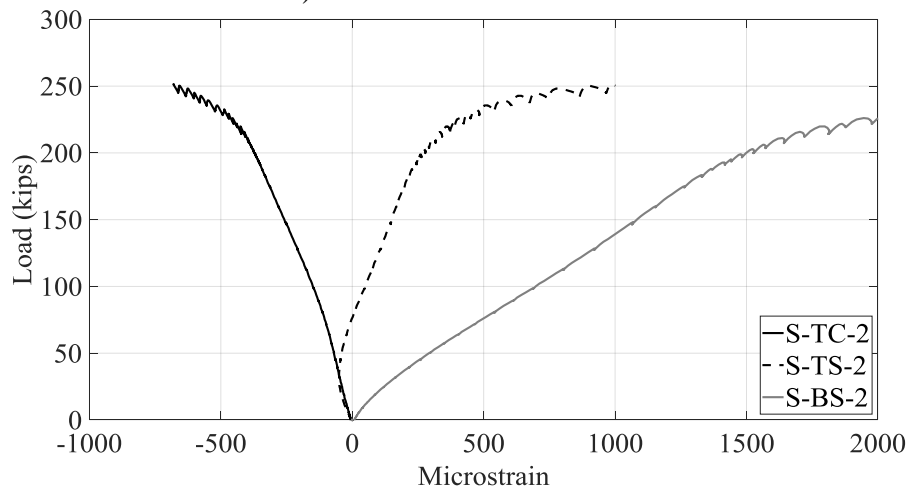
During the test, no significant debonding or sliding was found at the interface between the precast and CIP deck. The precast non-prestressed deck panel exhibited good composite action.

4.6.4.2 Test Data

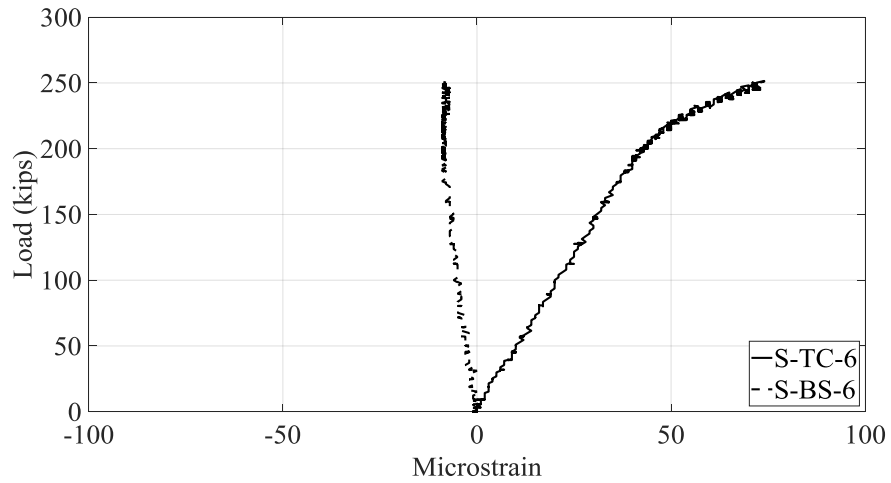
Figure 39 shows the strain and displacement data collected during LC9.



