Full Depth Precast Concrete Highway Bridge Decks

BY
PROF. M.G. OLIVA
PROF. L.C. BANK
PROF. J.S. RUSSELL
Full Depth Precast Concrete Highway Bridge Decks

Report to the Wisconsin Department of Transportation
October, 2007

M. G. Oliva, L.C. Bank, J.S. Russell
University of Wisconsin
Dept. of Civil and Environmental Engineering

Co-Authors: Scott Markowski, Greg Ehmke, Sung Je Chi, Research Assistants

Innovative Bridge Research and Construction
Contract Number: 0607-48-01

July 1, 2003 – October 31, 2007

Submitted to:
Wisconsin Department of Transportation (WisDOT)
DTID – Research Section
ATTN: Scott Becker
P.O. Box 7916
Madison, WI 53707-7916
Summary

Traditionally the Wisconsin DOT uses cast-in-place methods for building and replacing highway bridge decks. During this process, traffic must be completely closed off for a long period to allow for forming and curing of the concrete.

An alternative to using cast-in-place decks is the use of full-depth precast concrete deck panels. Panels are constructed off-site under controlled conditions and brought to the site ready for installation. Using precast deck panels can significantly reduce construction time, thereby decreasing delays to motorists. It also increases work zone safety by reducing the number of and exposure time of workers operating near moving traffic, reduces environmental impacts by minimizing the site access footprint, and improves the constructability of bridge designs.

This project, funded by FHWA through WisDOT as part of the Innovative Bridge Research and Construction program, developed a full depth precast concrete bridge deck system for Wisconsin. Use of the system was demonstrated through construction of a new deck on a heavily traveled Interstate highway (Door Creek Bridge on I-39/90). The construction process was closely monitored and the bridge was load tested to measure its performance. Analytic simulations were used to predict the overloads that the bridge could withstand before significant damage developed.

Unique Innovations:

Three unique innovations were used for the prototype Door Creek Bridge. First, the bridge was redecked in stages to maintain traffic, so panels were cast at approximately half the width of the bridge. The staging used a unique longitudinal joint between the precast panels. The joint was also placed in a unique location. It was not located over a girder as had been done in other projects. The new joint design proved to be very effective.

Second, the shear studs used to develop composite action between the panels and the girders were placed farther apart than typical AASHTO standards. The wider spacing for this project was accepted based on performance measurements from laboratory testing. The girder performance was not affected by the wider spacing.

Third, panels were developed with sufficient edge strength that wheel loads could be placed on the panel edges without damage. This would allow portions of an old deck to be removed, possibly during overnight construction, and replaced with new precast panels that would be able to withstand truck traffic the next day without continuity at the panel edge.

System Development through Laboratory Testing:

1. Deck Edge Strength
The edges of the precast deck panels were specially designed to be able to carry truck wheel loading without the normal edge diaphragm used in cast-in-pace decks. Laboratory load testing on the panel edges proved that the decks could resist loads 5.3 times the expected service wheel load plus impact.

2. Longitudinal Joint Performance
In staged construction one portion of the bridge is carrying truck live load while a second portion is placed. In this case the second portion is precast panels that must be connected to the first stage with a grouted joint. Laboratory testing proved that the grouted joint between panels could withstand relative movement of 0.01 inches while the joint grout cured without detrimental effect on the joint performance as long as the joint was subsequently post-tensioned.

The tests also proved that a prestress level of 250 psi is sufficient to insure that the joints do not open under service plus impact loading.
3. Transverse Joint Performance
The performance of transverse joints between panels was also measured by laboratory testing. The testing proved that the joints could perform without cracking if the prestress level was sufficient. Analytic non-linear models of the joint behavior were also developed from the test data that allowed subsequent modeling of the bridge behavior as failure developed.

4. Composite Girder Behavior
To avoid having numerous holes in the precast deck, necessary for connecting the deck to the girder, laboratory tests were used to prove that the spacing limits of the AASHTO LRFD design specifications could be violated. A \( \frac{1}{2} \) scale test girder, simulating the Door Creek Bridge girders, was subjected to over 2 million cycles of service load and tested to high overloads. The test proved that excellent composite action could be obtained with wide spacing of composite shear connectors.

Prototype Bridge Construction:

Prototype Bridge
This innovative full depth precast deck system was used to replace the old cast-in-place deck for the Door Creek Bridge, B-13-161, on I-39/90. The bridge is an 83ft. long single span structure with a 30° skew. Twenty precast deck panels were used. The panels were up to 34.5 ft. long, 6ft – 11in. wide and 8.75in. thick, weighing 31,100lbs.

The need to maintain traffic required staged construction with a longitudinal joint. A unique configuration was used to create this joint. Both the longitudinal and transverse joints were of a keyed female-female configuration. After the keyways were grouted, the bridge was post-tensioned both longitudinally and transversely to create an fully prestressed concrete deck. The full prestressing is expected to significantly reduce concrete cracking and improve the durability and life span of the bridge deck.

The contractor bid cost on this bridge was 280% of a companion cast-in-place bridge. Even though the low bid contractor was awarded the project, that bid had a unit price for the deck panels that was 35% higher than the second bidder. If the lower panel price had been used, and if a longitudinal joint with staged construction was not needed, the precast deck would have been priced competitively with the companion cast-in-place deck.

