Precast Full-Depth Deck Panels Supported on Inverted Bulb-Tee Bridge Girders

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PRECAST FULL-DEPTH DECK PANELS SUPPORTED ON INVERTED BULB-TEE BRIDGE GIRDERS

BY

MICHAEL JAMES MINGO

A thesis in partial fulfillment of the requirements for the

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This thesis is approved as a creditable and independent investigation by a candidate for the Master of Engineering degree and is acceptable for meeting the thesis requirements for this degree. Acceptance of this thesis does not imply that the conclusions reached by the candidate are necessarily the conclusions of the major department.

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<tbody>
<tr>
<td>AASHTO</td>
<td>American Association of State Highway and Transportation Officials</td>
</tr>
<tr>
<td>ABC</td>
<td>Accelerated Bridge Construction</td>
</tr>
<tr>
<td>ACM</td>
<td>Advanced Composite Material</td>
</tr>
<tr>
<td>AFRP</td>
<td>Aramid Fiber Reinforced Polymer</td>
</tr>
<tr>
<td>CFCC</td>
<td>Carbon Fiber Composite Cable</td>
</tr>
<tr>
<td>CFRP</td>
<td>Carbon Fiber Reinforced Polymer</td>
</tr>
<tr>
<td>CIP</td>
<td>Cast-in-place</td>
</tr>
<tr>
<td>COF</td>
<td>Coefficient of Friction</td>
</tr>
<tr>
<td>DOT</td>
<td>Department of Transportation</td>
</tr>
<tr>
<td>FE</td>
<td>Finite Element</td>
</tr>
<tr>
<td>FHWA</td>
<td>Federal Highway Administration</td>
</tr>
<tr>
<td>Ft</td>
<td>Feet</td>
</tr>
<tr>
<td>FDDP</td>
<td>Full-depth Deck Panels</td>
</tr>
<tr>
<td>HSS</td>
<td>Hollow Structural Steel</td>
</tr>
<tr>
<td>in.</td>
<td>Inch</td>
</tr>
<tr>
<td>Kip</td>
<td>1000 pounds</td>
</tr>
<tr>
<td>Klf</td>
<td>Kip per linear foot</td>
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<tr>
<td>kN</td>
<td>Kilonewton</td>
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<tr>
<td>ksi</td>
<td>Kip per square inch</td>
</tr>
<tr>
<td>lbs</td>
<td>Pounds</td>
</tr>
<tr>
<td>LRFD</td>
<td>Load and Resistance Factor Design</td>
</tr>
<tr>
<td>LVDT</td>
<td>Linear Voltage Differential Transformer</td>
</tr>
<tr>
<td>m</td>
<td>Meter</td>
</tr>
<tr>
<td>mm</td>
<td>Millimeter</td>
</tr>
<tr>
<td>NBI</td>
<td>National Bridge Inventory</td>
</tr>
<tr>
<td>NCHRP</td>
<td>National Cooperative Highway Research Program</td>
</tr>
<tr>
<td>psi</td>
<td>Pound per square inch</td>
</tr>
<tr>
<td>SD</td>
<td>South Dakota</td>
</tr>
<tr>
<td>Acronym</td>
<td>Description</td>
</tr>
<tr>
<td>---------</td>
<td>--------------------------------------------------</td>
</tr>
<tr>
<td>SDDOT</td>
<td>South Dakota Department of Transportation</td>
</tr>
<tr>
<td>SDSU</td>
<td>South Dakota State University</td>
</tr>
<tr>
<td>SLT</td>
<td>Stress-laminated-timber</td>
</tr>
<tr>
<td>UHPC</td>
<td>Ultra-high Performance Concrete</td>
</tr>
<tr>
<td>VDOT</td>
<td>Virginia Department of Transportation</td>
</tr>
<tr>
<td>VHPC</td>
<td>Very-high Performance Concrete</td>
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ABSTRACT

PRECAST FULL-DEPTH DECK PANELS SUPPORTED ON INVERTED BULB-TEE BRIDGE GIRDERS

MICHAEL JAMES MINGO

2016

The South Dakota Department of Transportation (SDDOT) currently uses precast double-tee bridge deck systems for many of its county bridges because they are economical and fast in construction. Current bridges are designed for a service life of 75 years. However, many double-tee girder bridges are deteriorating and some need total replacement after 40 years in service. Furthermore, the double-tee bridge system only has one supplier in South Dakota. Alternative durable precast or prefabricated bridge systems are needed to provide more options to local governments when designing a new bridge. Different alternatives will also give local governments more flexibility to select the best system by comparing performance, availability, and cost of different options.

The present study was carried out to investigate the feasibility of alternative prefabricated bridge systems that can be incorporated in South Dakota. The project technical panel approved testing of two superstructure bridge systems: (1) precast full-depth deck panels on prestressed inverted bulb-tee girders, and (2) glulam timber bridges. The present report includes the design, construction, and testing methods of the first bridge alternative.

The proposed bridge system (precast full-depth deck panels on prestressed inverted bulb-tee girders) was designed based on a 50-ft long by 34.5-ft wide prototype
bridge. The full-scale test bridge specimen was 50-ft long by 9.5-ft wide representing two interior girders from the prototype bridge. The bridge was first tested under 500,000 cycles of the AASHTO Fatigue II loading using a point-load applied at the mid-span. Next, the performance of transverse joints was evaluated by applying 150,000 AASHTO Fatigue II load cycles using two point loads applied adjacent to the middle panel transverse joints to maximize the shear transfer. Stiffness tests were performed at every 50,000 load cycle interval for both fatigue tests. No significant damage in addition to the shrinkage cracks was observed through the entire fatigue test, and the overall bridge stiffness did not show any signs of deterioration. Finally, the proposed bridge system was monotonically loaded to 263 kips to investigate the ultimate capacities. It was shown that the first crack loading magnitude was higher than the equivalent AASHTO Service and Strength I limit states, indicting sufficient performance. The design and construction of the proposed bridge system are simple and similar to current practice. Based on the construction, testing, and cost analysis, it can be concluded that the proposed bridge system, precast full-depth deck panels on prestressed inverted bulb-tee girders, is a viable alternative to the double-tee girder bridges.
1. Introduction

This report presents a study that was performed at South Dakota State University (SDSU) to develop different alternatives to double-tee bridge systems that are common in South Dakota (SD) on local roads.

An extensive literature review was performed to investigate the feasibility of existing bridge systems that might be viable alternatives for SD. Based on the typical properties of local bridges, a few criteria were selected to narrow down the literature review to alternatives that (1) are suitable for single-span bridges with a length of 70 feet or less, (2) can withstand the ASSHTO HL93 load, (3) are designed for the service life of 75 years, and (4) incorporate accelerated bridge construction techniques.

1.1 Problem Statement

Numerous bridges on the South Dakota local highway system are in need of replacement. South Dakota has 5,870 bridges, of which, 1,208 are structurally deficient and 237 are functionally obsolete according to the Federal Highway Administration (FHWA) (2012). This equates to 24.6 percent of bridges in South Dakota being structurally deficient or functionally obsolete. There are more than 700 bridges in SD with precast double-tee girder systems mainly because they are rapidly constructible and economical. Bridges are designed for a service life of 75 years. However, many of these
bridges are showing signs of deterioration and some are in need of replacement after 40 years.

The main problem associated with the currently used precast double-tee system (Fig. 1.1) is that it develops reflective cracking along the longitudinal joints. This is caused by an inadequate longitudinal joint detail that utilizes discrete welded plates to transfer shear forces through the joint. The reflective cracking provides a pathway for water and de-icing agents to seep through the joints, spall the concrete, and reach prestressing steel tendons. Accelerated deterioration begins when the joint starts cracking and corrosion starts to occur when water reaches the prestressing steel tendons.

The double tee bridge system only has one supplier in South Dakota. Alternative durable precast bridge systems are needed to provide more options to local governments when designing a new bridge. Different alternatives will also give local governments more flexibility to select the best system by comparing performance, availability, and cost of different options. The present study was carried out to investigate the feasibility of alternative precast bridge systems that can be incorporated in South Dakota.
1.2 New Double Tee Joint Detail

It was mentioned that the existing detailing for double tee girders (Figure 1.1) is not satisfactory due to poor durability issues. In an attempt to improve the detailing, Konrad (2014) performed a study at SDSU to develop a new detail that prohibited reflective cracking in the precast double tee girder longitudinal joints (Figure 1.2).
The new detail has a wider grouted keyway (4 in.) with overlapping welded wire meshes (4 x 8 D8.0 x D4.0) extending from the double tee section top flange to be spliced with adjacent girder wire meshes. The welded wire meshes were developed in the transverse direction by placing two 0.225-in. wires longitudinally in the joint spaced every 2 inches. Concrete spacers were placed in between the adjacent double tee section webs at 5 ft along the girder in the longitudinal direction and were tied to the webs using a ¾-in. bolt to limit the relative rotation of the adjoining girders. However, the revised longitudinal joint detail performed very well without this additional component so they were deemed unnecessary.

Two full-scale double tee girder systems were tested under fatigue and strength loading: one specimen with the existing longitudinal joint detail (conventional specimen) and one specimen with new joint detail (proposed specimen). The test results confirmed that the current double tee joint detail is inadequate because the first welded steel plate failed at 62,000 load cycles using the AASHTO LRFD Bridge Design Specifications (2013) fatigue load. This number of cycles is equivalent to 11.3 years of service load. However, the proposed longitudinal joint detail performed well under the fatigue testing. This joint did not show any deteriorate under 800,000 load cycles.
A point-load was applied at mid-span of the full-scale bridge girders adjacent to the longitudinal joint in order to simulate stresses that would be induced from vehicular loading. The stiffness of both the conventional specimen and proposed specimen was plotted with respect to the number of fatigue load cycles (Fig. 1.3). The conventional specimen stiffness degraded rapidly under fatigue loading. The proposed specimen stiffness did not change throughout the test. Figure 1.3a and Figure 1.3b show the stiffness of the conventional and proposed specimens versus load cycles, respectively.

![Girder Stiffness Test Results](Konrad, 2014)

Figure 1.4 shows the results for the strength test. Figure 1.4a shows that the two girders of the conventional specimen did not act as a monolithic member through the duration of the strength test because the longitudinal joint failed before the girder failure. Vertical loads were resisted by only the loaded girder as the discrete welded connections failed along the joint. However, Fig. 1.4b shows that both girders of the specimen with new joint detailing behaved monolithically throughout the strength test. This specimen failed in a ductile manner.
1.3 Objectives and Scope

The main objectives of the present study are to: (1) identify or develop new bridge systems that can resist the AASHTO HL93 load requirements, can span up to 70 feet, and have a design life of at least 75 years, (2) perform ultimate and fatigue testing on the selected alternative bridge systems, and (3) compare cost, constructability, and performance of the selected alternative bridge systems with the existing double tee girder decks.

A literature review was performed to identify new bridge system alternatives to the double tee girders that are suitable for SD. The good candidates were ranked for the selection by SDDOT. It is expected that SDDOT will select at least two new bridge systems. The selected alternatives will be constructed, instrumented, and tested under fatigue and ultimate loading to determine the performance of these alternatives. Fatigue loads are based on AASHTO (2013) to simulate traffic loading that the bridge would be subjected to in its 75-year design life.
Test results will be compared to those of the existing and revised double tee systems. The comparison will specifically include performance, constructability, cost, and strength of each system. Finally, a recommendation will be developed based on the aforementioned parameters.
This chapter includes a literature review of single-span bridge systems that might be considered as an alternative to double tee girders. Nine alternative bridge systems are introduced and reviewed herein: (1) full-depth deck panel precast concrete systems, (2) voided slab bridges, (3) ultra-high performance concrete waffle deck panels, (4) carbon fiber composite cable prestressed decked bulb-tee beams, (5) bridge decks reinforced with aramid fiber reinforced polymer, (6) stress-laminated-timber bridge decks, (7) glulam timber bridges, (8) advanced composite materials bridges, and (9) recycled plastic bridges.

2.1 Full-depth Deck Panel Precast Concrete Systems

A full-depth deck panel (FDDP) system allows for rapid construction since the deck panels and girders are precast and are connected at the construction site. The major components of this system are precast full-depth deck panels and precast prestressed concrete or steel girders. Figure 2.1 shows the general detail for FDDP supported on prestressed I-girders.

The panel-to-girder connections consist of a series of shear pockets in the precast concrete panels aligned with the girder centerlines. Either steel U-shape bars or headed shear studs are extended from the girders vertically to enter the shear pockets as the
panels are placed. The precast panels usually have embedded leveling bolts, which initially support the panels on the girder top flange to adjust the elevation of the panels upon placement. The shear pockets are subsequently filled with grout and the grout flows through a haunch void between the panel-girder interface. The grouted shear pockets and haunch creates composite action between the panels and girders.

Badie and Tadros (2008) performed a study to develop guidelines for design, fabrication, and construction of FDDP systems, and to develop connection details to eliminate the need for post-tensioning of the transverse joints.

The panel-to-panel connections usually consist of a grouted keyway and longitudinal post-tensioning tendons. However, Badie and Tadros (2008) developed two connection details that utilize longitudinal mild reinforcement steel bars extending from adjacent decks, which are spliced and confined to develop the full yield strength of the
steel bars. Chapter 3 presents more information about FDDP system component alternatives.

2.2 Voided Slab Bridges

Joyce (2014) investigated the suitability of voided slab bridge systems. The objective of the research was to develop an improved longitudinal joint detail between precast voided slabs to increase the durability and performance. The project was funded by the Virginia Department of Transportation (VDOT).

A partial depth grouted shear key was tested as a control specimen, since it is currently used by VDOT for voided slab bridges. Five other sub-assemblage voided slab specimens were tested with various materials and details. Figure 2.2 shows the sub-assemblage used to test the various longitudinal joint grouting materials with the blockout connection detail. The improved detail consisted of a blockout connection where dowel bars extended into 6-in. pockets from adjacent voided slabs. A 6-in. long conventional steel rebar was tied to bars extending from each voided slab to splice the steel. The joint filler materials were ultra-high performance concrete (UHPC), very-high performance concrete (VHPC), and a proprietary material named “Kevlar”. A finite element analysis software, ABAQUS, was used to model the bridge to reproduce the test data.