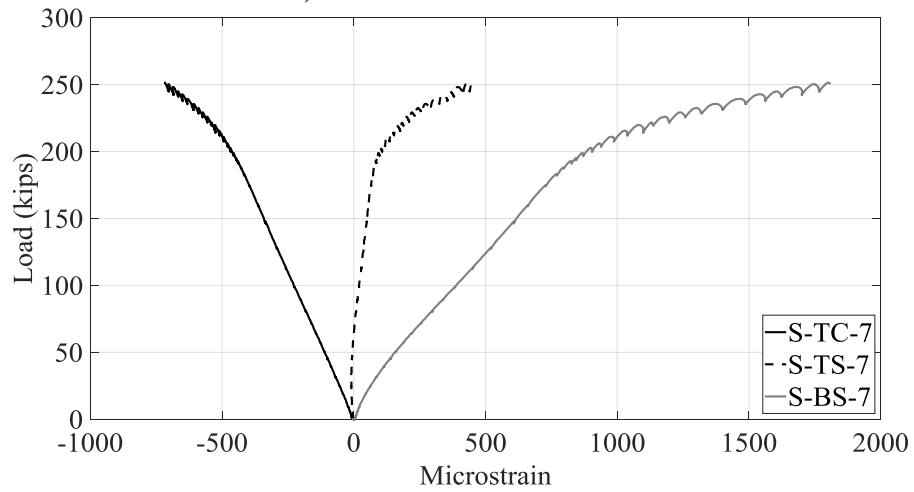
a) Stain data at Location-1



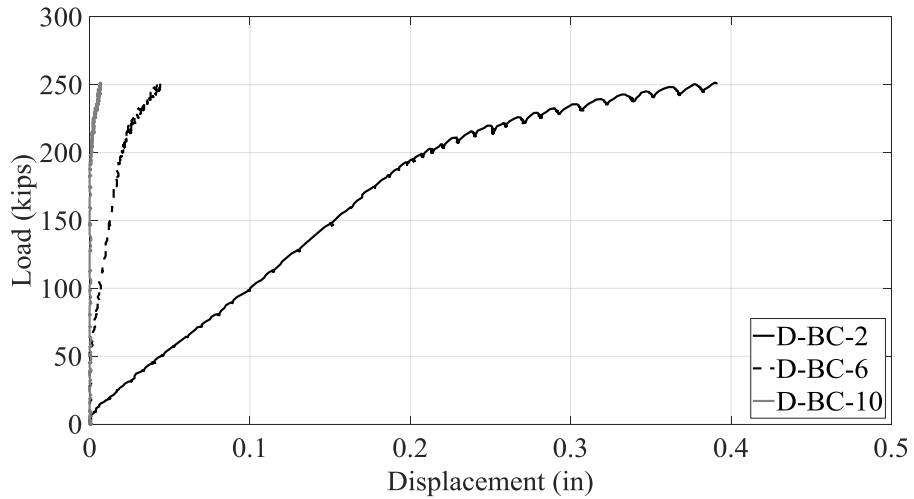
b) Stain data at Location-2



c) Stain data at Location-6



d) Stain data at Location-7



e) Selected displacement data

Figure 39. Strain and displacement data from LC9

The strain results indicated that the positive transverse moment occurred in the middle of the deck panel and negative moment occurred over the girders. The displacement data in Figure 39-e confirms the findings from LC7, that when the load is applied at the center of a non-prestressed interior panel and a large displacement occurs on that panel, the displacement at the center of the adjacent panel is very small.

4.6.5 LC10

Similar to LC9, the precast prestressed deck panel was loaded at the single loading point with the loading pad parallel to the traffic direction. The deck failed when the load reached 240 kips.

4.6.5.1 Crack Maps

Figure 40 shows the crack maps of the top and bottom surface of the deck after LC10.

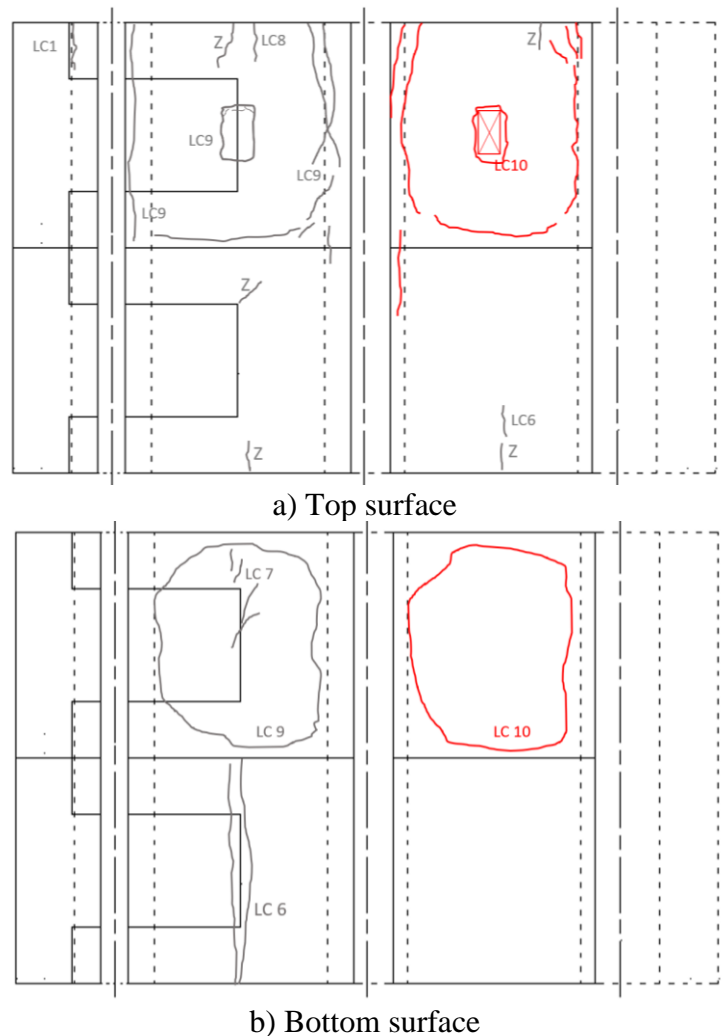


Figure 40. Crack maps after LC10

Flexural cracks were found on the top surface due to the negative moment.