The experience on this bridge clearly indicates that use of full depth precast decks is feasible and may be economically very competitive with cast-in-place. Where staged construction is needed the cost will be higher. In that case the precast deck will probably only be attractive when significant other incentives, such as rapid construction, are considered as offsetting the higher cost.

Problems
Unique problems were encountered during construction of this demonstration bridge that must be addressed the next time this system is used. First, the special provisions erringly required the deck concrete to meet ASTM C1202 test for chloride permeability. The ASTM test was later found to be inappropriate and should not be included in future projects.

Second, the most serious problem occurred when the contractor used poor techniques for splicing and sealing post-tensioning ducts. Grout leaked into the ducts and bonded the prestressing strand before it was post-tensioned. After removing the strand and cleaning the ducts the prestress was satisfactorily achieved.

As a result of having hardened grout in the duct the contractor was unable to pump grout through the duct from one end to the other after tensioning the strand. They then placed grout in the opposite port. This was specifically forbidden in the contract special provisions. As a result it is likely that 4 of the tendons in the bridge are not fully grouted and may be subject to corrosion.
These two errors likely occurred because the contractor did not follow requirements of the special provisions that specified a plan for grouting must be submitted. The project engineer did not receive the submittal and did not demand it. When a new procedure such as post-tensioning is used, it is essential that the engineer enforces the contract documents.

Third, insufficient oversight was provided for the contractor when shear studs for composite action were attached to the steel beams. Stud spacing and placement requirements were violated.

Suggestions
Suggestions from the precaster, contractor, WisDOT employees and the engineer of record were elicited at the end of the project. Two of the most important comments received were: 1) the State should specify a single supplier of post-tensioning materials to simplify design for the precaster, avoid confusion, improve construction coordination and simplify detailing; and 2) the State should use a special contracting method on bridges where rapid construction is desired, such as a time penalty and reward system, otherwise the contractor has no incentive to change normal construction scheduling or construction methods.

Analytic Prediction of Performance
The results from the laboratory testing of longitudinal and transverse joints were used to develop accurate non-linear performance models of the Door Creek and other bridges. These models served to evaluate the needed prestress levels for good joint performance in precast deck bridges and to predict how the bridge would perform if it was overloaded.

The analysis showed that normally the joints between precast panels should be prestressed to 250 psi. On multi-span bridges, however, where the deck provides a tension flange over the piers for the composite girder, higher levels of prestress may be needed in the joints and should be calculated as part of the design procedure.

While current practice aims at prestressing to completely eliminate tension stress in joints, the analysis showed that in fact overloading and opening of joints will not have immediate detrimental effects on the bridge performance. Also, since prestressing cause a non-linear elastic behavior in joints, after the linear load capacity is exceeded the joint still recovers its original position after an overload. There are not residual deformation effects.

Suggested Design Procedure
Based on the study of other precast deck projects, the laboratory testing, experience with the prototype bridge, and the analytical study of overload effects, a proposed design procedure is suggested. This procedure is provided in a format that could serve as a basis for creating a new section in the Wisconsin Bridge Design Manual – LRFD for full depth precast deck design.

A suggested set of special provisions, focused on the deck panels and their prestressing, are also provided as a basis for creating a future set of standard special provisions for full depth precast deck bridges.
Table of Contents

1. Introduction ............................................................................................. 1
   1.1 Objectives .......................................................................................... 1
   1.2 Scope ................................................................................................. 2

2. Background Review ............................................................................... 2
   2.1 Background ........................................................................................ 2
   2.2 Door Creek Bridge ............................................................................ 3
   2.3 Transverse Joints ............................................................................... 3
   2.4 Longitudinal Joint ............................................................................. 4
   2.5 Transverse Post-Tensioning .............................................................. 4

3. Research Phase Activities – Laboratory testing .................................. 4
   3.1 Transverse Edge Loaded Panels........................................................ 5
      3.1.1 Testing Procedures and Details ................................................. 5
      3.1.2 Testing Results ........................................................................... 6
      3.1.3 Edge Load Summary................................................................. 7
   3.2 Longitudinal Joint Capacity ............................................................... 7
      3.2.1 Testing Procedures and Details ................................................. 8
      3.2.2 Testing Results .......................................................................... 9
      3.2.3 Longitudinal Joint Summary ..................................................... 14
      3.2.4 Spring Constants for Joint Modelling ....................................... 14
   3.3 Transverse Joint Capacity ................................................................. 16
      3.3.1 Testing Procedures and Details ................................................. 16
      3.3.2 Test Results ............................................................................... 17
      3.3.3 Transverse Joint Test Summary ................................................ 18
   3.4 Composite Deck/Girder Action......................................................... 19
      3.4.1 Testing Procedure and Details................................................... 21
      3.4.2 Test Results ............................................................................... 21
      3.4.3 Composite Girder Test Summary .............................................. 25