The results showed that the current longitudinal joint detail used by VDOT is inadequate since the test sample failed at 94 load cycles. The Kevlar reinforced grouted shear key connection performed better than the current detail, but did not abate cracking. The blockout connection with UHPC and VHPC exhibited suitable performance and cracking was prevented. The sample underwent more than one million load cycles...
without failure. The VHPC connection did not leak during the ponding tests and had an ultimate to service load ratio of 8.5. The VHPC combined with the blockout detail was recommended since it had similar performance characteristics as UHPC and is cheaper.

**Figure 2.2- Sub-assemblage of Voided Slab System (Joyce, 2014)**

### 2.3 Ultra-high Performance Concrete Waffle Deck Panel

Aaleti and Sritharan (2014) investigated the performance of UHPC waffle deck panels (Fig. 2.3), which is similar to FDDP systems. This system utilizes a UHPC waffle deck supported by steel or prestressed precast concrete girders. The structural efficiency of the waffle deck geometry combined with the strength characteristics of UHPC allows for 30 to 40% reduction in slab weight.
The research by Aaleti and Sritharan (2014) investigated multiple simple and detailed finite element (FE) analyses of waffle decks in ABAQUS. A design guideline was developed including recommendations on maximum rib spacing, connection details, and positive and negative moment design of panels to allow for cost-effective implementation of the UHPC waffle deck panels for bridge systems. After the analytical study, they tested three waffle deck panels with different connections. The test results
showed that the waffle decks had desirable performance characteristics when subjected to fatigue, service, and overloads. Since good performance was observed in this study, Iowa DOT utilized the UHPC waffle deck system in the replacement of a bridge in Wapello County, Iowa.

### 2.4 Carbon Fiber Composite Cable Prestressed Decked Bulb-Tee Beam

Grace et al. (2012) experimentally investigated the performance of carbon fiber composite cable (CFCC) prestressed decked bulb-tee bridges under various limit states. The study aimed to develop a bridge system that utilized accelerated bridge construction (ABC) techniques, to extend the lifespan of the bridge through replacing steel reinforcement with carbon fiber reinforced polymer (CFRP) reinforcement, and to eliminate cracking of field cast longitudinal joints using UHPC shear key joints. Figure 2.4 shows cross section details of the decked bulb-tee bridge test model.

![Cross Section Details of the Decked Bulb-Tee Bridge Model](image)

**Figure 2.4- Cross Section Details of the Decked Bulb-Tee Beam Bridge Model (Grace et al., 2012)**

Three single half-scale decked bulb-tee beams with various reinforcement materials under flexure were experimentally investigated. Subsequently, a half-scale decked bulb-tee system (Fig. 2.4) that consisted of five adjacent decked bulb-tee beams was tested. The reinforcement in the three single beams consisted of: prestressed steel strands and reinforcing bars, prestressed CFCC strands and CFCC reinforcement, and
prestressed CFRP tendons and CFRP reinforcement. These beams were simply supported and had an effective span length of 31 ft (9,450 mm). A four-point-load configuration was used to test these beams to failure. The half-scale CFCC prestressed decked bulb-tee beam bridge was made with five adjacent beams connected at the top flanges with UHPC shear keys transversely post-tensioned with CFCC strands. The bridge specimen had an effective span length of 31 ft (9.45 m) and a deck width of 8.5 ft (2.59 m). The bridge specimen was tested under four-point loading setup using 6.5-ft (1.98 m) long spreader beam. The CFCC prestressed decked bulb-tee beams were found to be less ductile than the steel reinforced prestressed decked bulb-tee beams. The CFCC bridge specimen exhibited large deflections and multiple flexural crack patterns prior to failure, which can serve as a warning sign to replace the beam. It was reported that the performance of the CFCC reinforced prestressed beam was similar to the conventionally reinforced prestressed beam under the service limit state.

2.5 Bridge Decks Reinforced with Aramid Fiber Reinforced Polymer

Pirayeh Gar et al. (2013) investigated the feasibility of replacing conventional steel bars with aramid fiber reinforced polymer (AFRP) bars in prestressed full-depth precast panel bridges (Fig. 2.5). A full-scale AFRP reinforced full-depth concrete panel was tested with prestressed AFRP in the transverse direction and non-prestressed AFRP in the longitudinal direction of the deck. The test specimen consisted of two bridge deck panels with dimensions of 5.49 x 2.44 x 0.2 m (216 x 96 x 8 in.). Three steel reinforced concrete beams supported the concrete deck slab at a spacing of 1.83 m (6 ft) on center. The results showed that full-depth precast concrete panels reinforced with AFRP had
acceptable strength and serviceability based on AASHTO requirements. The main advantage of this system is eliminating the risk of corrosion-induced deterioration since FRP does not corrode. Disadvantages of this system include a lack of research on full-scale specimens, a lack of in-service performance information, and difficulty of bending FRP bars.
Figure 2.5 - Deck Test Specimen with Aramid FRP (Pirayeh et al., 2013)
2.6 Stress-Laminated-Timber Bridge Decks

Ekholm (2013) experimentally investigated the ultimate-load carrying capacity of stress-laminated-timber (SLT) bridges and their response under non-destructive loads (Fig. 2.6). The research also explored the cause and effects of interlaminar slip as well as the applicability of current design codes for SLT bridges. Furthermore, the long-term performance of timber bridges was studied. A SLT bridge deck consisting of 84 glulam beams with a length of approximately 18 ft (5.4 m) and a width of approximately 26 ft (8 m) was constructed and tested under two point loading positions. The coefficient of friction (COF) for different wood species was found by applying varying normal forces to timber beams. The COF was determined for shear perpendicular and parallel to the wood grain. Furthermore, different joint configurations were tested. Joints consisted of wood extending past the adjacent longitudinal members and connecting the joint with a post-tensioned steel bar. Linear-elastic analysis of the full-scale deck was carried out using ABAQUS. Interlaminar slip resulted in nonlinear behavior. It was found that the ultimate-load carrying capacity of the SLT was 4.5 times larger than the serviceability limit state load.
Composite action of the SLT decks was achieved through the use of transverse post-tensioning and butt-joints that were overlapped. The friction between the contact surfaces transfers the shear forces. Shrinkage of the wood can occur if the wood has a high initial moisture content. This leads to a loss in post-tensioning force and interlaminar slip. However, re-stressing the deck tendons can help overcome this shortcoming. The use of dry wood (such as glulam) can reduce the presstressing loss associated with shrinkage. Figure 2.7 shows various prestress anchorage systems available for SLT bridge decks.
2.7 Glulam Timber Bridge

Wood is a renewable resource that is plentiful in South Dakota. Glulam timber has a good resistance to deicing agents, is lightweight, is easy to fabricate, can be constructed in any weather condition, has minimal environmental impacts, and is economical. Timber bridges also do not require any special equipment and can be constructed without highly skilled labor in a relatively short amount of time (Ritter 1990).

Since glulam bridges were recently emphasized (Fig. 2.8), long-term performance data is scarce. These bridges would also require more maintenance and routine inspections. Glulam timber bridges will deteriorate rapidly if they are exposed to moisture. Early detection of moisture is critical in extending the life of the bridge (Ritter 1990). Chapter 3 presents more information on these types of bridges.
2.8 Advanced Composite Materials Bridges

Ji et al. (2007) investigated the service performance of advanced composite material (ACM) bridges. Field testing and visual inspections were conducted in South Korea on a full-scale ACM bridge. Figure 2.9 shows the ACM bridge superstructure cross-section as well as the field placement of an ACM superstructure.

This single short-span ACM bridge was fabricated with a sandwich structure and corrugated core. The bridge superstructure was created with two longitudinal panels.
connected with a joint along the longitudinal centerline. Each module was approximately 33-ft (10-m) long and approximately 13-ft (4-m) wide. The ACM bridge superstructure was fabricated using E-glass stitched bonded fabric and vinyl ester resin.

The field test showed that the maximum deflection under two loaded lanes was 46.7% lower than the AASTHO specified deflection limit. It was reported that this bridge is performing well after two years in service. However, there is no official design criteria available for ACM bridges. Furthermore, the long-term service performance of ACM bridge superstructures is uncertain due to a lack of test data.

2.9 Recycled Plastic Bridges

A study by Chandra and Kim (2012) presented a discussion on the existing bridges built with recycled plastic materials, including a vehicular bridge. A majority of the bridges discussed in that study were railroad bridges. However, the first vehicular plastic bridge was constructed in 1998 in Missouri, which consisted of a thermoplastic deck with a rectangular cross section supported by steel girders. The bridge had a maximum live load capacity of 12.5 tons (111.2 kN). The next bridge was built in 2002 in New Jersey. It had a live load capacity of 36 tons (160.1 kN) and was the first plastic bridge that utilized I-beams. Recycled plastic bridges do not corrode. The application of plastic in bridges is a relatively new technology, which has not been extensively investigated in the field. It is worth mentioning that plastic bridge span lengths are limited to 25 ft.
2.10 Structure Alternatives for Local Roads

Jones and Oppong (2015) performed a study to categorize existing precast bridge system alternatives for use on local roads in South Dakota. Various superstructures, substructures, and foundations were examined. The superstructures reviewed included precast inverted tee systems, hollow core slabs, double tee girders, precast modified beam-in-slab systems, UHPC waffle deck panels, and adjacent channel beams. Also, precast decked bulb tee girders, old rail flatcars, and wide flange steel girders were examined based on information gathered from a survey sent to neighbor DOTs and South Dakota contractors. Substructures reviewed in the study included geosynthetic-reinforced soil abutments, mechanically stabilized earth walls, and sheet pile abutments. Various construction materials were also investigated. These included UHPC, high strength lightweight concrete, expanded polystyrene geofoam, self-consolidating concrete and cellular confinement material. Two entire bridge structure systems were examined in this study. These included a large precast box culvert and a three-sided structure. The off-system bridge catalogue developed in this research can be used as a general guideline to select optimum bridge systems for various project scenarios in South Dakota. However, the catalogue and flowchart for bridge selection are subjective since they have not yet been implemented and tested. Therefore, the information for selecting bridges from this research should only be used as an initial bridge selection aid.
2.11 Alternative Bridge Systems Summary

Eight bridge systems were reviewed. Table 2.1 presents a summary of the eight bridge systems with pros and cons of each bridge system listed. The main factors considered for comparison were: (1) cost, (2) durability, (3) ease of construction and construction time, (4) ease of fabrication, and (5) in-service performance information.
<table>
<thead>
<tr>
<th>Construction Material</th>
<th>Bridge System</th>
<th>Pros</th>
<th>Cons</th>
</tr>
</thead>
</table>
| **Concrete with Conventional Steel** | **Full-Depth Precast Concrete Panels with Precast Concrete or Steel Girders** | • Accelerated construction time  
• Versatile  
• Durable decks  
• Potential low life-cycle cost | • Higher initial cost compared to CIP bridges  
• Current design provisions do not address design of shear connectors for precast bridge deck panel systems |
| | **Concrete with Conventional Steel** | • Economical  
• Accelerated construction time  
• Ease of construction  
• High torsional stiffness  
• Reports of good performance after 50 years in-service | • Reflective cracking in longitudinal connection detail  
Note: VDOT study developed new connection detail with good performance (2014) |
| | **Voide Slab Bridges** | • Economical  
• Accelerated construction time  
• Ease of construction  
• High torsional stiffness  
• Reports of good performance after 50 years in-service | • Reflective cracking in longitudinal connection detail  
Note: VDOT study developed new connection detail with good performance (2014) |
| | **Ultra-High Performance Concrete Deck Panel** | • Superior durability against:  
  o Chlorides  
  o Freeze-thaw effects  
  o Salt scaling  
  o Abrasion  
  o Fatigue  
  o Overload  
• 30%-40% lighter than comparable precast FDDP | • Lack of in-service performance information |
| | **Concrete with Fiber Reinforced Polymer Rebar** | • Corrosion-free cables  
• Similar performance as conventional reinforced concrete under service limit state | • Less ductile failure at ultimate limit state  
• Less literature than other systems |
| | **Concrete with Fiber Reinforced Polymer Rebar** | • Expedited construction  
• Enhanced safety and quality controls  
• Reduced on-site labor  
• Less risk of corrosion-induced deterioration | • FRP bars are brittle—may not be possible to bend at precast plant  
• Aggressive weathering durability needs further research  
• Deformability of panels under ultimate load is design concern due to nonductile nature of FRP bars |
| | **Glulam Timber** | • Economical | • Shrinkage of wet timber decreases transverse prestress allowing slip among adjacent beams |
| | **Advanced Composites** | • Potential lower life-cycle cost  
• Strength  
• Stiffness, transportation  
• Ease of construction  
• Environmental durability  
• Preliminary service environment tests indicate no structural problems and performing well in-service | • Data not available for long-term in-service performance  
• High initial cost  
• No local suppliers  
• Connection details  
• Availability of design codes and methodologies |
| | **Advanced Composites** | • Non-corrosive  
• Light  
• Environmentally friendly | • Very few manufacturers  
• Lack of design information  
• Expensive  
• Span limited to 25 ft |
3. Proposed Prefabricated Bridge Element Systems

The research team proposed three alternatives for double tee deck systems based on the findings of the literature review as well as the input from the SDDOT technical panel. This chapter includes more information regarding these alternatives.

3.1 Full-depth Deck Panels (FDDP) Supported on Inverted Bulb-Tee Girders

The main components of a FDDP system typically include: precast full-depth concrete deck slabs, prestressed concrete or steel girders, transverse joints, a longitudinal joint, shear pockets, and horizontal shear reinforcement. These components are discussed herein.

3.1.1 Transverse Joints

Different detailings have been developed for transverse joints of FDDP systems. These detailings usually incorporate longitudinal post-tensioning to aid in moment and shear transfer and to prohibit reflective cracking. However, post-tensioning was not preferred in the present study since many local counties may not have the technology and skilled labor to utilize post-tensioning.

It is also common to splice the longitudinal steel reinforcement of precast deck panels to avoid post-tensioning. Badies and Tadros (2008) reported that some highway
agencies (e.g. the Alaska DOT and the New Hampshire DOT) did not use any reinforcement crossing the transverse joint (Fig. 3.1). The Alaska DOT has not reported any significant joint cracking or leakage on simply supported bridges on low-volume traffic roads when there was no transverse joint reinforcement.

Figure 3.1- Transverse Joint Detail of Bridges Used by Alaska DOT (Badies and Tadros, 2008)

3.1.1.1 Shear Key Types

Various shear key details exist for FDDP systems (Fig. 3.2). Shear keys transfer both shear forces and bending moments. The shear force transfer is achieved through a combination of bearing against the concrete-grout surfaces and bond between the concrete-grout surfaces.
Two methods have been used to contain the grout poured into the shear keyways: inserting a polyethylene backer rod towards the bottom of the keyway, and using wood formwork placed from under the panel. Badie and Tadros (2008) recommended roughening the surface of the shear key for deck systems that do not include post-tensioning.