Figure 41-a shows the punching effect on the top deck surface directly underneath the loading pad.

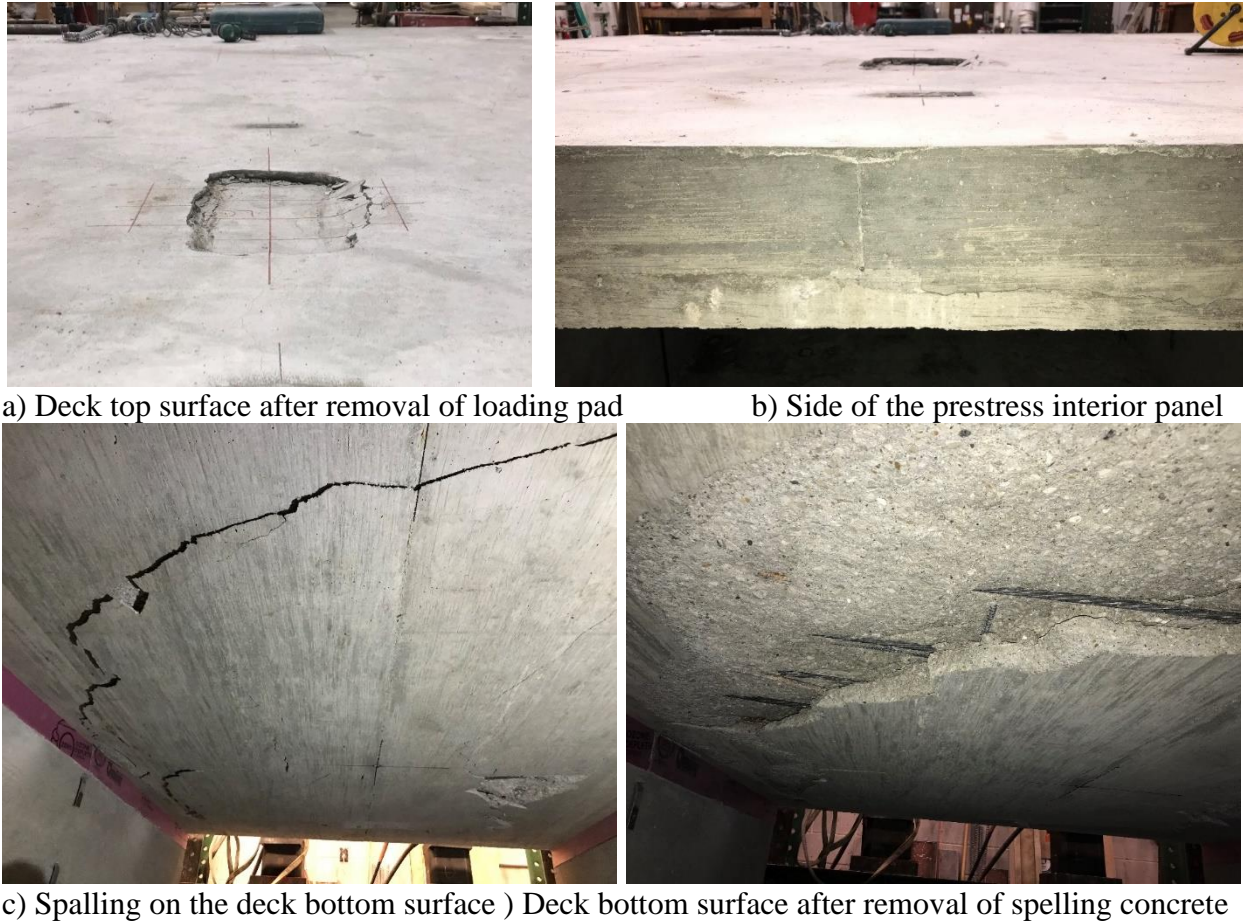


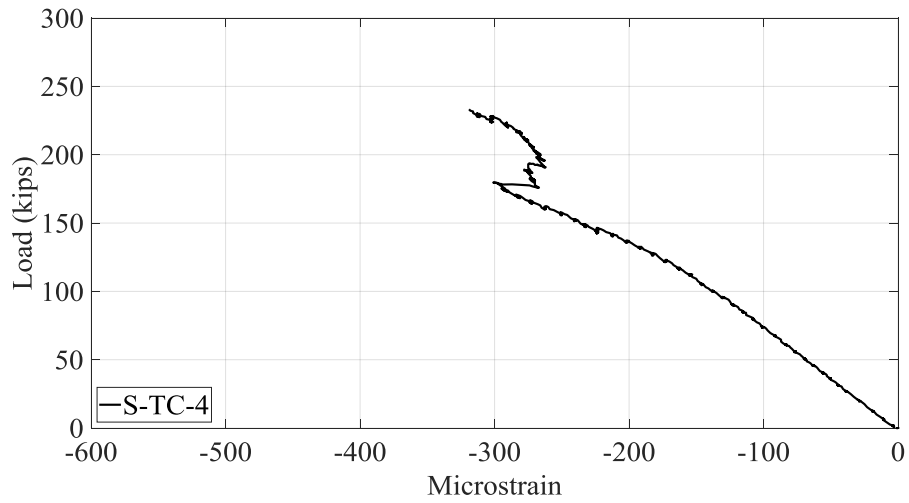
Figure 41. Crack situation on the deck after LC10

Unlike the non-prestressed deck panel, no cracks were found on the side of the prestressed deck panel. This is due to the prestressing force at the lower level precast deck. The spalling of the bottom deck concrete was found, as shown in Figure 41-c, and Figure 41-d shows