4. Prototype Bridge Construction ............................................................. 25
   4.1 Description of Prototype Bridge ....................................................... 26
   4.2 Precast Bridge Deck Characteristics ................................................. 27
      4.2.1 Precast Panel Layout ................................................................. 27
      4.2.2 Joint Details ............................................................................... 28
      4.2.3 Post-tensioning Systems and Prestressing ................................. 28
      4.2.4 Deck to Girder Connection ....................................................... 31
         4.2.4.1 Leveling Bolts and Haunches ............................................ 31
         4.2.4.2 Shear Studs ........................................................................ 32
      4.2.5 Parapet Wall .............................................................................. 33
      4.2.6 Epoxy Overlay ........................................................................... 33
   4.3 Precast Deck Panel Production ......................................................... 34
      4.3.1 Form Blockouts ......................................................................... 34
      4.3.2 Welded Wire Fabric .................................................................. 35
      4.3.3 Prestressing Strand, Ducts, and Reinforcing ............................ 35
4.3.4 Prestress Transfer ................................................................. 36
4.3.5 Storage and Shipping ........................................................... 37
4.4 Bridge Construction Observations ........................................... 38
  4.4.1 Proposed Project Schedule .................................................. 38
  4.4.2 Stage 1 Precast Deck Bridge ................................................. 38
    4.4.2.1 Longitudinal Duct Splicing ........................................... 40
    4.4.2.2 Grouting Transverse Joints ............................................ 41
    4.4.2.3 Longitudinal Post-tensioning Validation Test ................. 42
    4.4.2.4 Longitudinal Post-tensioning Duct Grouting
      and Sealing ................................................................. 44
    4.4.2.5 Closure Pours ........................................................... 44
    4.4.2.6 Grouting Shear Stud Blockouts and Haunches ................ 45
    4.4.2.7 Grinding the Deck Surface ......................................... 45
    4.4.2.8 Epoxy Overlay .......................................................... 45
  4.4.3 Stage 2 Precast Deck Bridge ................................................. 47
    4.4.3.1 Transverse Strand Coupling ........................................ 47
    4.4.3.2 Grouting Joints and Post-tensioning ......................... 47
  4.4.4 Conventional Cast-in-place Deck ......................................... 49

5. Prototype Bridge Evaluation .................................................... 50
  5.1 Cost Analysis ........................................................................... 50
  5.2 Labor Hours Required ........................................................... 52
  5.3 Specialized Equipment ......................................................... 54
  5.4 Subcontractors ..................................................................... 55
  5.5 Contractor Submittals ............................................................ 56
  5.6 Problems/Difficulties Encountered with the
    Precast Deck Bridge ............................................................ 57
    5.6.1 Concrete Mix Design for Precast Deck Panels .................. 57
    5.6.2 Shear Stud Placement ...................................................... 58
    5.6.3 Sealing of Post-tensioning Ducts .................................. 59
    5.6.4 Summary ....................................................................... 60
  5.7 Post Construction Feedback ................................................... 60
    5.7.1 Comments from Precast Deck Manufacturer’s Employees .. 61
    5.7.2 Comments from the Field Construction Team ................ 61
    5.7.3 Comments from WisDOT Personnel ................................ 61
    5.7.4 Design Engineer’s Comments ........................................ 62
    5.7.5 Summary ....................................................................... 62

6. Bridge Testing .......................................................................... 62
  6.1 Load Testing of Conventional Bridge ..................................... 63
  6.2 Load Testing of Precast Deck Bridge .................................... 65
  6.3 Longitudinal Joint Vertical Movement during Grout Hardening ... 66
  6.4 Crack Mapping at Completion of Construction ....................... 67

7. Analytical Modelling ................................................................. 68
  7.1 Description of Bridges Analyzed .......................................... 68
    7.1.1 Culpeper Bridge ........................................................... 69
    7.1.2 Welland River Bridge .................................................... 69
1. Introduction

Aging and deterioration problems associated with bridges are compounded by increasing levels of traffic on roadways. As of December 2005, the Federal Highway Administration (FHWA) deemed 156,177 of 594,616 (26%) total bridges nationwide as structurally deficient. In Wisconsin alone there are 2,262 of 13,687 (16%) total bridges listed as deficient (National Bridge Inventory). Because of these factors, rehabilitation and/or replacement of bridges is necessary; however, the need for reducing traffic congestion due to construction and limiting construction time is also a concern.

According to the FHWA Work Zone Mobility and Safety Program’s Facts and Statistics Page, work zones account for nearly 24 percent of non-recurring congestion; 1,028 fatalities in 2003 resulted from motor vehicles crashes in work zones; and more than 40,000 people are injured in work zone related crashes each year. To try and alleviate some of these problems, the University of Wisconsin – Madison, in conjunction with the Wisconsin Department of Transportation, the FHWA, and the private engineering firm Alfred Benesch & Co., has researched and developed methods for optimizing bridge construction and implemented these methods during the construction of a bridge deck on I-39/90 over Door Creek (structure B-13-161).

Traditionally, highway agencies use cast-in-place methods for building and replacing bridge decks. During this process, traffic must be completely closed off for a long period to allow for forming and curing of the concrete. An alternative to using cast-in-place decks is the use of full-depth precast concrete deck panels. Panels are constructed off-site under controlled conditions and brought to the site ready for installation. Using precast deck panels can significantly reduce construction time, thereby decreasing delays to motorists. It also increases work zone safety by reducing the number of and exposure time of workers operating near moving traffic, reduces environmental impacts by minimizing the site access footprint, and improves the constructability of bridge designs by controlling manufacturing environments.