3.1.1.2 Block-out with Tied-in Lap Splice and Spiral Confinement

Figure 3.3 shows the transverse joint detail that consists of a series of block-outs along the joint. Bridge deck longitudinal reinforcement extends from panels into the block-outs and a steel bar is tied to the deck longitudinal reinforcement. Steel spirals are used to confine the concrete and shorten the lap splice length by 40% to 50% and to simplify the construction since deck steel does not extend into the transverse joint (Badie and Tadros, 2008).
3.1.1.3 Hollow Structural Steel Confinement

Badies and Tadros (2008) developed two new FDDP transverse joints with external confinement (Fig. 3.4 and 3.5). Hollow structural steel (HSS) tubes are embedded in the FDDP decks adjacent to the transverse joint. Figure 3.4 shows the first joint detail. Deck steel bars extend out the transverse joint on one side of the slab and are inserted into the HSS tube in the adjacent slab during construction.

Figure 3.4- HSS Tube Confinement Detail for Transverse Joint

Figure 3.5 shows the second joint detail. HSS tubes are embedded in both adjacent panel transverse joints. Deck steel bars extend into the HSS tubes. The main
difference with respect to the first detailing is that the top portion of the HSS tube is open to allow placement of deck steel bars in the HSS tubes. It should be noted that these types of joints have a tight construction tolerance.

![Diagram of HSS Tube Confinement Detail](image)

(a) HSS Tube Confinement Detail (Badie and Tadros, 2008)  
(b) Galvanized Bulged HSS 4x12x3/8" (Badie and Tadros, 2008)

**Figure 3.5- HSS Tube Confinement Detail for Transverse Joint**

### 3.1.1.4 UHPC for Transverse Joints

Graybeal (2010) tested various transverse joint details incorporating UHPC as joint filler. One detail consisted of non-contact headed mild-steel reinforcement extending from the bridge deck into the joints. Two No. 5 bars were placed along the length of the connection between the heads. Figure 3.6 shows the layout and rebar plan of the connection. Another connection consisted of epoxy-coated No. 4 hairpin bars extending from the deck into the joint (Fig. 3.7). Two No. 5 bars were placed inside the hairpins along the length of the joint. The third detail consisted of straight lapped No. 5 mild-steel reinforcement extending from the deck into the joint (Fig. 3.8). Two No. 5 bars were placed along the length of the connection between the top and bottom layer of joint reinforcement.
No debonding between the joint-panel interface occurred during cyclic loading. Also, no rebar debonding occurred in the joints of the test specimens. Cracks propagated perpendicular to the transverse joints when subjected to ultimate loading. All of the details tested by Graybeal (2010) are expected to perform acceptably in the field.
Figure 3.6 - Layout and Rebar Plan with UHPC and Headed Mild-Steel Reinforcement (Graybeal, 2010)
Figure 3.7- Layout and Rebar Plan with UHPC and Hairpin Mild-Steel Reinforcement (Graybeal, 2010)
Figure 3.8- Layout and Rebar Plan with UHPC and Straight Lapped Mild-Steel Reinforcement (Graybeal, 2010)
3.1.2 Longitudinal Joint

For FDDP systems, a longitudinal joint is located in the center of the bridge deck in the direction of traffic that enables the bridge to be crowned for water drainage. Typically, transverse steel U-shape bars extend from adjacent panels to splice the panels and to provide reinforcement continuity to resist bending and shear forces. Steel bars are installed in the length of the longitudinal joint inside the U-bars to increase the bond strength. Figure 3.9 shows a longitudinal joint detail used on Bill Emerson Bridge in Missouri.

![Figure 3.9](image)

(a) Photo. Longitudinal Joint Detail.  (b) Illustration. Longitudinal Joint Detail.

**Figure 3.9- Longitudinal Joint Detail of Bill Emerson Memorial Bridge, Missouri DOT (Bill Emerson Memorial Bridge, 2003)**

A UHPC Waffle Deck Panel system was designed with a longitudinal joint with 1-in. diameter straight dowel bars extending from the deck into the joint and rebar running the length of the joint to aid in developing the dowel bars (Aaleti and Sritharan, 2014). Figure 3.10 shows the longitudinal joint detail of the UHPC Waffle Deck Panel system.
3.1.3 Shear Pockets

The shear pockets connect the concrete panels to the girder to create composite action. Scholz (2007) performed a study on shear pocket connections funded by the Virginia DOT. Eight various grout types were investigated to determine the optimum grout. This study also investigated the strength of the two shear planes at the girder-grout and grout-deck interfaces. Each of the eight candidate grouts was tested according to ASTM procedures for properties. These properties included flow and workability, horizontal shear strength with two planes of shear, various shear pocket reinforcement types, grout compressive and tensile strength, shrinkage, and adhesion strength between the grout-concrete interface. Four neat grouts and four grouts with a pea gravel extension were tested to develop recommendations for grouts. Furthermore, inverted U-bar stirrups and headed shear studs were tested through push-off tests.
Two grouts were found to be suitable for use in a FDDP system based on this research: Five Star® Highway Patch and Set® 45 Hot Weather. Two types of shear reinforcement between the precast concrete I-beams and bridge deck panels were tested and provided adequate shear resistance. These included two No. 4 or No. 5 bars extending from the I-beam into the shear pocket and headed shear studs, which were welded to steel plates embedded in the I-beams.

Badie et al. (2006) developed two types of shear pockets that can be used in FDDP systems: partial-depth and full-depth shear pockets (Fig. 3.11 and 3.12). The partial-depth shear pocket was recommended when no overlay is used to protect the deck from water leakage at the grout and surrounding concrete interface.

(a) Partial-depth Shear Pocket  (b) Full-depth Shear Pocket

Figure 3.11- Shear Pocket Details (Badie et al., 2006)
3.1.4 Horizontal Shear Reinforcement

Two types of reinforcement detailing were used in the past to transfer horizontal shear forces between the girder and the deck: inverted U-bar and headed shear studs (Fig. 3.13 to 3.15). The U-bars placed transversely minimize the length of shear pockets and the U-bars placed longitudinally can be used in girders with small web widths. The headed shear stud detail (Fig. 3.15) proposed by Badie and Tadros (2008) requires welding of shear studs to a steel plate and embedding the plate in the top flange of the prestressed concrete girder.
3.1.5 Summary of Details for FDDP Test Model

Based on the recommendation by Badie and Tadros (2008), 10-ft long panels were selected for the present study since the test bridge is 50-ft long to replicate the typical length of local highway bridges in South Dakota. The panels are approximately 9.5-ft wide. The panel width was selected to fit inside the 10-ft wide steel loading frame and to allow for testing of two typical interior girders of the FDDP bridge system. Two 50-ft inverted bulb-tee girders were spaced at 4’-8” on center to replicate the expected spacing of seven girders on a 34’-6” wide bridge.
3.2 Voided Slabs with Revised Longitudinal Joint

The main components of a voided slab bridge are precast voided slab girders, longitudinal joints with a partial-depth grouted shear key, and transverse post-tensioning. Some DOTs have used voided slab bridges without transverse post-tensioning on low-volume roads. However, cracking at the longitudinal joint has been reported in many voided slab bridges (Joyce 2014). Figure 3.16 shows a cross-section of a voided slab with a partial-depth shear key.

Joyce (2014) developed a new longitudinal joint that eliminated cracking (Fig. 3.17). The detail utilized VHPC with block-outs and lap-spliced reinforcement, which extends from the girders into the block-outs. The detail developed by Joyce (2014) has the potential to eliminate the need for transverse post-tensioning of the voided slab system.
3.2.1 Practical Lengths of Voided Slab Bridges

According to the California Department of Transportation (2012), the possible span length for voided slab bridges is 20 to 70 ft (6.1 to 21.3 m) with a preferred length of 20 to 50 ft (6.1 to 15.2 m). Furthermore, the Idaho DOT published a design chart for span length of voided slab bridges based on different slab depths (Fig. 3.18).
3.2.2 Summary of Details for Voided Slab Test Model

Based on preliminary design tables by VDOT (2005), two 50-ft long (15.2 m), 21-in. (0.533 m) deep, and 4-ft (1.22 m) wide voided slabs are proposed. This section geometry was listed in the VDOT (2005) preliminary design tables for a 32-ft roadway width. The proposed longitudinal joint detail consists of block-outs with tied-in lap-spliced rebar spaced at 2-ft (0.61 m) along the length of the longitudinal joint. The proposed longitudinal joint filler material is non-proprietary VHPC or UHPC.

3.3 Treated Glulam Composite Timber Bridges

Glulam bridges are constructed of glulam beams manufactured from lumber laminations that are bonded together on their wide faces with waterproof structural adhesives. Glulam is the most common timber bridge material because they are virtually
unlimited in depth, width, and length and can be manufactured in a wide range of shapes. Glulam bridges are most commonly used for spans of 20 to 80 feet, but can be used for clear spans over 140 feet. Long-term performance, wearing surfaces, maintenance, epoxy properties, railing systems, abutments, fabrication and construction, and the inspection of glulam timber bridges are discussed herein.

3.3.1 Types of Glulam Timber Bridges

There are two types of glulam timber bridges: longitudinal glulam decks and transverse glulam decks (Fig. 3.19). A longitudinal glulam deck bridge consists of glulam planks placed longitudinally supported by transverse stiffeners. They can only span up to 38 feet (Wacker and Smith, 2001). A transverse glulam deck bridge consists of glulam panels placed transversely supported by stringers and diaphragms. These bridges can span up to 80 feet.

(a) Longitudinal Glulam Deck  
(b) Transverse Glulam Deck

Figure 3.19- Glulam Bridges (Wacker and Smith, 2001)
3.3.2 Long Term Performance of Glulam Timber Bridges

Wood has been used as a bridge material for hundreds of years, but untreated timber was used primarily until the early 1900s. Many of these untreated timber bridges performed well, but their use has declined since naturally resistant North American wood species are no longer available in the size and quantity needed for construction. Furthermore, it is no longer economical to cover the timber bridges for protection (Ritter 1990).

Brashaw et al. (2013) investigated the long-term performance of five glulam bridges (Table 3.1) located in southern Minnesota. The National Bridge Inventory (NBI) ratings as well as the rating system for these glulam bridges are presented in Tables 3.2 and 3.3. It was concluded that if a glulam bridge is properly maintained, the bridge can last more than 60 years.

Table 3.1: Glulam Timber Stringer Bridges located in Minnesota (Brashaw et al., 2013)

<table>
<thead>
<tr>
<th>Bridge ID</th>
<th>Year Built</th>
<th>Span (ft)</th>
<th>Average Daily Traffic</th>
<th>Width (ft)</th>
<th>Wearing Surface</th>
</tr>
</thead>
<tbody>
<tr>
<td>22508</td>
<td>1968</td>
<td>33.5</td>
<td>95</td>
<td>33.3</td>
<td>Bituminous</td>
</tr>
<tr>
<td>22514</td>
<td>1968</td>
<td>40</td>
<td>35</td>
<td>26</td>
<td>Gravel</td>
</tr>
<tr>
<td>22518</td>
<td>1969</td>
<td>38.5</td>
<td>70</td>
<td>33.1</td>
<td>Gravel</td>
</tr>
<tr>
<td>22519</td>
<td>1969</td>
<td>33.5</td>
<td>539</td>
<td>32</td>
<td>Bituminous</td>
</tr>
<tr>
<td>9967</td>
<td>1951</td>
<td>36.2</td>
<td>175</td>
<td>27.4</td>
<td>Bituminous</td>
</tr>
</tbody>
</table>

Table 3.2: NBI Condition Rating (Brashaw et al., 2013)

<table>
<thead>
<tr>
<th>NBI Condition Rating</th>
<th>Bridge Number</th>
<th>Group Mean</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>22508 22514 22518 22519 9967</td>
<td></td>
</tr>
<tr>
<td>Deck</td>
<td>7 6 7 6 7</td>
<td>6.6</td>
</tr>
<tr>
<td>Superstructure</td>
<td>7 7 7 7 6</td>
<td>6.8</td>
</tr>
</tbody>
</table>
Table 3.3: NBI Condition Rating System

<table>
<thead>
<tr>
<th>FHWA - SI&amp;A Sheet Condition Rating Code</th>
<th>FHWA- SI &amp; A Sheet Condition Rating Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>N</td>
<td>Not Applicable</td>
</tr>
<tr>
<td>9</td>
<td>Excellent Condition - New or like new condition.</td>
</tr>
<tr>
<td>8</td>
<td>Very Good Condition - No problems noted.</td>
</tr>
<tr>
<td>7</td>
<td>Good Condition - Some minor problems but no structural defects at critical locations (wood decay is a defect).</td>
</tr>
<tr>
<td>6</td>
<td>Satisfactory Condition - Structural elements show some minor defects and/or deterioration at critical locations. No measurable section loss.</td>
</tr>
<tr>
<td>5</td>
<td>Fair Condition - All primary structural elements are sound but may have minor to moderate defects and/or deterioration with measurable section loss at critical locations. No significant reduction in primary structural member load carrying capacity.</td>
</tr>
<tr>
<td>4</td>
<td>Poor Condition - Primary structural elements show moderate to serious defects, deterioration, corrosion, cracking, crushing, and/or scour. Advanced section loss at critical locations. Diminished load carrying capacity of members is evident.</td>
</tr>
<tr>
<td>3</td>
<td>Serious Condition - Serious and widespread defects have substantially reduced load carrying capacity of primary structural members. Local failures may be evident. Deflection/misalignment of members may be evident. Signs of severe structural stress are visible. Fatigue cracks in steel, shear cracks in concrete, and severe decay, checking, splitting, and crushing of beams or stringers in wood elements may be present.</td>
</tr>
<tr>
<td>2</td>
<td>Critical Condition - Advanced deterioration of primary structural elements. Defects have now resulted in significant local failures. Scour may have removed substructure support. Unless closely monitored it may be necessary to close the bridge until corrective action is taken.</td>
</tr>
<tr>
<td>1</td>
<td>Imminent Failure Condition - Major deterioration or section loss present in critical structural components and/or obvious vertical or horizontal movements affecting structure stability. Bridge is/should be closed. However, corrective action may put bridge back in light service.</td>
</tr>
<tr>
<td>0</td>
<td>Failed Condition- Out of service. Beyond corrective action.</td>
</tr>
</tbody>
</table>

3.3.3 Wearing Surfaces for Timber Decks

A wearing surface is a layer placed on the bridge deck to form the roadway surface. According to Ritter (1990), the main purpose of the wearing surface is to improve safety, provide a smoother surface, improve skid resistance, and protect the deck. Typically, a wearing surface of a timber bridge can consist of (1) an asphalt overlay, (2) an asphalt chip seal, (3) sacrificial lumber covering the whole deck, (4) cover steel plates, (5) cover lumber planks, and (6) aggregate overlay. In the case that no
wearing surface is used, routine inspections are required to ensure that the deck remains properly sealed.