the bottom of the deck after removal of the spalled concrete. It was found that the spalling of the concrete occurs in the concrete cover underneath the bottom layer reinforcing steel bar.

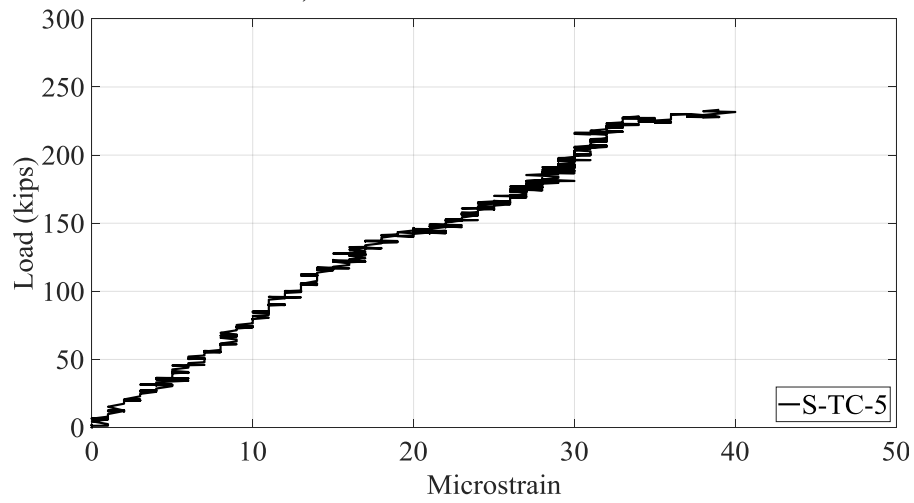
Similar to the results from LC9, no debonding or sliding was observed at the horizontal surface between the CIP and precast deck concrete.

4.6.5.2 Test Data

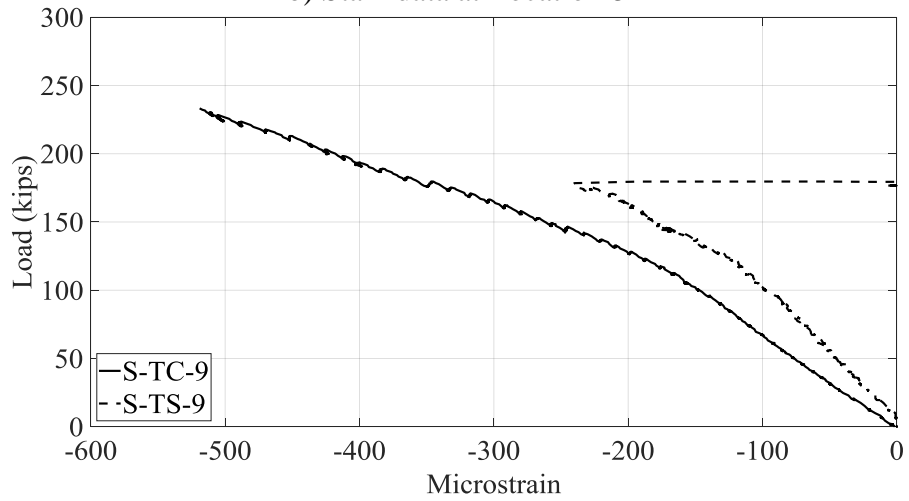
Figure 42 shows the strain and displacement data collected during LC10.