Two unique innovations were used for the Door Creek Bridge. The bridge was redecked in stages to maintain traffic, so panels were cast at approximately half the width of the bridge. This allowed traffic to move through one side of the bridge while the other side was being constructed. The staging used a unique joint between the precast panels. The joint was also placed in a unique location. Also, the shear studs used to develop composite action between the panels and the girders were placed farther apart than typical AASHTO standards. The wider spacing for this project was accepted based on performance measurements from laboratory testing.

1.1 Objectives

The overall research and design objectives for this precast full-depth deck panel research and development program are as follows:

- To demonstrate the use and effectiveness of rapid installation of innovative full depth prefabricated concrete deck panels in staged bridge deck construction;
- To identify and develop connection details between panels that will prove durable in Wisconsin’s harsh climate;
To introduce prefabrication to local precast manufacturers and obtain their participation in developing efficient detailing;
To create a guide for bridge engineers so that future prefabricated deck panels can be used efficiently for the construction of bridge decks.

To achieve the objectives listed above, goals were set for each phase of research and construction. Some of these goals include the following:

- To determine the edge strength of the deck panels;
- To define needed post-tensioning levels for longitudinal and transverse joints;
- To develop connection details between concrete deck panels and girders;
- To perform a constructability study and compare the construction of the innovative and conventional systems;
- To evaluate structural performance of structures as constructed.
- To conduct analytical studies to identify required prestress amount in joints.

1.2 Scope

IBRC 3 was a multi-stage project completed over a four year span. The three phases of the process are outlined below.

- Phase 1: Complete background research and preliminary engineering required to develop a proposed precast deck system and design procedure for the full depth, precast, prestressed concrete deck panels. Develop laboratory tests based on the proposed system and carry out all testing.
- Phase 2: Implement the full depth precast, prestressed concrete deck panel system on the Door Creek Bridge. Perform a constructability study and compare the construction of the innovative and conventional bridge deck systems. Evaluate structural performance. Determine minimum needed prestress levels across joints for future projects.
- Phase 3: Monitor both conventional and innovative bridge decks. Use non-destructive load tests and inspect for cracking to determine feasibility of using prefabricated, prestressed concrete deck panels in the future. Run analytical studies of loaded bridge.

2. Background Review

2.1 Background

Recently there has been increased attention to accelerated bridge construction techniques to limit road closure time during construction. There are many systems in use that show promise for minimizing road closure time while maintaining bridge stability and strength. Previous researchers have completed surveys of the use of precast full-depth deck systems and parts of the design for the Door Creek bridge system were based on this previous research. Extensive descriptions of full depth precast bridge construction and behavior reviews may be found in Markowski (2005).
2.2 Door Creek Bridge

The bridge for which this research was conducted is part of an existing state owned bridge over Door Creek on Interstate I-90 near the city of McFarland, Wisconsin. Plans for the original bridge are included in Appendix A. Re-construction of the bridge included deck replacement and widening to accommodate an additional lane, making each bridge a 3-lane system. Since I-90 is a divided highway, two bridges were re-constructed (B-13-160 and B-13-161). Plans for the new bridges are shown in Appendix B. Each is an 83’ single span with a 30° skew. The existing bridges were 40’-2” wide; however, the current bridges are 64’-6” to accommodate an extra lane. Three 60” deep steel plate girders were used in addition to the existing five girders on each bridge. A composite concrete slab with an epoxy overlay was used for the deck of the precast prototype bridge.

2.3 Transverse Joints

The transverse joint configuration is a crucial component of the prefabricated system. It was found that all previous projects reviewed used one of the following configurations (Figure 1): female-female joint; male-female joint (i.e. tongue-and-groove); butt-end joint.

![Figure 1. Common joints: a) female-female, b) male-female, c) butt ends](image)

Male-female joint configurations have performed poorly in previous projects (Issa et al 1995, Chi 1985) because of poor fit or poor sealing. Other projects, including the Bayview Bridge (Issa et al 1995) and High Street Overhead (and Issa et al 1995) have utilized a butt-joint between panels. The joint performed satisfactorily for the Bayview Bridge because the cable-stayed precast, prestressed deck is in compression. However, the joint for the High Street Overhead performed poorly, with excessive leakage due poor sealing. A plethora of projects employed the use of female-female joints. It has been noted that not all of the projects with this joint configuration performed satisfactorily. The poor performing joints did not have post-tensioning applied across the joint. Projects that utilized longitudinal post-tensioning tend to perform much better than those that don’t (Chi 1985 and Issa et al 1995, Issa et al 1995, Culmo 2000, Babaei 2001, and Perry 2004). The female-female joint allows for more tolerance in fit up of two adjacent panels.