Asphalt is the most commonly used overlay since it provides a smooth but skid-resistant surface, while providing a water-proof layer that protects the timber deck from abrasion. The only drawback of using asphalt is the reflective cracking that can allow water to seep into the wood. Geotextile fabrics are encouraged with this method to prevent the reflective cracking and to improve the bond between the glulam and the asphalt. The asphalt also must be well-maintained to prevent moisture from reaching the deck. When paving, it is important that the approaches are paved a minimum of 75 feet beyond the bridge ends to prevent the potholes that commonly form at the ends.

Asphalt chip seal has also been recommended by Ritter (1990). The asphalt chip seal consists of liquid asphalt covered with a layer of aggregate. They are comparable to an asphalt overlay in a way that they are smooth and skid-resistant. The chip seal is thinner and more flexible than an asphalt overlay resulting in less cracking. A double treatment of layers approximately ¾-inch thick was recommended to insure the sealing of the deck. A geotextile fabric was also recommended with this method.

The application of an aggregate overlay is scarce. A 3-in gravel overlay was used over an epoxy-flooded deck of a timber bridge in Buchanan County, Iowa.

The remaining overlays were not recommended by Ritter (1990) as they can trap water. These methods were typically used on low volume roads.
3.3.4 Maintenance and Inspection Required for Glulam Timber Bridges

Routine maintenance, which varies based on the wearing surface, is required to minimize the moisture content since dry wood lasts longer. It was recommended that timber bridges be inspected every 2 years and any exposed wood to be retreated every 6 years (Ritter, 1990).

A bridge inspector can use several methods including visual inspection, probing, and sounding to inspect the bridge. If decay is suspected, the inspector then must drill or core the area for further inspection. If decay is found, a retrofit plan is needed.

Preventative maintenance such as resealing exposed wood is performed when decay or deterioration has not started. Remedial maintenance is performed when decay or deterioration is present but it does not affect the performance of the bridge. This includes replacing small sections of the bridge. Major maintenance is performed when deterioration has reached a point where strength loss has occurred. This also includes replacing sections of the bridge to return the bridge to its original load-carrying capacity.

The epoxy is typically applied in three layers with an approximately 3/8-in. thickness. The life of the epoxy depends on its exposure. However, it is expected that epoxy lasts for a long time (e.g. the life of the bridge) should a wearing surface be maintained.

3.3.5 Railing System

The vehicular railing must be positioned to safely contain an impacting vehicle without allowing it to pass over, under, or through the rail elements. It also must be free of features that may catch on the vehicle or cause it to overturn or decelerate too rapidly.
Any railing configuration can be used for timber bridges as long as it complies with the minimum criteria specified by AASHTO or it has been verified by full-scale crash testing. The rail material can be timber, metal, or concrete. One example of a timber railing is shown in Fig. 3.20.

![Figure 3.20- Railing on a Glulam Bridge (laminatedconcepts.com)](image)

### 3.3.6 Timber Bridge Abutments

Many studies stated that existing abutment detailing can be used for glulam timber bridges. Timber bridge abutments can be made of timber or concrete (Fig. 3.21). It is clear that the connections should be designed to resist appropriate design loads.
3.3.7 Timber Bridge Fabrication

Glulam timber bridges can be completely prefabricated offsite then shipped to the project site for assembly, which accelerates construction (Fig. 3.22). Assembly is typically started with the center stringer working outwards. Subsequently, the deck panels are placed. The curbs and railings are then installed. Finally, the substructure backwalls are placed and the approach can be backfilled. The whole construction process for a 60-ft bridge can be completed in 60 hours (Ritter 1990).
3.3.8 Summary of Details for Timber Bridge Test Model

The glulam timber bridge test model to be designed will be approximately 50-feet (15.2-m) long and 9-feet (2.7-m) wide. The glulam will consist of southern pine. The bridge will be supported by three stringers approximately 6.75-inch wide by 38-inch deep, each connected by four diaphragms. Eleven interior 4-ft wide deck panels will be placed transversely with two additional end panels each 3-ft wide. It will be assumed that a 3-in. asphalt wearing surface will be used.

3.4 Proposed Bridge System Cost Estimates

Table 3.4 presents cost estimate information for the three proposed alternatives to double-tee bridges. Preliminary cost estimates of the three proposed bridge systems were
developed based on information provided by South Dakota manufactures and contractors.

The items considered in the cost estimation were the cost of the superstructure materials, superstructure fabrication, and superstructure construction. Note that the cost estimate does not include the cost of foundation, foundation construction, mobilization, and railings.

<table>
<thead>
<tr>
<th>Bridge System</th>
<th>Timber ($K)</th>
<th>Full-Depth Deck Panels ($K)</th>
<th>Voided Slab ($K)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Materials/Fabrication</td>
<td>86</td>
<td>88</td>
<td>94</td>
</tr>
<tr>
<td>Construction ($K)</td>
<td>19 - 30</td>
<td>47 - 63</td>
<td>45 - 60</td>
</tr>
<tr>
<td>Total ($/sq. ft.)</td>
<td>61 - 67</td>
<td>78 - 88</td>
<td>81 - 89</td>
</tr>
</tbody>
</table>

Note: Preliminary estimates provided by Gage Brothers and Journey Construction of Sioux Falls.
4. Precast Full-Depth Deck Panel Bridge Specimen

The structural performance of a full-scale bridge specimen incorporating precast full-depth deck panels supported on inverted bulb-tee girders was experimentally evaluated under fatigue and ultimate loading. This chapter includes design, fabrication, instrumentation, test setup, and test procedures for the test specimen.

4.1 Design of Bridge Test Specimen

Bridges on South Dakota local roads usually consist of two lanes and two shoulders with a total width of 34.5 ft. The prototype single-span bridge was assumed to be 50-ft long and 34.5-ft wide as shown in Fig. 4.1. The proposed fully precast deck system incorporates precast full-depth panels connected to prestressed inverted bulb-tee girders through pockets. For a 50-ft long bridge, five 8-in. deep precast full-depth deck panels supported on seven 21-in. deep prestressed inverted bulb-tee girders are needed based on a preliminary design.

Precast panels can be built either as a single unit with a single grade in the transverse direction (Fig. 4.2a) or two units in the transverse direction as shown in Fig. 4.2b. The proposed bridge with single-unit panels (Fig. 4.2a) offers minimal onsite activities and thus accelerates the construction.
Figure 4.1 - Plan View of Prototype Bridge

(a) Single-Unit Panel

(b) Two-Unit Panels with a Longitudinal Joint

Figure 4.2 - Cross-Section of Fully-Precast Proposed Prototype Bridge
A full-scale bridge model was selected for laboratory testing. The bridge test specimen was designed for HL-93 loading according to the American Association of State Highway and Transportation Officials LRFD Bridge Design Specifications (AASHTO, 2013), which includes both the design truck (Fig. 4.3) or tandem load as well as the design lane load. The design truck load consists of two 32-kip axle loads (one rear truck axle and one trailer axle) spaced 14 ft apart and an 8-kip front axle load spaced 14 ft in the front of the rear truck axle. The design lane load was a 0.64-kip-per-foot distributed load applied along the length of the bridge over a 10-ft width.

Figure 4.3- HL-93 Design Truck (AASHTO, 2013)

The bridge model was analyzed and designed according to the AASHTO (2013). Appendix A presents a summary of shear and bending moment envelopes. Because of test setup limitations, only a 10-ft wide bridge could be tested in the Lohr Structures laboratory at South Dakota State University (SDSU). Therefore, two interior girders of the prototype bridge were selected for testing. The full-scale bridge test model consisted of five precast full-depth deck panels with a 9.5-ft width (in the bridge transverse direction) and a 10-ft length (in the bridge longitudinal direction) and two 50-ft long prestressed inverted bulb-tee girders spaced 4.67 ft on center. Shop drawings for the test specimen are provided in Appendix C.
The main objectives of the laboratory tests were to assess the bridge system performance under fatigue and strength loading. In-depth discussion of the design and detailing of the girders and the panels are presented herein.

4.1.1 Inverted Bulb-tee Girders

The moment and shear demands transferred to each girder was determined using the AASHTO (2013) live load distribution factors in which the moment live load distribution factor for the interior girders was 0.46 for both the Strength I and Fatigue limit states and the shear live load distribution factor was 0.57 for the Strength I limit state. The complete calculations for the live load distribution factors are provided in Appendix B.1 and shear and moment envelopes for an interior girder are provided in Appendices A.

A software, PS Beam (Ericksson Technologies, 2011), was used to design the prestressed inverted bulb-tee girders according to AASHTO (2013). A total of 20 grade 270 low relaxation prestressing strands with a diameter of 0.6 in. were used in the inverted bulb-tee girders to meet the design requirements. Of which, two strands were harped to avoid concrete cracking at the girder ends. Figure 4.4 shows the cross-sections of the bridge test girders at the mid-span and the girder end. Figure 4.5 shows the tendon profile for the inverted bulb-tee girders. The girders were transversely reinforced with ASTM A615 Grade 60 No. 4 bars at a spacing varying from 0.5 ft to 1.5 ft.
4.1.2 Full-depth Deck Panels

The full-depth deck panel top and bottom reinforcement in the transverse direction of the bridge was designed using table A4-1 in AASHTO (2013), which provides maximum live load moments per unit width for both positive and negative transverse deck moments. The tabulated values are based on the equivalent strip method. The deck longitudinal reinforcement was designed to accommodate creep and shrinkage requirements and to allow splicing of reinforcement at transverse joints to provide adequate shear and moment transfer between the transverse joints. The deck longitudinal
steel was placed in one layer at 4-1/4” below the deck surface to allow for splicing of the steel at the transverse joints. The complete detailing of the test bridge specimen is provided in Appendix C.

An unfactored positive live load transverse moment of 4.63 kip-ft/ft was used to design the deck bottom layer of transverse reinforcement assuming a 4.75-ft girder spacing (the actual girder spacing was 4.67 ft). The unfactored negative live load transverse deck moment used to design the top layer of transverse reinforcement was 2.90 k-ft/ft. The live load transverse deck moments were then multiplied by a factor of 1.75 associated with Strength I Limit State. Dead load positive and negative transverse moments were multiplied by a factor of 1.25 for the Strength I Limit State and were added to the live load design moments.

4.1.2.1 Shear Pockets

The precast girders and panels were connected using shear studs extending from the girder top flange into panel shear pockets to make the deck system composite (Fig. 4.6a and Fig. 4.7a). The deck system will be composite since horizontal shear stresses are transferred through the bond in the haunch region as well as the shear studs when the grout is cured.
Two types of shear pockets were incorporated in the test bridge (Fig. 4.8): (1) full-depth pocket in which the full-depth of the deck was open, and (2) hidden pocket in which the large portion of the pocket was coved with 3 in. of concrete. Grout can be poured from the top of the pockets into the full-depth pocket or through pipes in the hidden pocket. The location of the pockets on the plan view of the test bridge is shown in Fig. 4.9.
4.1.2.2 Horizontal Shear Studs

Two types of horizontal shear studs were incorporated in this study: (1) inverted U-shape bars in the full-depth pockets, and (2) double headed studs in the hidden pockets. ASTM A615 Grade 60 No. 4 bars were used to form the inverted U-bar studs. Each full-depth pocket (Fig. 4.8c and 4.8d) contained eight legs of No. 4 inverted U-bars and was
spaced at 2-ft on center. The double headed studs were made of ASTM A615 Grade 60 No. 5 bars. Eight double headed studs were used in the hidden pockets (Fig. 4.8a and 4.8b) and pockets were spaced at 2-ft on center.

Horizontal shear studs were designed based on AASHTO (2013) equation 5.8.4.1-3:

\[ V_{ni} = cA_{cv} + \mu(A_{vf}f_y + P_c) \leq \min(0.2f'_{ce}A_{cv}, 0.8A_{cv}) \]  

Eq. 4-1

where:

- \( V_{ni} \) = the nominal shear resistance of the interface plane,
- \( c \) = the cohesion factor (ksi),
- \( A_{cv} \) = the area of concrete considered to be engaged in interface shear transfer (in.\(^2\)),
- \( \mu \) = the friction factor,
- \( A_{vf} \) = area of interface shear reinforcement crossing the shear plane within the area \( A_{cv} \) (in.\(^2\)),
- \( f_y \) = yield stress of reinforcement but design value not to exceed 60 (ksi),
- \( P_c \) = permanent net compressive force normal to the shear plane, \( P_c = 0 \) (kip).

The cohesion factor and the yield strength were assumed to be 0.075 and 60 ksi, respectively. The friction factor was 0.6 and the \( P_c \) was assumed to be zero. The shear demand was calculated based on the average maximum shear force of a 10-ft length using the Strength I limit state shear envelope, starting at the support of the bridge (Appendix A.1). Since, the stud shear force for the exterior girders were higher than that for the interior girders, the larger force was used for the design of studs on both interior and exterior girders. The factored horizontal shear demand is:
\[ V_h = \frac{V_u}{d_v} \quad \text{Eq. 4-2} \]

where:

\( V_h \) = horizontal factored shear force per unit length of the beam (kips/in.),

\( V_u \) = factored shear force at a specified section due to superimposed loads (kips) = 90.4 kips,

\( d_v \) = the distance between resultants of tensile and compressive forces, (de-a/2) = 24.0 in.