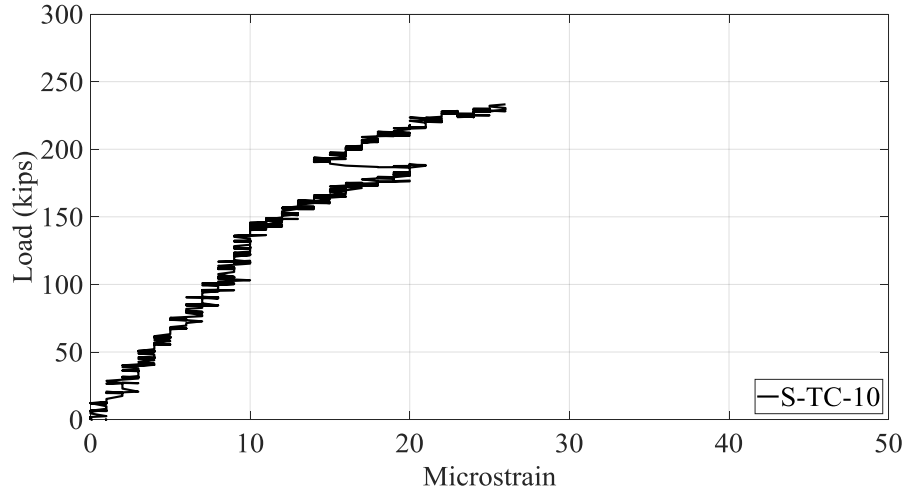
a) Stain data at Location-4



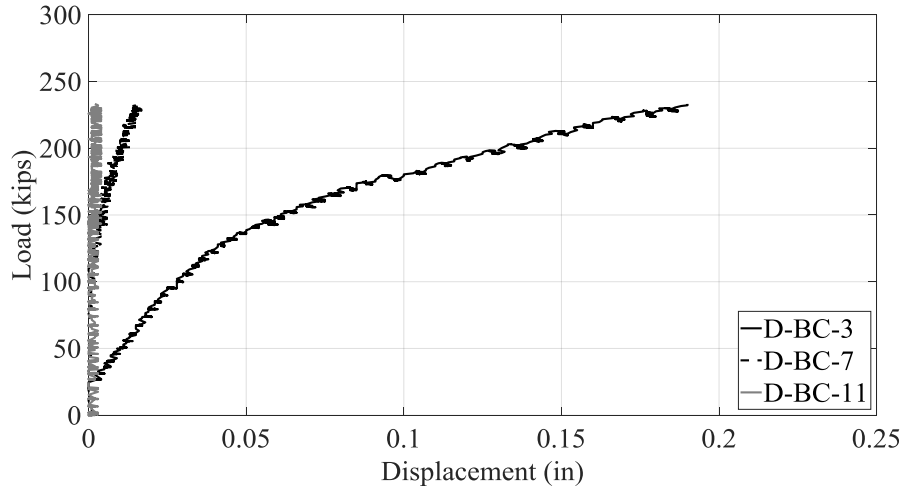
b) Stain data at Location-5



c) Stain data at Location-9



d) Stain data at Location-10



e) Selected displacement data

Figure 42. Strain and displacement data from LC10

Similar to LC9, the results indicated that the positive transverse moment occurred in the middle of the deck panel and negative moment occurred over the girders. The displacement data in Figure 42-e indicated that the load applied on one prestressed interior panel was mainly carried by that single panel, given the vertical displacement at the center of the adjacent deck is minimal.

Comparing the displacement data in Figure 42-e to that in Figure 41-e, it was found that the prestressing force in the precast deck panel significantly reduced the vertical deformation of the deck panel when it was subjected to the vertical load.

CHAPTER 5. SUMMARY AND CONCLUSIONS

In this research project, the laboratory tests were conducted to validate design assumptions and evaluate the performance of the structural components and construction approaches provided in the design documents of a PPCB-supported, partial-depth precast deck system with propped cantilever precast overhang panels that were planned to replace the bridge that carries US 52 and IA 64 over the Mississippi River overflow.