A post-tensioned, grouted female-female joint was chosen as the best joint candidate for the Door Creek Bridge.
2.3 Longitudinal Joint

Because the Door Creek Bridge was constructed in stages to maintain traffic, there is a longitudinal joint between the two halves of the deck. Two viable options for the placement of the joint were established based on reviewing previous research projects. The first was to leave a 3ft. open strip as access to splice reinforcing bars from two separate precast deck panels and later fill with cast-in-place concrete. The second was to use a similar female-female joint as in the transverse joints. The joint would be post-tensioned after it was filled with high-strength non-shrink grout. Previous projects placed this joint above a girder. When the joint was used above girders it was found that conflicts arose in the need to connect post-tensioning ducts, develop composite action between the deck and girder, and to have a tight grouted joint.

The longitudinal joint selected for the Door Creek Bridge utilizes a post-tensioned female-female joint. The joint is not located over a girder, but between. This was to simplify construction by avoiding the congestion of the girder-to-deck connection and panel connection in the same location. Also, if or when the deck cracks at the joint, it will occur at the bottom of the deck since the joint is placed in the positive moment region. This should reduce the possibility of leakage, making the deck more durable.

2.4 Transverse Post-Tensioning

A staged bridge deck was built in Canada (Seal Island Bridge – Perry 2004) and used transverse post-tensioning to connect the deck panels. A different system was proposed for the Door Creek Bridge in which the stage one panels were fully pretensioned transversely to resist vehicle-induced bending. This allowed traffic to be placed on that half of the bridge without transverse post-tensioning. Half of the transverse pre-tensioning strands were left protruding from the stage one panels. The post-tensioning ducts in the stage two panels were placed to match the locations of the protruding pre-tensioning strands from stage one. Post-tensioning strands were placed in these ducts and coupled to the stage one strands to achieve a prestressed longitudinal joint. An innovation of the Door Creek Bridge is in the design and construction of that longitudinal deck joint.

3. Research Phase Activities – Laboratory Testing

This project required the construction of precast concrete deck panel specimens for a series of serviceability and strength tests to prove the proposed design of the joints. In addition to concrete deck panels, a half scale steel plate girder with deck was constructed in order to investigate the composite beam action between the concrete deck panels and the steel plate girder.

Four different types of concrete panel specimens were prepared for testing. The following Table outlines the quantity, size, and purpose of each specimen.
List of specimens constructed for testing

<table>
<thead>
<tr>
<th>Specimen Category</th>
<th>Size (L x B x thick)</th>
<th>Quantity</th>
<th>Purpose</th>
</tr>
</thead>
<tbody>
<tr>
<td>Edge Strength Panels</td>
<td>8'-9&quot; x 14'-3&quot; x 8&quot;</td>
<td>2</td>
<td>To determine strength of panels when loaded near the edge</td>
</tr>
<tr>
<td>ES-1 &amp; ES-2</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Longitudinal Joint Panels</td>
<td>4'-0&quot; x 13'-3&quot; x 8&quot;</td>
<td>6</td>
<td>To determine joint stiffness under different post-tensioning levels</td>
</tr>
<tr>
<td>LJ-1 through LJ-6</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Transverse Joint Panels</td>
<td>4'-0&quot; x 8'-0&quot; x 8&quot;</td>
<td>4</td>
<td>To determine joint stiffness under different post-tensioning levels</td>
</tr>
<tr>
<td>TJ-1 through TJ-4</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Composite Behavior Panels</td>
<td>4'-0&quot; x 4'-0&quot; x 4&quot;</td>
<td>10</td>
<td>To determine composite behavior of panels and steel beam with different shear stud layouts</td>
</tr>
<tr>
<td>CM-1 through CM-10</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

All testing was completed at the Wisconsin Structures and Materials Testing Laboratory at the University of Wisconsin - Madison. A description of the test purpose, test configuration and test results for each of the test types follows. An in-depth description is in Markowski (2005).

### 3.1 Transverse Edge Loaded Panels

In order to show that the use of full-depth precast, prestressed concrete deck panels can work well in all applications, not just single span bridges or new bridge construction, testing for panel edge strength at the edge of a non-continuous deck panel was completed. The edge must be able to resist vehicle loading during staged construction when only a portion of the deck is replaced and an un-connected joint exists between a portion of the old deck and new precast panels.

#### 3.1.1 Testing Procedures and Details

Two full-scale tests were completed on panels ES-1 and ES-2. Panels ES-1 and ES-2 were both cast at the dimensions listed above. After reviewing the test data from ES-1, however, it appeared that a wide panel was unnecessary because the damage was limited to a small edge region, panel ES-2 was cut down to 5' wide. This facilitated easier handling. Both specimens were cast using Wisconsin DOT Grade D, Size 1 (3/4" maximum aggregate size) concrete mix with a target 4ksi strength. The panels were constructed such that there was concentrated prestressing placed near to and parallel to the loaded edge; this was an attempt to strengthen the edge.