### 4.1.2.3 Transverse Joint

The full-depth deck panel (FDDP) transverse joints consisted of (a) female-to-female grouted shear keys (Fig. 4.10) in the transverse direction of the bridge and (2) dowel bars in the bridge longitudinal direction to be embedded in hollow structural steel (HSS) members (Fig. 4.11). The gap between the two adjacent precast decks in the bridge longitudinal direction is usually 1 to 1.5 in. for a typical FDDP transverse joint. However, a 2.75-in. wide transverse joint was used to allow a transverse steel bar to be placed in the joint to meet maximum rebar spacing requirements of 18 in. and to allow 1 in. of clear cover from the face of the joint. Two No. 5 bars were placed beneath HSS sections to meet creep and shrinkage requirements (Fig. 4.11b).
Figure 4.10- Female-to-Female Transverse Joint Detailing

Figure 4.11- Transverse Joint Detailing for Full-Depth Precast Panels
The transverse joints were reinforced with 26.25-in. No. 6 ASTM A615 Grade 60 dowels, which were inserted into HSS from the top of the deck after the panels were placed. ASTM A500 Grade B steel was used to form HSS (Fig. 4.11). HSS will increase the confinement resulting in a shorter lap-splice for dowels.

Two types of failure (Fig. 4.12) can be assumed for the proposed transverse joint: (1) bearing and (2) vertical shear. Modified shear friction theory was used to check the strength of the transverse joints with the longitudinal dowels (App. B in Badie and Tadros, 2008).

![Figure 4.12- Transverse Joint Failure Modes (Badie and Tadros, 2008)](image)

4.2 Fabrication and Assembly of Test Specimen

The girders and panels for the test bridge specimen were fabricated at the Gage Brothers Concrete Products facility in Sioux Falls, South Dakota. This section includes the fabrication of bridge members and construction stages for the bridge test specimen.
4.2.1 Inverted Bulb-Tee Girders

The inverted bulb-tee girders were prepared and cast on a single bed (Fig. 4.13). Low relaxation Grade 270 prestressing strands were initially tensioned to 10,000 lbs to eliminate slacks and to straighten tendons for instrumentation (see the “Strain Gauges” section of the chapter). Then, the girder shear reinforcement as well as shear studs (to be inserted into the deck pockets) were installed. Strain gauge data from strands were obtained before tensioning. Finally, each strand was tensioned to 44,000 lbs, which is equivalent to 75% of the its ultimate stress. Strain gauge readings were also taken during jacking.

The girders were cast in two consecutive days. The one-day strengths of the first and the second girders were 6,820 and 6,190 psi, respectively. Since the specified concrete strength at the time of tendon release was 6,000 psi, the strands were concurrently cut one-day after casting. Strains were also measured during the tendon release.
The test girders were shipped to the Lohr Structures Laboratory at South Dakota State University after releasing the tendons. The girders were unloaded using a 15-ton overhead crane and placed on concrete reaction blocks (abutments). Figure 4.14 shows the girder unloading and installation sequences.
The elevations of each girder top flange at mid-span and at the girder ends were surveyed. The data was used to determine the girder individual and differential cambers. The camber of the west (Girder A) and the east (Girder B) girders were respectively 2.0 and 2.5 in. before panel installation. The 0.5-in. differential camber may be attributed to the one-day difference in casting of the two girders on the same prestressing bed.
4.2.2 Full-Depth Deck Panels

Five precast panels were fabricated in an indoor construction site (Fig. 4.15). Three interior panels were 9.5-ft wide in the transverse direction and 9.77-ft long in the longitudinal direction of the bridge and two exterior panels had the same width, but were 9.89-ft long. Each of the five panels contained 10 pockets. Three panels (C, D, and E in Fig. 4.9) had full-depth pockets (Fig. 4.15b) while the remaining two panels (A and B) had hidden pockets (Fig. 4.15c). The hidden pocket forms were constructed using plywood for the pockets and polyvinyl chloride pipes were installed to form the grouting pipe and vents. The full-depth pocket forms were constructed using cut-out hardboard insulation in stacked layers. Four leveling bolts were placed in each panel. Leveling bolt forms consisted of a nut tack-welded to a vertical steel pipe, and a 2 by 4-in. lumber piece to form a blockout at the top of the steel pipe (Fig. 4.15d).
Figure 4.15- Fabrication of the Full-depth Deck Panels

(a) Panel Formwork
(b) Full-Depth Pockets
(c) Hidden Pockets
(d) Leveling Bolt
(e) Concrete Casting
The full-depth deck panels were shipped to the Lohr Structures Laboratory and were unloaded using a 15-ton overhead crane. The pockets, joints, and the embedded hollow structural steel members were cleaned to avoid any bond issues (Fig. 4.16). Petroleum jelly was applied to the leveling bolt shaft (Fig. 4.17) at the bottom end to allow bolt removal after pouring the grout in the haunch. Next, the panels were placed (Fig. 4.18) starting from one end of the bridge (the south end) toward the other end (the north end). Then, the leveling bolts (Fig. 4.17) were adjusted with a wrench to level the deck panels. The target grouted haunch depth was 1 in. at the mid-span, which was achieved using the leveling bolts. Figure 4.18 illustrates the panel installation sequences.
Figure 4.16 - Debris Removal from Precast Panel Pockets and Joints
Figure 4.17- Precast Panel Leveling Bolt Details
Figure 4.18 - Installation Sequences for Full-depth Deck Panels
Plywood was attached at the bottom of the transverse joints using tie wires, which were tied to the transverse joint reinforcement. The plywood and tie wires (Fig. 4.19) were installed from the top of the bridge. Then, silicone was applied around the concrete-plywood edges from the top of the bridge to create water-tight joints.

A No. 6 bar was placed and centered on the spliced bars of each transverse joint. The transverse No. 6 bar was tied to the spliced bars in three locations. Figure 4.20 shows the transverse bar after placement in a transverse joint.
Based on the initial construction plan, the grouted haunch region (the space between the girder and the panel above the girder) was confined utilizing non-compressive hardboard insulation foam and 1-in. diameter backer rods (compressive foam) at the top of the hardboard. The hardboard was secured to the girders using a glue (PL300). The same glue was used to install the backer rods to the top surface of hardboards. Since this formwork was not sufficient in the first attempt to pour the haunch, a somewhat different formwork was incorporated.

The modified grouted haunch dam was formed using 0.75-in. thick plywood and 2 by 4-in. lumber as shown in Fig. 4.21. The lumber was used as a strut to hold the plywood in place. For the exterior of the girders, a longitudinal lumber was clamped to the deck and was used as a reaction block for the transverse struts.
(a) Wood Clamped to Deck
(b) Plywood Installed Outside of Girder
(c) Plywood Installed between Girders
(d) Finished Haunch Forms
(e) Girder End Formwork

Figure 4.21- Grouted Haunch Dam Formwork
Two types of filler material were incorporated in the grouted haunch, shear pockets, and transverse joints (Fig. 4.22): (1) conventional non-shrink grout and (2) and latex modified concrete (LMC). Technical data sheets for the two materials are provided in Appendix D. As was discussed before, two types of pockets were used in the test specimen: (a) hidden, and (b) open (full-depth). Since durability of the open pockets was a concern, LMC was proposed as an alternative fill material for this type of pocket because the durability of LMC is better than conventional grout (Baer, 2013; BASF, 2011; Wenzlick, 2006). Half of the open pockets were filled with LMC and the remaining pockets were cast with conventional grout (Fig. 4.22). Figure 4.23 shows pouring of the grout. Figure 4.24 shows the LMC during and after pour. Sections of the bridge were isolated with plywood inserts to separate the two filler materials.
Figure 4.23: Pouring of the Shear Pockets, Haunch, and Transverse Joints
4.3 Test Setup

The test bridge was placed under a vertical loading frame (Fig. 4.25a) in a way that a 146-kip hydraulic actuator was at the center of the bridge at its mid-span. The girders were supported on concrete reaction blocks. A 6 by 6-in. elastomeric bearing pad was placed between the girder and the reaction block. The effective span length of the test bridge was 49.13 ft. Water ponds were formed on the top of the pockets and joints to investigate the integrity of the precast joint detailing during fatigue testing (Fig. 4.26).

As was mentioned before, the bridge was built with two girders and five panels. The west girder was labeled as Girder A and the east girder was labeled as Girder B (Fig. 4.25). The five panels were labeled A to E starting from the south side of the bridge toward the north.

Fatigue testing was performed in two phases: (1) Phase I in which bridge overall performance was investigated, and (2) Phase II in which the performance of the transverse joint was emphasized. In Phase I, a single point-load was applied at the center of the bridge at the mid-span using a 146-kip actuator (Fig. 4.25). The load was applied
to a 10 by 20-in. steel loading plate to simulate the AASHTO (2013) design truck tire bearing area. A 0.5-in. thick layer of plaster was poured beneath the steel loading plate to ensure a level and uniform bearing surface.
Figure 4.25- Phase I Test Setup Detailing
An 8-ft long W12x93 steel spreader beam was utilized in Phase II to spread the load directly to the transverse joints and to maximize the shear transfer from panel to panel (Fig. 4.27). Two 10 by 20-in. steel loading plates were positioned at the ends of the spreader beam and were leveled. The center-to-center distance between the two loading plates was 7.5 ft.
After the completion of the fatigue testing, an ultimate test was conducted using a 328-kip actuator. A W12x93 steel beam was used to spread the load over the girder centerlines at the mid-span to avoid punching shear failure of the deck. Figure 4.28 shows the test setup for the strength test.
4.4 Instrumentation

The test bridge was heavily instrumented with axial strain gauges, shear strain gauges, linear variable differential transducers (LVDTs), and load cells. The instrumentation plan is discussed herein.

4.4.1 Strain Gauges

Three types of strain gauges (Fig. 4.29) were used on different materials: (1) surface-mounted axial strain gauges were used to measure axial strains in mild and prestressing reinforcement, (2) surface-mounted shear strain gauges were used to capture shear strain data on mild steel bars, and (3) embedded concrete strain gauges were used to measure the concrete strain.
At mid-span, five axial strain gauges were mounted to the top surface of the deck longitudinal mild steel bars (Fig. 30f). Two axial strain gauges were installed on the prestressing tendons at the bottom layer of each girder. One embedded concrete strain gauge was installed slightly above the composite bridge section neutral axis per girder. A total of nine axial strain gauges and two embedded concrete strain gauges were used at the mid-span (Fig. 4.30f).

Strain gauges were installed on the studs in four of the shear pockets (Fig. 30a, 30c, 30h, 30j, and 31). Of which, two were hidden pockets with No. 5 double headed studs and filled with non-shrink grout and the other two were full-depth pockets with No. 4 inverted U-shape bars (one filled with non-shrink grout and the other with latex modified concrete). Eight studs/legs were extended into each pocket to resist horizontal shear. Two axial strain gauges were mounted to the pocket corner studs in a diagonal pattern (Fig. 31) and two shear strain gauges were mounted on the opposite two diagonal studs. The combination of one axial and two shear strain gauges enable the measurement of strains in three different directions. Thus, principal strains and stresses can be measured. The ultimate goal of this instrumentation plan was to determine the maximum principal stress on the horizontal shear studs. The axial strain gauges in Fig. 4.30 were
labeled with a suffix “A” and the strain gauges were labeled with a suffix “S”. For example, SG-A-1 indicates an axial strain gauge and SG-S-2&3 indicates a pair of strain gauges installed in two different directions other than axial.

(a) Hidden Pocket with No. 5 Double Headed Studs

(b) Strain Gauge Plan for Section A-A

(c) Hidden Pocket with No. 5 Double Headed Studs

Figure 4.30- Test Bridge Strain Gauge Locations
(d) Strain Gauge Plan for Transverse Joint Transverse No. 6 Bar

(e) Mid-Span Section

(f) Strain Gauge Plan for Section B-B

Figure 4.30- Cont’d
(g) Strain Gauge Plan for Transverse Joint No. 6 Lap-Spliced Bar

(h) Full-depth Pocket with No. 4 Inverted U-Bars

(i) Strain Gauge Plan for Section C-C

Figure 4.30- Cont’d
(j) Full-depth Pocket with No. 4 Inverted U-Bars

(k) Strain Gauge Plan for Section D-D

Figure 4.30- Cont’d
Figure 4.31 - Surface-Mounted Strain Gauges on No. 5 Double Headed Shear Studs
4.4.2 Linear Variable Differential Transformers

A total of 13 linear variable differential transformers (LVDTs) were attached to the test specimen to measure deflections and rotations in various directions as shown in Fig. 4.32.

Vertical deflections (VD in Fig. 4.32) were measured both at the mid-span of the bridge as well as the girder ends using LVDTs. The difference between the girder mid-span and the girder end displacements was reported as actual (net) girder deflections. This was done to account for deformation of the elastomeric bearing pads. Figure 4.33 shows the LVDTs used to measure the net mid-span deflections.
Deck-to-girder slippage was measured using horizontal LVDTs (HD in Fig. 4.32) mounted to the top of the girder as shown in Fig. 4.34. They were mounted at three locations to observe the performance of: (1) full-depth pockets with latex modified concrete and No. 4 inverted U-shape bars, (2) full-depth pockets with non-shrink grout and No. 4 inverted U-shape bars, and (3) hidden pockets with non-shrink grout and No. 5 double headed studs. Each HD LVDT was installed 15 ft away from the mid-span.
Joint rotations were also measured with LVDTs mounted adjacent to the two transverse joints of the middle panel (Panel C) as shown in Fig. 4.35. These LVDTs were labeled as “R”. Each joint had an LVDT mounted horizontally in the longitudinal direction of the bridge on the top and bottom of the deck at the same section. The LVDTs were offset 13 in. from the longitudinal centerline of the bridge to allow ponding of the joint. Figure 4.35 shows the joint rotation LVDT configuration.

![Figure 4.35- LVDTs for Joint Rotation Measurement](image)

The relative vertical deflection across the two transverse joints of the middle panel (Panel C) was measured with a single vertical LVDT mounted adjacent to each joint (Fig. 4.36). These LVDTs were labeled as “JVD” in Fig. 4.32. Similar to the previous measurement, these LVDTs were offset 13 in. from the longitudinal centerline of the bridge to allow ponding of the joint.
4.4.3 Load Cells

Four load cells were placed under the south end of the girders to measure the support reactions (Fig. 4.37). Two load cells were utilized per girder and they were offset 6.25 in. from the girder centerline to enhance overall stability and 6 in. from the girder end to provide sufficient seat length. Steel plates with a dimension of 6 by 6 by 1 in. were placed at the top and the bottom of the load cells to create a level bearing surface (Fig. 4.38). Elastomeric bearing pads were placed on top of the steel plates to allow the girders to freely rotate.
4.4.4 Data Acquisition System

A 128-channel data acquisition device was used, which can read between 10 and 2,048 readings per second. Stiffness and ultimate tests were scanned at a rate of 10 readings per second. For the fatigue testing, intermediate data were recorded at a scan rate of 100 points per second for 30 load cycles at the beginning and the end of the test.
4.5 Test Procedure

The full-scale bridge was tested (Fig. 4.39) under fatigue, stiffness, and ultimate loading (Table 4.1). Fatigue testing was performed by applying cyclic loads either at the mid-span (phase I) or close to the transverse joints (phase II). Stiffness tests, which consisted of applying monotonic point load(s), were performed at an interval of 50,000 cycles to determine the effect of fatigue on the bridge performance and to measure the bridge overall stiffness. The ultimate test was carried out by applying point loads to the girders at the mid-span with a monotonic loading protocol.