Although the proposed precast deck system was instead replaced with a conventional CIP deck, the research work was continued in the laboratory using the partial-depth precast deck panel system with precast overhang panels. The specific objectives of this research work included the following: 1) evaluation of the sufficiency of the proposed polyethylene bearing between the precast deck panels and the supporting girders, 2) verification of sufficiency and constructability of using flexible polyethylene foam as a sealing strip in conjunction with leveling bolts to create an adjustable haunch, and 3) evaluation of the composite action between the CIP concrete deck and the precast panel.

To achieve the objectives, laboratory tests were conducted on two specimens with horizontal loading on the barrier and vertical loading at various locations of the deck panels. The deck of each specimen generally consisted of two propped cantilever precast overhang deck panels, two precast, prestressed interior panels, and a portion of the CIP concrete deck.

The specimen details and construction work were carefully documented. The results indicated that the high-density polyethylene foam has sufficient stiffness and strength to support the precast deck panels and the construction load during concrete placement of the deck. The leveling bolt with normal polyethylene foam worked fine to support the deck panel and to resist the lateral concrete load. However, special attention during the gluing of the polyethylene foam is needed to ensure a good bond between the girder/deck concrete surface and the polyethylene foam.

The load test results indicated that both types of interior panels reached ultimate capacity when a point load of about 240 to 250 kips was applied, leading to punching shear failure, exceeding the demands of the bridge service life. The composite action between the CIP and precast deck concrete is functional through the load application process, and no debonding or sliding was found at the horizontal interface between the CIP and precast concrete.

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https://www.pci.org/PCI_Docs/Publications/PCI%20Journal/1998/May-June/Full%20Depth%20Precast,%20Prestressed%20Concrete%20Bridge%20Deck%20System.pdf.

APPENDIX. RESPONSES ON THE INTERVIEW QUESTIONNAIRE

A.1 Response from a Non-Winning Bidder

1. At the beginning of the project, what were seen as the benefits of the **Partial-depth Precast Deck System** over the **AccelBridge Deck System**?

I do remember this project. We actually teamed up with Cramer & Associates on it to ensure that we had the staff to meet the schedule. We went with the partial-depth precast panel option in our bid. We have a lot of experience with concrete deck panels with all of the work we perform in the southern states.

2. What concerns did you have with the **AccelBridge Deck System** and the **Partial-depth Precast Deck System**?

The full-depth panels looked very cumbersome and tough to construct. We were concerned that there could be a lot of risk and high potential with issues in the field given we didn't have any experience with that style.

3. Why did you submit a VE proposal to change the deck system from the **Partial-depth Precast Deck System** to **conventional method**?

Given our experience with the partial-depth deck panels, I doubt we would have submitted a V.E. to perform it conventionally. It is really nice not having to strip a bridge deck because that is probably one of the most unsafe operations we perform. It is amazing there aren't more people hurt stripping than there are.

I think through crew staffing and working additional hours, most northern contractors would rather perform the deck conventionally because we can place it very quickly and it is one operation that you can put additional workers on and get expedited results. A lot of our other operations don't necessarily go faster by just putting more people on it. If you have to ultimately construct pier diaphragms and abutment diaphragms, they are the activities that take the most time and while a few guys are building them, the rest of the deck can typically be placed before the diaphragms are finished.

I think the best way to expedite the superstructure would be by using partial-depth precast panels and figure out a way to eliminate the pier and abutment diaphragms. You still have to build an overhang, which takes time, but the overhang can be installed quickly.

A.2 Response from the Winning Bidder

1. At the beginning of the project, what were seen as the benefits of the **Partial-depth Precast Deck System** over the **AccelBridge Deck System**?

Partial-depth Precast Deck allows the use of stay-in-place decking. Minimal stripping needs and requirements. It is a Cheaper alternative to the AccelBridge deck. We anticipated the labor cost on the AccelBridge Deck System was about the same.

2. What concerns did you have with the **AccelBridge Deck System** and the **Partial-depth Precast Deck System**?

AccelBridge: Price is high and the Cold joints in the deck from UHPC and future maintenance issues.

Partial-depth Precast Deck System: Very little time savings on the schedule (if any).

3. Why did you submit a VE proposal to change the deck system from the **Partial-depth Precast Deck System** to **conventional method**?

The conventional method provides the best product at the best price within our schedule window.

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