During testing, load was applied at the transverse joint edge of the panel to simulate a wheel load moving onto the panel at that edge as shown in Figure 2. Two full-scale tests were completed by loading at AASHTO service load levels and then increasing to ultimate capacity. The loading was completed in sequential stages. Panels were first subjected to several cycles, with maximum load equaling 20 kips (wheel plus some impact). After cyclic loading, panels were loaded manually and monotonically to 20 kips. Finally, after these cycles, the panels were loaded to failure. Panel ES-1 was tested with the load located between two longitudinal post-tensioning anchors; panel ES-2 was tested with the load directly above one post-tensioning anchor.
3.1.2 Testing Results

After each loading sequence, the panels were checked for concrete cracking. No cracks were found on either the top or bottom surface of either of the panels when loaded at service load levels (20k). When loaded to ultimate capacity, panel ES-1 was subjected to 84 kips applied between two post-tensioning anchorages, at which point it failed abruptly due to punching shear. Panel ES-2 was loaded directly over a post-tensioning anchor as mentioned above, and a load of 42 kips was reached when the top cover over the post-tensioning anchorage block-out broke off. At 86 kips, the panel itself failed abruptly at the edge due to punching shear. The angle at which the punch through failure developed for ES-1 was difficult to determine due to the fact that the crack did not fully propagate through the entire specimen, though the angle at which the failure plane propagates from the top of the slab down the edge was calculated to be approximately 40°. The angle at which the punch failure developed for ES-2 was 27°. The shallower angle at ES2 is expected due to the existence of high compression from the post-tensioning. It would be expected to have reached a higher capacity than ES1, but its strength was affected by the size of the anchorage and the disruption caused by the placement of the anchorage.

Traditionally, highway bridge decks are designed for flexure, not punching shear, even though punching shear has been identified as the likely failure mode for bridge decks continuous over girders (Perdikaris et al., 1989). The reason for this is that flexural behavior of concrete is well understood while punching shear is not. However, the AASHTO LRFD Bridge Design Specifications [2007] does have an empirical design equation for punching shear capacity in slabs and footings that can be found in section 5.13.3.6. According to this section, the nominal punching shear capacity is:

\[
V_{AASHTO} = \left(0.063 + \frac{0.126}{\beta_c}\right) \cdot \sqrt{f'\text{c} \cdot b_{0.5} \cdot d} \leq 0.126 \sqrt{f'\text{c} \cdot b_{0.5} \cdot d}
\]

where
- \(\beta_c\) is the ratio of the long side to the short side of the rectangular load area;
- \(b_{0.5}\) is the critical section perimeter;
- \(d\) is the effective depth;
- \(f'\text{c}\) is the compressive strength of the concrete is ksi.
The perimeter of the critical section is located in such a manner that it is a minimum, but need not approach a distance closer than 0.5d from the edges of the concentrated load area. Unlike other design codes, AASHTO does not provide special provisions for prestressed slabs. Therefore, punching shear capacity is independent of the prestressing force or layout and is simply a function of the concrete strength and loaded area. Similar methods for predicting punching shear have been developed by the American Concrete Institute (ACI), Eurocode2, and the Canadian Highway Bridge Design Code. The following Table outlines the key input values for the AASHTO equation along with the results.

<table>
<thead>
<tr>
<th>Panel</th>
<th>$f'_c$ (psi)</th>
<th>$b_{0.5}$ (in)</th>
<th>Experimental Capacity $V_u$ (kips)</th>
<th>AASHTO Capacity $V_{AASHTO}$ (kips)</th>
<th>$\frac{V_{AASHTO}}{V_u}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>ES-1</td>
<td>5840</td>
<td>52</td>
<td>84</td>
<td>90.7</td>
<td>1.08</td>
</tr>
<tr>
<td>ES-2</td>
<td>5897</td>
<td></td>
<td>86</td>
<td>91.1</td>
<td>1.06</td>
</tr>
</tbody>
</table>

* Compressive strength was measured on day of test.

3.1.3 Edge Load Summary

The ratio of the AASHTO theoretical capacity to the experimental capacity is greater than one for both panels; thus, the AASHTO model is a very slight over-estimation of punching shear capacity for loading at the edge of the slab. This holds true for the ACI, Eurocode2, and the Canadian Highway Bridge Design Code models also.

It is likely for both specimens that the pre-tensioned strands parallel to the edge were fully developed; it is unlikely, however, that the post-tensioning strands were able to create internal concrete prestress in the failure region for the first specimen in which load was applied directly between anchorages. On the other hand, the post-tensioning did create internal prestress where the load was applied directly above the anchorage, but the anchorage itself probably caused a disturbance in the region, resulting in weakness.

The panels have ample wheel load resistance capacity. The load at which the panels fail is approximately 5.3 times greater than the AASHTO service wheel load of 16 kips, proving the panels do have sufficient capacity.

3.2 Longitudinal Joint Capacity

Three full-scale longitudinal joint tests were completed to measure the amount of post-tensioning stress required across the joints to keep them tight under service level vehicle loads. These tests were used to verify analytic models of the joint performance under bending and to determine when bottom joint opening occurs at different post-tensioning levels. In addition, the effects of vibration caused by traffic loading while the joint grout cured were investigated. This was completed to examine the effects, if any, that the movement of one panel edge from traffic on one side of the staged construction could have on the hardening joint grout.
3.2.1 Testing Procedures and Details

The longitudinal joint test panels were fabricated using self-compacting concrete with 3/8” aggregate instead of Wisconsin DOT Grade D, Size 1 (3/4” maximum aggregate size) concrete mix. Since actual joint behavior under different prestressing levels was the focus of these tests, the concrete mix was expected to have little impact on the test results. Prestress for the slabs was applied by 1/2” diameter 270 ksi low-relaxation strands each stressed to 31 kips. The prestressing runs in the 13’-3” direction of the slab, at both top and bottom and creates an internal concrete stress of 0.65ksi. In addition to the prestressing, 6 x 6, W4 x W4 welded wire fabric was added to the top and bottom of the panel for crack control while handling specimens.