<table>
<thead>
<tr>
<th>Test</th>
<th>Test Description</th>
<th>Load Location</th>
<th>Load (kips)</th>
<th>Number of Cycles</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Phase I- Fatigue Test</td>
<td>Mid-Span</td>
<td>Cyclic with amplitude of 27.7</td>
<td>500,000</td>
</tr>
<tr>
<td>2</td>
<td>Phase II- Fatigue Test</td>
<td>Transverse Joints of Middle-Panel</td>
<td>Cyclic with amplitude of 27.7</td>
<td>150,000</td>
</tr>
<tr>
<td>3</td>
<td>Ultimate Load Test</td>
<td>Mid-Span of Girders</td>
<td>Monotonic from zero to 263</td>
<td>-</td>
</tr>
</tbody>
</table>
For Phase I, a 27.7-kip point-load was applied at the center of the bridge at the mid-span at a loading rate of 1 cycle per second. The actuator was controlled by force to ensure that the cyclic load magnitude remained the same even if the bridge stiffness degraded. The magnitude of the point-load was calculated based on the moment.
envelope demand for a typical interior girder of the 50-ft span two-lane prototype bridge (see App. A.3) according to AASHTO (2013).

Since the proposed bridge will be used on local roads in South Dakota, the average daily traffic (ADT) was assumed to be 100 vehicles per day with a 15% truck density (ADDT=15). Therefore, 410,625 trucks would cross the bridge over a 75-year design life. The test bridge was subjected to 500,000 load cycles to account for the possibility of increased truck traffic.

After completion of the Phase I loading, fatigue testing was continued with two point loads adjacent to the transverse joints. The distance between the two point loads was 7.5 ft on center. The same load magnitude as that of Phase I was applied to the beam resulting in a 13.9-kip load at each end of the spreader beam. The load magnitude was determined by matching the girder shear demand in the test girder from the Phase I loading. The test was terminated at 150,000 cycles since no stiffness degradation was observed.

4.5.2 Stiffness Testing

Stiffness tests were performed at the beginning of the testing and then at every 50,000 load cycle increment thereafter. The stiffness load magnitude was 55.4 kips, which was applied monotonically using a displacement-control loading protocol. The load was calculated based on the moment demand on a typical interior girder for the mid-span according to the AASHTO (2013) Fatigue I limit state. Displacements were applied with an interval of 0.01 in. with a speed of 0.007 in./sec.
4.5.3 Strength Testing

A point-load at the mid-span of the bridge was monotonically applied to a beam placed in the transverse direction of the bridge to spread the load to the two girders. The girders were loaded under a displacement-control loading protocol in which displacements were applied with an increment of 0.02 in. and rate of 0.007 in./sec.
5. Experimental Results and Analysis

This chapter includes the results of an experimental study of a full-scale fully precast bridge system detailed in the previous chapter. The measured material properties and the performance of the bridge under fatigue and ultimate loading are discussed herein.

5.1 Material Properties

Different materials were incorporated in different bridge components. Mix design and mechanical properties for (1) concrete used in the deck, (2) concrete used in the girders, (3) conventional non-shrink grout used in the joints, (4) latex modified concrete used in the joints, (5) deck mild steel, (6) inverted U-shape shear studs, (7) double headed shear studs, and (8) prestressing strands used in the girders are presented in this section.

5.1.1 Mix Design

The design concrete compressive strength for the full-depth deck panels and the prestressed inverted bulb-tee girders at 28 days was 6,000 psi and 8,000 psi, respectively. The concrete mix design for the deck panels and girders are presented in Appendix F.
5.1.2 Properties of Concrete

The properties of the fresh concrete used in the full-depth deck panels and inverted bulb-tee girders were measured in accordance with ASTM C143 and C231 standards (2010) and are summarized in Table 5.1.

<table>
<thead>
<tr>
<th>Component</th>
<th>Temperature (° F)</th>
<th>Air Content (%)</th>
<th>Unit Weight (lb/ft³)</th>
<th>Slump (in.)</th>
<th>Spread (in.)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Deck Panels</td>
<td>72</td>
<td>6.2</td>
<td>142.6</td>
<td>9</td>
<td>NA</td>
</tr>
<tr>
<td>Girder A</td>
<td>82</td>
<td>6.0</td>
<td>143.2</td>
<td>NA</td>
<td>19.5</td>
</tr>
<tr>
<td>Girder B</td>
<td>80</td>
<td>4.1</td>
<td>143.2</td>
<td>NA</td>
<td>27</td>
</tr>
</tbody>
</table>

Standard 6 by 12-in. cylinders were used for concrete sampling. The cylinders were first placed next to the deck panels and girders for 24 hours. Molded girder samples were stored in the structures laboratory while deck concrete samples were unmolded and placed in a moist cure room. Note that both methods are acceptable by the ASTM standard. Compressive strength tests were performed in accordance with ASTM C39 standard (2010). Table 5.2 presents the compressive strength for concrete used in the deck panels and girders. The compressive strength history is shown in Fig. 5.1.

<table>
<thead>
<tr>
<th>Time (Day)</th>
<th>Deck Panels, f'c (psi)</th>
<th>Girder A*, f'c (psi)</th>
<th>Girder B, f'c (psi)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>-</td>
<td>6,820</td>
<td>6,190</td>
</tr>
<tr>
<td>7</td>
<td>7,471</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>28</td>
<td>7,921</td>
<td>7,971</td>
<td>7,420</td>
</tr>
<tr>
<td>Bridge-Strength Test-Day</td>
<td>8,752</td>
<td>8,722†</td>
<td>8,339‡</td>
</tr>
</tbody>
</table>

† Tested 77 days after casting
‡ Tested 76 days after casting
*

* Girder A was poured one day before Girder B
** The measured compressive strengths are the average of three 6 by 12-in. concrete cylinder test data
5.1.3 Properties of Grout

Fifteen standard 2 by 2-in. cube samples were collected for each mix of conventional non-shrink grout and latex modified concrete (LMC), which were used as filler materials in different precast joints. Table 5.3 presents a summary of the compressive strength for the filler materials used in the shear pockets and haunch region. The compressive strength of the filler materials used in the transverse joints is presented in Table 5.4. Figure 5.2 shows the compressive strength history for these materials.

Table 5.3: Compressive Strength of Grout Used in Haunch and Shear Pockets

<table>
<thead>
<tr>
<th>Time (Day)</th>
<th>Non-shrink Grout (psi)</th>
<th>Latex Modified Concrete (psi)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>4,427</td>
<td>6,178</td>
</tr>
<tr>
<td>7</td>
<td>6,846</td>
<td>6,595</td>
</tr>
<tr>
<td>28</td>
<td>9,099</td>
<td>7,695</td>
</tr>
</tbody>
</table>

Bridge-Strength Test-Day: 9,402† 8,118‡

† Tested 38 days after casting
‡ Tested 37 days after casting

* The measured compressive strengths are the average of three 2-in. cubes
Table 5.4: Compressive Strength of Grout Used in Transverse Joints

<table>
<thead>
<tr>
<th>Time (Day)</th>
<th>Non-shrink Grout (psi)</th>
<th>Latex Modified Concrete (psi)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>-</td>
<td>5,934</td>
</tr>
<tr>
<td>3</td>
<td>6,120</td>
<td>-</td>
</tr>
<tr>
<td>7</td>
<td>7,044</td>
<td>6,042</td>
</tr>
<tr>
<td>28</td>
<td>8,685</td>
<td>7,359</td>
</tr>
<tr>
<td>Bridge-Strength Test-Day</td>
<td>9,564†</td>
<td>7,487†</td>
</tr>
</tbody>
</table>

†Tested 36 days after casting

* The measured compressive strengths are the average of three 2-in. cubes

5.1.4 Properties of Prestressing Strands

Low-relaxation Grade 270 prestressing strands with a diameter of 0.6 in. were utilized in this project. The mechanical properties of the strands are summarized in Table 5.5 based on the mill certificate provided by the manufacturer (Appendix E).
Table 5.5: Prestressing Strand Properties

<table>
<thead>
<tr>
<th>Cross-Sectional Area (in.²)</th>
<th>Modulus of Elasticity (ksi)</th>
<th>Yield Stress (psi)</th>
<th>Ultimate Stress (psi)</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.22</td>
<td>29,000</td>
<td>254,386 at 1% extension</td>
<td>287,809 at 7.4% extension</td>
</tr>
</tbody>
</table>

5.1.5 Properties of Horizontal Shear Studs

Dog-bone samples were prepared for the tensile testing of reinforcement used as horizontal shear studs in accordance with ASTM 370. This section includes a summary of the measured data.

5.1.5.1 Inverted U-Bars

No. 4 inverted U-bars that extended from the girder top flange into the full-depth shear pockets were made of ASTM A615 Grade 60 reinforcing steel bars. Table 5.6 presents the measured mechanical properties for the inverted U-bars.

Table 5.6: Inverted U-Bar Mechanical Properties

<table>
<thead>
<tr>
<th>Bar Size</th>
<th>ASTM Type</th>
<th>Yield Strength, $f_y$ (ksi)</th>
<th>Ultimate Strength, $f_u$ (ksi)</th>
<th>Strain at Peak Stress (%)</th>
<th>Strain at Fracture (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>No. 4</td>
<td>A615 Grade 60</td>
<td>74.9</td>
<td>113.6</td>
<td>7.0</td>
<td>13.4</td>
</tr>
</tbody>
</table>

Note: Measured data were based on the average of two tensile tests.

5.1.5.2 Double Headed Studs

No. 5 ASTM A706 Grade 60 double headed reinforcing steel bars were used in the hidden shear pockets as shear studs. Table 5.7 presents the mechanical properties of the double headed stud according to the mill certificate provided by the manufacturer (Appendix E).

Table 5.7: Double Headed Stud Mechanical Properties

<table>
<thead>
<tr>
<th>Cross-Sectional Area (in.²)</th>
<th>Yield Strength (ksi)</th>
<th>Ultimate Strength (ksi)</th>
<th>Strain at Fracture (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.31</td>
<td>69.9</td>
<td>90.7</td>
<td>17</td>
</tr>
</tbody>
</table>
5.1.6 Properties of Reinforcement in Panels and Joints

Tensile tests were performed on dog-bone samples of steel bars used in the test bridge transverse joints and deck panels. A summary of the test data is presented in Table 5.8 and table 5.9.

<table>
<thead>
<tr>
<th>Bar Size</th>
<th>ASTM Type</th>
<th>Yield Strength, ( f_y ) (ksi)</th>
<th>Ultimate Strength, ( f_u ) (ksi)</th>
<th>Strain at Peak Stress (%)</th>
<th>Strain at Fracture (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>No. 6</td>
<td>A615 Grade 60</td>
<td>71.5</td>
<td>112.5</td>
<td>7.4</td>
<td>14.8</td>
</tr>
</tbody>
</table>

Note: Measured data were based on the average of two tensile tests.

<table>
<thead>
<tr>
<th>Bar Size</th>
<th>ASTM Type</th>
<th>Yield Strength, ( f_y ) (ksi)</th>
<th>Ultimate Strength, ( f_u ) (ksi)</th>
<th>Strain at Peak Stress (%)</th>
<th>Strain at Fracture (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>No. 6</td>
<td>A615 Grade 60</td>
<td>63.4</td>
<td>107.3</td>
<td>7.2</td>
<td>14.9</td>
</tr>
</tbody>
</table>

Note: Measured data were based on the average of two tensile tests.

5.1.7 Properties of Elastomeric Neoprene Bearing Pads

A 6 by 6 by 3/8-in. elastomeric neoprene pad was tested in a compression machine to determine the force-deformation relationship of the bearing pads used at the supports (Fig. 5.3). The stiffness of the linear portion of the force-displacement relationship was 1,128 kip/in.
5.2 Bridge Test Results

The bridge specimen was first tested under 500,000 cycles of the Fatigue II loading using a point-load applied at the mid-span. Then, it was subjected to 150,000 cycles using two point loads applied adjacent to the middle panel (Panel C) transverse joints. Finally, it was loaded monotonically to failure.

5.2.1 Phase I- Fatigue II Loading

A 27.7-kip point-load was applied at mid-span at a rate of 1 cycle per second for a total of 500,000 cycles. Stiffness tests were performed at 50,000 load cycle intervals.

5.2.1.1 Observed Damage

At 25,000 load cycles, which corresponds to 4.6 years of service, a vertical hairline crack was observed on the grouted haunch of Girder A located approximate 4.2-
ft south of mid-span in one of the latex modified concrete (LMC) joints. One of the water ponds was on the top of this joint. Since (1) the crack width did not change over the entire fatigue test (Fig. 5.4), (2) the pond did not lose water from this leak, and (3) this joint was the last joint filled with LMC (LMC sets approximately in 30 min.), it was concluded that the leak was because of construction issues but not structural degradation due to fatigue. Furthermore, there was no change in bridge overall stiffness due to this crack.
At 125,000 load cycles, which corresponds to 22.8 years of service, vertical hairline cracks were observed along the length of the grouted haunch of both girders approximately evenly spaced between 2 to 4 in. (Fig. 5.5). Both the conventional non-shrink grout and the latex modified concrete exhibited vertical hairline cracking in the haunch area. Also, hairline shallow cracks were observed in all transverse joints (Fig. 5.6) and most shear pockets (Fig. 5.7). Since water did not leak through these cracks, the crack width did not increase over time, and there was not significant change in the bridge...
overall stiffness, it was concluded that these hairline cracks were caused by shrinkage but not fatigue loading.

Figure 5.5- Vertical Hairline Shrinkage Cracks at 125,000 Load Cycles
Figure 5.6- Transverse Joint Shrinkage Cracks

(a) Latex Modified Concrete at 125,000 Cycles  (b) Latex Modified Concrete at 650,000 Cycles

Figure 5.7- Full-Depth Shear Pocket Shrinkage Cracks at 125,000 and 650,000 Load Cycles

(a) Non-Shrink Grout at 125,000 Cycles  (b) Non-Shrink Grout at 650,000 Cycles

(c) Latex Modified Concrete at 125,000 Cycles  (d) Latex Modified Concrete at 650,000 Cycles
5.2.1.2 Stiffness Degradation and Joint Integrity

The measured force-displacement relationship for each stiffness test performed after every 50,000 load cycles is shown in Fig. 5.8. The stiffness was measured based on the applied loads and the average girder net mid-span deflections. It can be seen that the bridge overall stiffness essentially remained the same throughout fatigue testing indicating sufficient detailing for the proposed bridge system. The measured effective stiffness (EI) of the bridge versus the number of load cycles is shown in Fig. 5.9. The Phase I loading EI values were calculated as:

\[ EI = \frac{PL^3}{48\Delta} \]  

Eq. 5-1

where:

\( E \) = the concrete modulus of elasticity (ksi),
\( I \) = the moment of inertia of the cross section (in.\(^4\)),
\( P \) = the applied load from stiffness test (kips),
\( L \) = the test bridge effective span length (in.),
\( \Delta \) = the test bridge net mid-span deflection from stiffness test (in.)

The Phase II loading EI values were calculated using the following equation:

\[ EI = \left( \frac{P}{2} \right)^a \frac{a}{24\Delta} * \left( 3L^2 - 4a^2 \right) \]  

Eq. 5-2

where \( a \) = the distance between two point loads (in.). All other parameters were previously defined.
Figure 5.8- Measured Stiffness from Phase I Loading

Figure 5.9- Stiffness Degradation during Phase I and Phase II of Fatigue Testing

Figure 5.10 shows the measured joint relative deflections versus the number of load cycles for the two joints of Panel C. The joint relative deflections were negligible and remained essentially constant through all 500,000 load cycles of the Phase I fatigue testing. Figure 5.11 shows the measured joint rotations versus number of load cycles for
the two joints of Panel C. The joint rotations were negligible and remained essentially constant through all 500,000 load cycles of the Phase I fatigue testing.

![Figure 5.10- Transverse Joint Relative Deflection vs. Number of Load Cycles during Phase I Fatigue Testing](image)

![Figure 5.11- Transverse Joint Rotation vs. Number of Load Cycles during Phase I Fatigue Testing](image)

The relative displacement between the girder top flange and the deck bottom (deck-girder slippage) was measured during each stiffness test (Fig. 5.12). The deck-
girder slippage was negligible and remained essentially constant through all 500,000 load cycles of Phase I fatigue testing.

Figure 5.12- Deck-Girder Slippage vs. Number of Load Cycles for Phase I Fatigue Testing

5.2.2 Phase II- Joint Loading

A 27.7-kip point load was applied to a spreader beam to induce two 13.9-kip point-loads adjacent to Panel C’s transverse joints at a rate of 1 cycle per second for a total of 150,000 cycles. Stiffness tests were performed every 50,000 load cycles.

5.2.2.1 Observed Damage

Figure 5.13 shows the middle panel transverse joints with either non-shrink grout or latex modified concrete after applying 150,000 cycles of joint loading. All joints remained water tight through the duration of joint loading. No significant damage of the bridge components in addition to what was reported in Phase I testing was observed.
5.2.2.2 Stiffness Degradation and Joint Integrity

The measured force-displacement relationship for each stiffness test performed after every 50,000 load cycles of the transverse joint fatigue testing is shown in Fig. 5.14. The stiffness was measured based on the applied loads and the girder net mid-span deflections. It can be seen that the bridge overall stiffness essentially remained the same throughout the transverse joint fatigue testing indicating sufficient transverse joint detailing for the proposed bridge system.

Figure 5.13- Transverse Joint Damage Under Deck

(a) Non-Shrink Grout  (b) Latex Modified Concrete
Figure 5.15 shows the measured joint relative deflections versus number of load cycles for both joints of Panel C during Phase II. The joint relative deflections were negligible and remained essentially constant through all 150,000 load cycles of the Phase II transverse joint fatigue testing. Figure 5.16 shows the measured joint rotations vs. number of load cycles for both joints of Panel C under the Phase II loading. The joint rotations were negligible.
The deck-girder slippage versus number of load cycles is shown in Fig. 5.15. The deck-girder slippage remained essentially constant and negligible through all 150,000 load cycles of Phase II.

Figure 5.15- Joint Relative Deflection vs. Number of Load Cycles during Phase II Loading

Figure 5.16- Joint Rotation vs. Number of Load Cycles during Phase II Loading
5.2.3 Strength Test

A point-load at the mid-span of the bridge was applied to a beam placed in the transverse direction of the bridge to spread the load to the two girders. The girders were loaded under displacement-control monotonic loading to 263 kips, where the test was stopped because of the setup limitation.

5.2.3.1 Observed Damage

The first crack in the girder was observed at the mid-span at an actuator load of 149 kips (Fig. 5.18a). Subsequently, more cracks were formed on the girders close to the mid-span at higher loads as shown in Fig. 5.18.
The first horizontal shear cracks in the grouted haunch area (Fig. 5.19) were observed at an actuator load of 200 kips, which corresponds to a girder load of approximately 100 kips. However, horizontal shear stud strain gauge data (see section 5.2.3.3) suggests that cracking occurred at lower loads. Additional shear cracks appeared at an actuator load of 226 kips (Fig. 5.20). Note that shear cracks did not form under an equivalent Strength I Limit State load for this bridge, which was 131.4 kips, indicating that the shear reinforcement detailing was sufficient.
Figure 5.19- Haunch Region Horizontal Shear Cracks at an Actuator Load of 200 kips
5.2.3.2 Force-Displacement Relationship

Figure 5.21 shows the average force-displacement relationship for the girders at the mid-span. The figure also shows the equivalent loads for different limit states. The mid-span net girder deflection at the Service I limit state load of 76.7 kips was 0.29 in., which was only 39% of the AASHTO allowable deflection at this limit state (0.74 in.) for this bridge. The girder deflection at the peak applied load of 263 kips was 1.14 in. The test was stopped at 263 kips because of the setup limitation.
Figure 5.21 shows that the first girder cracking occurred at a higher load than that of the Strength I limit state indicating that the bridge design was sufficient (since the superstructure should remain capacity protected). No yielding of the prestressing tendons was observed during the ultimate test. The calculated tendon yield force based on a moment-curvature analysis (Appendix B.3) was 362 kips. Overall, the bridge showed satisfactory performance in terms of displacement and force capacities.

![Figure 5.21- Measured Girder Force-Deflection Relationship at Mid-Span Under Strength Test](image)

Four load cells were installed under the South end girders to measure the girder reactions continuously. Reactions at applied loads corresponding to the Service I limit state, the Strength I limit state, first cracking, and the ultimate load are shown in Fig. 5.22. It can be seen that approximately 49% of the applied load was resisted by Girder A and the remaining load was resisted by Girder B. The total south end reaction force was 24.9 kips under the equivalent Service I limit state load, 58.8 kips under the equivalent Strength I limit state load, 67.0 kips under the first cracking load, and 119.1 kips under the peak load. It was noticed that the South end measured reactions were always 10%
lower than the calculated reactions from statics. This can be because of a slight offset in the actual location of the applied load.

![Diagram of reaction force distribution](image)

**Figure 5.22- Measured End Support Reactions**

### 5.2.3.3 Measured Strains

Strain gauges were installed on prestressing strands and reinforcing steel bars. The measured strain data during the strength test is discussed herein.
5.2.3.3.1 Tendon and Reinforcement Strains

Figure 5.23 shows the strain of prestressing strands measured during the strength testing. The initial strains of the strands were determined using the strain gauge data collected during stressing. Note that the strand initial strains account for the short-term losses such as elastic shortening but not long-term losses such as relaxation, creep, and shrinkage. It can be seen that the tendons did not yield up to 263 kips where the test was stopped. The yield strain of the tendons is 8,772 micro-strain. Figure 5.24 shows the measured strains for the longitudinal deck mild steel and the embedded concrete strain gauges during ultimate loading. The embedded concrete strain gauges include the initial strain recorded during cutting of the prestressing strands. It can be seen that the longitudinal deck mild steel did not yield up to 263 kips. The embedded concrete strain gauges were located 1.6 in. below the theoretical composite girder section neutral axis. The measured concrete strains are in agreement with calculated strains from statics.

![Figure 5.23- Measured Prestressing Strand Strains during Strength Test](image-url)
5.2.3.3.2 Shear Stud Strains and Stresses

The actuator load versus measured strain for the double-headed shear studs is shown in Fig. 5.25. It can be seen that the double-headed studs did not yield in any direction. Since the strain gauges were installed in a rosette type layout in each pocket (one in the axial direction of the stud, and two at ±45-degrees with respect to the stud longitudinal axis), the maximum principal stresses (Fig. 5.26) could be estimated for the studs in each pocket. It can be seen that the maximum principal stress of the double-headed studs is 19.4 ksi, which is well below the yield strength (69.9 ksi) indicating sufficient design.

The load required to cause horizontal shear cracks in the girder haunch can be determined using the stud strain or stress data where strains or stresses suddenly change (Fig. 5.26). The deflection associated with the sudden change of strains in studs was
identified then converted to the actuator load using the force-displacement relationship. The first haunch cracks based on the measured data (Fig. 5.26) of the headed studs in the hidden pockets filled with non-shrink grout occurred at an actuator load of 100.6 kips, which is larger than the Service I limit state load of 76.7 kips.

![Figure 5.25- Measured Strain for No. 5 Double Headed Studs during Strength Test](image-url)
The actuator load versus measured strain for the inverted U-shape shear studs is shown in Fig. 5.27. It can be seen that these studs did not yield in any direction. The maximum principal stresses (Fig. 5.28) were estimated in each pocket similar to what was done for the double-headed studs. It can be seen that the maximum principal stresses of the inverted U-shape shear studs (23.9 ksi for latex modified concrete (LMC) and 27.6 ksi for non-shrink grout) are well below the yield strength indicating sufficient design.

The load required to cause horizontal shear cracks in the girder haunch was also determined. The first haunch cracks based on the measured data (Fig. 5.28) for the inverted U-shape shear studs in the full-depth pockets filled with non-shrink grout occurred at an actuator load of 124 kips and at a load of 149 kips for the full-depth pockets filled with LMC. Both aforementioned loads are larger than the Service I limit state load of 76.7 kips.
Figure 5.27- Measured Strain for No. 4 Inverted U-Shape Studs during Strength Test

Figure 5.28- Maximum Principal Stresses for No. 4 Inverted U-Shape Studs vs. Mid-Span Deflection during Strength Test
5.2.3.3 Transverse Joint Reinforcement Strains

Figure 5.29 shows the measured strains of the transverse bars in the transverse joints during the strength test. Strain gauge SG-A-15 failed at 170 kips (marked with * in Fig. 5.29). It can be seen that none of the strains exceeded 50 microstrain and were negligible.

Figure 5.29- Measured Strains of No. 6 Transverse Bars in Transverse Joints during Strength Test

Two transverse joint lap-spliced No. 6 bars had strain gauges in a rosette type layout to estimate the maximum principal stresses (Fig. 5.30). It can be seen that the maximum principal stress for reinforcement in joints filled with either non-shrink grout or LMC are well below the yield strength indicating sufficient design.
5.2.3.4 Performance of Joints

The middle panel’s joints relative deflections and rotations during strength testing are shown in Fig. 5.31. The joint filled with non-shrink grout had a relative deflection of 0.0014 in. at 263 kips. The joint filled with latex modified concrete had a relative deflection of 0.0015 in. at 263 kips. Both deflections were negligible. Furthermore, the joint filled with non-shrink grout had a rotation of 0.009 degrees at 263 kips. The joint filled with latex modified concrete had a rotation of 0.01 degrees at 263 kips. Both joint rotations were negligible.
The relative displacement between the bottom of the deck and the top of the girder (deck-girder slippage) was measured in three locations. Figure 5.32 shows the deck-girder slippage during the strength test. A plateau can be seen at a girder load of approximately 60 kips, which can be attributed to the cracking of the haunch region (Fig. 5.19), and the shear deformation of the haunch.
6. Evaluation of Full-Depth Deck Panels on Inverted Bulb-Tee Girders System

This chapter includes an evaluation of the full-depth deck panels supported on inverted bulb-tee girders system for field deployment. The evaluation includes: (1) structural performance, (2) comparison with the modified double-tee bridge girders (a new section with improved long-term joint performance), (3) constructability, and (4) cost of the superstructure.

6.1 Performance under Service, Fatigue II, and Strength Limit States

The number of trucks passing the prototype bridge over a 75-year design life is 411,000 based on the average daily truck traffic (ADTT) of 15 for local roads in South Dakota. The full-scale single-lane test bridge was subjected to 500,000 load cycles at the mid-span and an additional 150,000 load cycles adjacent to the mid-span panel transverse joints to maximize the shear transfer. The load at the mid-span corresponded to the moment experienced by the interior girders of the prototype bridge based on the Fatigue II limit state loading specified in AASHTO (2013).

The test bridge showed no signs of stiffness degradation and remained water-tight through 650,000 fatigue load cycles (Fig. 6.1). Note that 650,000 fatigue load cycles is equivalent to 119 years of service for this bridge located on South Dakota local roads.
The stiffness change during the entire fatigue test was less than 3% with respect to the bridge initial stiffness. Shallow shrinkage cracks were observed in the haunch area and the deck full-depth pockets as well as the transverse joints filled with either conventional grout or latex modified concrete. However, no crack was observed on the hidden pockets. No other significant damage was observed during the entire fatigue testing for decks, joints, and girders.

The equivalent AASHTO (2013) Service I limit state load was 76.7 kips and the Strength I limit state load was 131.4 kips. The test bridge girders did not crack at these limit states. The test bridge girder first flexural crack occurred at a load of 149 kips, which indicates that the bridge has adequate capacity (Fig. 6.2). More cracks formed on the girders at higher loads. The test was stopped at 263 kips due to setup limitations. The load corresponding to the flexural failure of the test bridge was 402 kips based on a moment-curvature analysis. No significant damage was observed in the deck panels, joints, and haunch region under the entire ultimate testing.
6.2 Comparison with Modified Double-Tee Girders

Double-tee girders are a precast and prestressed bridge section commonly used for bridge superstructures on South Dakota local highways. A previous experimental study by Wehbe et al. (2016) was performed to modify the longitudinal joint detail to improve serviceability and strength performance. They showed that the stiffness of the modified double-tee girders did not deteriorate under 500,000 fatigue load cycles while original double-tee girders were not structurally sufficient for long-term performance (Fig. 6.1). The present experimental study confirmed that the stiffness of the full-depth deck panels supported on inverted bulb-tee girders will remain essentially unchanged for a 75-year design life.
6.3 Constructability

The constructability of main components of the precast full-depth deck panels supported on inverted bulb-tee girders is evaluated herein.