As stated above, three tests were carried out to analyze joint performance under bending. A diagram of the test setup is shown in Figure 3. Two of these three tests had a cyclic load placed on one panel for 6 hours immediately after the joint was cast. This was meant to simulate traffic loading while the longitudinal joint grout cures. The panels were placed end-to-end, creating a female-female joint. Since load was applied by a 3 ft. wide bar, only 3ft. of the 4ft. joint length was grouted using Dayton Superior non-shrink grout, thereby creating equal internal force across the entire length of the joint. For the third test the grout was allowed to fully cure before any loading was applied to the specimen.

![Figure 3. Test setup is shown for the longitudinal joint tests with loading actuator.](image)

For all the specimens the joint grout was cured until it reached a minimum strength of 4,000 psi, at which point the specimens were post-tensioned together using the strands mentioned above. After the tendons were stressed, grout was pumped into the post-tensioning ducts filling the entire area of the duct and bonding the strand to the specimens. For the first two tests, the post-tensioning strands were stressed and grouted after the cyclic loading. Each test had a different post-tensioning level, which is outlined in the Table following.

Since the tendons were grouted in place, we were unable to de-stress the tendons as in the edge strength specimens; therefore, load cells could not be used to measure the force in the tendons. Instead, total elongation was measured and the force in the tendons was found using the following equation:

\[
\delta = \frac{P \cdot L}{A \cdot E}.
\]
3.2.2 Testing Results

Panels LJ-1 and LJ-2 did not receive loading until after the joint grout cured and then were initially loaded to 20 kips to measure the initial stiffness and determine when cracking first occurred at the bottom of the joint (Test 1). During the first cycle, the first visible crack occurred at a joint moment of approximately 109 kip-in/ft (see Figure 4). However, the cracking in the bottom joint occurred in a gradual manner; 109k-in/ft is an approximation based on visual evidence and inspection of the load-deflection curve to determine when the loss of stiffness occurred. The maximum deflection recorded during the initial loading was 0.086 inch. The maximum joint opening measured 0.015 inch. It should be noted that there was little to no residual vertical displacement measured at the joint when the specimen was unloaded.

After initial loading, a cyclic load ranging from 0 to 20 kips was applied for 100 cycles at 2 Hz (Test 2). This was done to condition the slab in an attempt to simulate repeated vehicle load and further soften the panels. After conditioning, the panels were again loaded manually to 20 kips (Test 3). Test 1 and Test 3 moment vs. joint opening (proportional to rotation) curves are very similar in terms of slope, as seen in Figure 5.

<table>
<thead>
<tr>
<th>Test Panels</th>
<th>Tendon Force (kip)</th>
<th>Anchor Set @ Dead (in)</th>
<th>Total Elongation (in)</th>
<th>Post-Tensioning Stress Level Concrete (psi)</th>
</tr>
</thead>
<tbody>
<tr>
<td>LJ-1 &amp; LJ-2</td>
<td>1 32.8</td>
<td>0.375</td>
<td>1.6875</td>
<td>358.7</td>
</tr>
<tr>
<td></td>
<td>2 32.8</td>
<td>0.3125</td>
<td>1.6875</td>
<td>358.7</td>
</tr>
<tr>
<td></td>
<td>3 32.8</td>
<td>0.25</td>
<td>1.9375</td>
<td>37.7</td>
</tr>
<tr>
<td>LJ-3 &amp; LJ-4</td>
<td>1 31.6</td>
<td>0.625</td>
<td>1.625</td>
<td>337.5</td>
</tr>
<tr>
<td></td>
<td>2 30.4</td>
<td>0.6875</td>
<td>1.5625</td>
<td>337.5</td>
</tr>
<tr>
<td></td>
<td>3 35.2</td>
<td>0.4375</td>
<td>1.8125</td>
<td>35.2</td>
</tr>
<tr>
<td>LJ-5 &amp; LJ-6</td>
<td>1 32.8</td>
<td>0.3125</td>
<td>1.6875</td>
<td>232</td>
</tr>
<tr>
<td></td>
<td>2 34</td>
<td>0.25</td>
<td>1.75</td>
<td>232</td>
</tr>
</tbody>
</table>

Figure 4. Initial cracking in the joint between panels LJ-1 & 2.
Figure 5. Applied joint moment plotted with joint opening for LJ1 & 2.