6.3.1 Precast Inverted Bulb-Tee Girders

The precast inverted bulb-tee girders were cast using partial-depth I-girder forms. Overall, the proposed girder design and construction are similar to current practice.

An actual bridge on a local road will typically consist of seven inverted bulb-tee girders. Whereas, local road bridges built with double-tee girders consist of nine girders. Onsite construction is expected to be rapid for each system but more involved for the proposed inverted bulb-tee girder bridges since there are more joints to be filled.

6.3.2 Full-Depth Deck Panels

The formwork for full-depth deck panels were made of 2 by 4-in. lumber and plywood. Overall, current practice can be applied for the design and construction of the proposed deck panels.

The full-depth deck panels were quickly installed in the laboratory. In terms of onsite activities, special care should be taken on the adjustment of the panel grades, which can be easily done by adjusting the leveling bolts. Double-tee bridges will be easier to install onsite since the deck is integrated with the webs.
6.3.2.1 Shear Pockets

The hidden pocket detail was formed using plywood for the pocket and polyvinyl chloride pipes for the grout and vent ports. Fabrication of the hidden shear pockets was relatively easy and efficient. Pockets should be cleaned before installation of the panel.

6.3.2.2 Horizontal Shear Studs

Both the double headed and inverted U-shape shear studs were found to be viable options for use in inverted bulb-tee girders. The studs are installed prior to girder casting using current practice.

6.3.2.3 Transverse Joints

The transverse joint female-to-female shear key geometry was formed with plywood. The hollow structural steel sections (HSS) were secured to the transverse joint formwork using threaded rods, nuts, and steel plates inside the HSS. Figure 6.3 shows HSS in the panel formwork prior to casting. Transverse joints can be easily prepared, sealed, and filled with grout from the top of the bridge during onsite construction.
6.3.3 Leveling Bolts

The leveling bolt detail was formed using a threaded rod welded to a steel plate at the bottom and a nut at the top, a vertical steel pipe embedded in concrete to encase the rod, and a 2 by 4-in. lumber piece for the blockout at the top of the deck (Fig. 6.4).
6.3.4 Grouted Haunch

It is expected that forming and sealing the grouted haunch of the proposed system from the top of the bridge will be the most challenging onsite activity. In the laboratory, the grouted haunch was formed by securing 2 by 4-in. lumbers between girders to hold ¾-in. thick plywood against the girder sides (Fig. 6.5). Placing the forms inside the girders was easier than placing forms outside the girders, which required clamping reaction lumber to the bridge deck to secure 2 by 4-in. struts and to hold the plywood.

![Grouted Haunch Dam Formwork Installed between Girders](image)

Figure 6.5- Grouted Haunch Dam Formwork Installed between Girders

6.4 Cost

Table 6.1 presents a comparison of superstructure materials and fabrication cost for the double-tee and the proposed bridge systems for a 50-ft long by 34.5-ft wide bridge. The materials and fabrication cost for 46-in. wide by 23-in. deep precast double-tee girders is approximately $247 per linear foot based on data provided by the South Dakota Department of Transportation. Nine double-tee girders are used in a 50-ft long by 34.5-ft wide bridge, which would cost approximately $111,150 for the superstructure materials and fabrication.
The 21-in. deep precast inverted bulb-tee girders were estimated to cost $130 per linear ft, and the precast 8-in. thick full-depth deck panels were estimated to cost $45 per square ft. The 50-ft long by 34.5-ft wide actual bridge materials and fabrication cost estimate in Table 6.1 was calculated based on seven 50-ft long by 21-in. deep precast inverted bulb-tee girders with a total cost of $45,500, and five 34.5-ft wide by 10-ft long by 8-in deep precast full-depth deck panels with a total cost of $77,625. The total materials and fabrication cost estimated by the manufacturer for precast full-depth deck panels supported on inverted bulb-tee girders is approximately $123,000 for the actual bridge. Therefore, the materials and fabrication cost of this type of bridges is approximately 11% more than that for double-tee bridges.

<p>| Table 6.1: Bridge Superstructure Materials and Fabrication Cost Estimate Comparison |
|-----------------------------------------------|------------------|------------------|</p>
<table>
<thead>
<tr>
<th>Bridge System</th>
<th>Full-Depth Deck Panels on Inverted Bulb-Tee Girders</th>
<th>Double-Tee Girders</th>
</tr>
</thead>
<tbody>
<tr>
<td>Materials/Fabrication ($)</td>
<td>123 K</td>
<td>111 K</td>
</tr>
<tr>
<td>Total ($/sq. ft.)</td>
<td>71</td>
<td>64</td>
</tr>
</tbody>
</table>

An approximate mobilization cost estimate can be determined by assuming $4 per loaded mile on a legal load of 42,000 lbs.

Additional costs such as onsite activities and substructure fabrication and construction should be included in the total bridge cost. However, comparing the superstructure cost will better show the benefit of each design alternative. The superstructure cost for a fully precast 50-ft long full-depth deck panels supported on inverted bulb-tee girder bridge (including material and fabrication, placing the girders and panels, grouting the shear pockets, haunch, and transverse joints) is presented in
Table 6.2. Note that the cost estimate does not include the cost of the substructure, mobilization, and railings.

<table>
<thead>
<tr>
<th>Bridge System</th>
<th>Full-Depth Deck Panels on Inverted Bulb-Tee Girders</th>
</tr>
</thead>
<tbody>
<tr>
<td>Materials and Fabrication ($)</td>
<td>123,000</td>
</tr>
<tr>
<td>Onsite Activity ($)</td>
<td>47,000 – 63,000</td>
</tr>
<tr>
<td>Total ($/sq. ft.)</td>
<td>99 - 108</td>
</tr>
</tbody>
</table>

Overall, the cost of the proposed bridge system is slightly more than the double-tee bridge system, which is the most common type of bridge on South Dakota local roads. It is expected that the proposed bridge system will be more competitive with the double-tee bridges when spans are more than 40 ft.
7. Design and Construction Recommendations

This chapter includes design and construction recommendations for full-depth deck panels supported on inverted bulb-tee girder bridge systems. The design recommendations are based on the experimental data of a full-scale bridge test model with the proposed detailing. The construction recommendations are based on literature review, fabricating and assembling of the test girders in the Lohr Structures laboratory, and engineering judgment.

7.1 Inverted Bulb-Tee Girders

Inverted bulb-tee girders should be designed and constructed using current codes and practices.

Horizontal shear studs installed in the inverted bulb-tee girders that will be extended into precast deck pockets require a tight construction tolerance.

7.2 Full-Depth Deck Panels

Full-depth deck panels should have a minimum thickness of 7 in. according to AASHTO LRFD (2013). The width of the full-depth deck panels is recommended to be the same as the bridge width (in the transverse direction) resulting in a single-grade for the bridge deck (Fig. 7.1). Single-grade decks do not need longitudinal joints to connect
precast panels resulting in lower cost, faster construction, and improved durability. The length of each full-depth precast panel (in the longitudinal direction of the bridge) should not exceed 12 ft.

Figure 7.1- Cross-Section of Bridge System with Single-Unit Panel

If a crown along the longitudinal centerline is desired (deck with two grades), the precast panels should be connected with a longitudinal joint along the center of the bridge (Fig. 7.2). Previous studies developed detailing for longitudinal joints (Baer, 2013; Aaleti and Sritharan, 2014). One of the tested longitudinal joint details is shown in Fig. 7.3, which utilizes U-shape reinforcing steel bars extending from two adjacent panels into the longitudinal joint to transfer shear and moment as well as headed bars in the longitudinal direction of the bridge to aid in developing the U-shape reinforcing steel bars.

Figure 7.2- Cross-Section of Bridge System with Two-Unit Panels and a Longitudinal Joint
Figure 7.3 - Longitudinal Joint Detailing (Baer, 2013)
The deck reinforcement should be designed according to a legally adopted code such as AASHTO LRFD (2013). All deck reinforcing steel bars should be epoxy coated since shrinkage cracks may develop in the full-depth pockets, grouted haunch, and at the transverse joints. Epoxy coated bars would increase the durability of the joints.

### 7.2.1 Shear Pockets

The center-to-center spacing of the shear pockets should not exceed 24.0 in. according to Article 5.8.4.2 of AASHTO LRFD (2013). Only hidden pockets should be used (Fig. 7.4) since they provide a better durability. Furthermore, the shear pockets should be designed to allow a minimum of 0.75-in. clear spacing between the shear studs and all side surfaces of the shear pockets. The hidden-pocket grout port diameter should be at least 2-in. to allow grout to be easily poured (Fig. 7.4b). Two ¾-in. diameter vent ports should be provided on the opposite side of the grout port to avoid air pockets. The shears studs should be designed according to Article 5.8.4 of AASHTO LRFD (2013). The embedment length of shear studs into the pocket should not be less than six times the stud diameter \((6d_b)\). Minimum AASHTO required concrete cover from the surface of the deck should be provided for the studs. Two types of shear stud, double-headed and inverted U-shape, are allowed to be inserted in hidden pockets (Fig. 7.4). Full-depth pockets should be avoided since shrinkage cracks may develop at the edge of the full-depth pocket. The pocket can be filled with conventional non-shrink grout.
All pockets should be free of debris, oil, or any other foreign materials to ensure good bond.

7.2.2 Transverse Joints

Figure 7.5 shows the proposed transverse joint detailing. A minimum gap of 2.75 in. in the longitudinal direction of the bridge should be provided between the precast panels. A reinforcing steel bar with the same type, grade, and size as those of the largest deck transverse reinforcement should be placed in the transverse joints. Steel dowels
with the same type, grade, and size as those of the deck largest longitudinal reinforcement should be spliced with the deck reinforcement (Fig. 7.6). Hollow structural steel sections used to reduce the splice length should be galvanized to avoid corrosion and to increase the bridge overall durability.

![Figure 7.5- Female-to-Female Transverse Joint Detailing](image)
All transverse joints should be clean and free of debris or any foreign contaminant to ensure good bond.

### 7.2.3 Leveling Bolts

Leveling bolts should be incorporated in the precast full-depth deck panels to adjust their grades (Fig. 7.7). The use of long bolts in lieu of threaded rods and welded nuts is recommended. Each bolt should be designed to take at 25% of the weight of the precast panel.
7.3 Grouted Haunch

The haunch depth at the bridge mid-span should not be less than 0.75 in. to allow the grout to easily flow through the haunch and to avoid air pockets (Fig. 7.8). A minimum of two longitudinal reinforcing steel bars should be placed in the haunch region and sized according to Article 5.10.8 AASHTO LRFD (2013) to eliminate shrinkage cracking (Fig. 7.8).
Several methods can be used to form the haunch from the top of the bridge. One example is shown in Fig. 7.9 in which the form was made using threaded rods and anchorage plates to clamp plywood to the girder top flange. Compressible foam was glued to the top of the plywood to seal the haunch area after the panel placement.
Figure 7.9- Grouted Haunch Formwork (Aktan and Attanayake, 2013)
8. Summary and Conclusions

The present study was conducted at South Dakota State University to explore the feasibility of different bridge system alternatives for South Dakota local roads. The performance of full-depth deck panels supported on inverted bulb-tee girders was experimentally investigated and the findings are presented herein.

8.1 Summary

The proposed bridge system incorporates precast full-depth deck panels and prestressed inverted bulb-tee girders. A full-scale test bridge specimen was 50-ft long by 9.5-ft wide. The test bridge represented two interior girders from the prototype bridge and was constructed and tested to evaluate the proposed system performance. The precast panels were connected to the precast girders using two types of shear studs: inverted U-shape and double-headed studs. Two types of pockets were used in the test model: full-depth and hidden pockets. The precast panels were connected incorporating transverse joints in which the panel longitudinal reinforcing steel bars were spliced utilizing steel bar dowels dropped in hollow structural steel sections. The pockets, transverse joints, and haunch region were filled with either conventional non-shrink grout or latex modified concrete (LMC).
The bridge was first tested under 500,000 cycles of the AASHTO Fatigue II loading using a point-load applied at the mid-span. Next, the performance of transverse joints was evaluated by applying 150,000 AASHTO Fatigue II load cycles using two point loads applied adjacent to the middle panel transverse joints to maximize the shear transfer. Stiffness tests were performed at every 50,000 load cycle interval for both fatigue tests. Finally, the proposed bridge system was monotonically loaded to 263 kips to investigate the ultimate capacities.

8.2 Conclusions

- The proposed construction process does not require any advanced technology and was relatively simple.
- The proposed bridge system did not exhibit any sign of deterioration or water leakage through 500,000 Fatigue II load cycles (91 service years) and an additional 150,000 Fatigue II load cycles adjacent to the interior panel transverse joints (27 service years). The bridge overall stiffness essentially remained the same throughout the fatigue testing.
- Shrinkage cracks were observed in almost all full-depth shear pockets, all transverse joints, and grouted haunch regions at 125,000 load cycles. Shrinkage cracks in the haunch can be minimized by using two longitudinal reinforcing steel bars placed in the haunch region.
- The first horizontal shear cracks in the grouted haunch region were observed at an actuator load of 200 kips, which was higher than the equivalent AASHTO Strength I limit state load of 131.4 kips.
Both inverted U-shape shear studs and double headed shear studs performed adequately through the entire fatigue testing as well as the ultimate testing.

The hidden pocket detail was found to be a better alternative than the full-depth pockets since they provide a better durability. Shrinkage cracks were observed in almost all full-depth pockets, but none for hidden pockets.

The test bridge girders did not crack until the applied load exceeded the equivalent Strength I limit state load indicating adequate design and performance.

No significant damage in addition to the shrinkage cracks was observed through the entire fatigue test, and the overall bridge stiffness did not deteriorate.

The superstructure materials and fabrication cost of the proposed system for a 50-ft long by 34.5-ft wide bridge is 11% higher than that for a double tee bridge with the same bridge geometry.

Overall, it can be concluded from the design, construction, testing, and cost data that the proposed bridge system, full-depth deck panels supported on inverted bulb-tee girders, is a viable alternative to the precast double-tee girder bridges, which are common on South Dakota local roads.
9. REFERENCES


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