After Test 3, the panels were intended to be loaded to their ultimate capacity. Since the panels were not restrained in any way at the outside supports, the panels were free to lift off the supports and flex at the joint location, acting similar to a hinge. Therefore, achieving failure was very difficult. Instead, the panels were loaded until the vertical deflection measuring LVDT’s went out of range (~0.75 inches) and/or there was a significant opening at the bottom of the joint. Maximum deflection for this test was recorded at 0.797 inch when the LVDT went out of range at a joint moment of 203 k-in/ft. A maximum measured strain of 1,581 microstrain in compression at the top surface was recorded at the same joint moment. The maximum recorded joint opening using standard instrumentation on the bottom was 0.18 inch and was recorded at a joint moment of 196 k-in/ft; however, the specimen was loaded beyond this and the joint opened up to 0.25 inch as checked by a tape measure. The joint opening returned to almost zero when the panels were unloaded.

A final load sequence was performed in which the outside west support was tied down and the panels were loaded until failure. This loading condition is meant to investigate the shear capacity of the joint; however, the actual failure mode was cracking on the top surface of the panel over the inside western support due to negative moment. First cracking began over the support at a moment of approximately -94 k-in/ft. In addition to the cracks over the support, cracking began on the northeast side of the shear key around 44 kips (8k/ft). The Table below summarizes the results of these tests.

<p>| Summary of data recorded over sequential loading of panels LJ-1 and LJ-2 |
|-----------------------------|-----------------------------|-----------------------------|-----------------------------|
| Load                        | Joint Opening (in)          | Vertical Deflection (in)    | Top Joint Strain (microstrain)*** |
| Test 1**                    | 0.0013                      | 0.019                       | -150                         |
| (M-73 k-in/ft)              |                             |                             |                               |
| Max                         | 0.015                       | 0.0857                      | -595                         |
| (M=135.5 k-in/ft)           |                             |                             |                               |</p>
<table>
<thead>
<tr>
<th>Test 3</th>
<th>Service (M=73 k-in/ft)</th>
<th>0.0022</th>
<th>0.029</th>
<th>-160</th>
</tr>
</thead>
<tbody>
<tr>
<td>Max    (M=135.5 k-in/ft)</td>
<td>0.013</td>
<td>0.0784</td>
<td>-491</td>
<td></td>
</tr>
<tr>
<td>Test 4</td>
<td>Service (M=73 k-in/ft)</td>
<td>0.0027</td>
<td>0.034</td>
<td>-220</td>
</tr>
<tr>
<td>Mac    (M=135.5 k-in/ft)</td>
<td>0.25****</td>
<td>0.797</td>
<td>-1581</td>
<td></td>
</tr>
<tr>
<td>Test 5</td>
<td>Service (M=73 k-in/ft)</td>
<td>0.008</td>
<td>0.0117</td>
<td>-21</td>
</tr>
<tr>
<td>Max    (M=347 k-in/ft)</td>
<td>0.18</td>
<td>0.79</td>
<td>-600</td>
<td></td>
</tr>
</tbody>
</table>

*Test 1 refers to initial loading, Test 3 refers to loading after cyclic loading, Test 4 refers to peak loading, and Test 5 refers to loading in which outside west support is tied down.

**Max refers to the peak load applied to the specimens, service refers to equivalent service load level. Service load is based on an unfactored HS-20 truck loading with impact.

***Negative value denotes compression strain.

****Measured with tape measure, not LVDT.

Testing on panels LJ-3 and LJ-4 utilized a cyclic load in the eastern exterior span while the grout in the joint cured. A cyclic load ranging from 0 to 9.9 kips was applied for 6 hours of joint cure, during which time joint displacements were measured using a dial gauge at the onset of loading, 3 hours into loading, and at the end of loading. The following Table displays the joint displacements measured during at those intervals.

**Joint displacement measured curing cyclic loading with grout curing on panels LJ-3 and LJ-4**

<table>
<thead>
<tr>
<th>Time (hrs)</th>
<th>Joint Displacement (in)</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
<td>0.015</td>
</tr>
<tr>
<td>3</td>
<td>0.011</td>
</tr>
<tr>
<td>6</td>
<td>0.008</td>
</tr>
</tbody>
</table>

Just over one hour into the cyclic loading, small cracks started forming at the top of the joints between the grout and the precast piece, which propagated through the joint over the course of the loading. However, no cracks were visible in the bottom of the joint. Cracks are shown in Figure 6.

![Figure 6](image.png)

Figure 6. Cracks in the southern and northern ends of the joint on the top surface during cure.
(Note: cracks have been marked with a pen to enhance visibility.)
After the grout cured, post-tensioning tendons were stressed and the ducts were grouted. The grout was allowed to cure for two days, after which the specimen was loaded to 20 kips. The first visible crack opening in the bottom of the joint occurred at a moment of approximately 91.6 k-in/ft. Similar to panels LJ-1 and LJ-2, cracking occurred in a gradual manner; 91.6 k-in/ft is an approximation based on visual evidence and inspection of the load-deflection curve to determine when loss of stiffness occurs. Maximum joint opening measured during this test was 0.0355 inch. Also analogous to testing on panels LJ-1&2, a cyclic load was applied to the specimen after the initial loading ranging from 0 to 20 kips for 100 cycles at 2 Hz. After conditioning (cyclic loading), the panels were manually loaded to 20 kips. Panels were then loaded to ultimate capacity, or when the LVDT's went out of range and/or there was a significant opening at the bottom of the joint. The test was terminated when both the deflection LVDT and the LVDT measuring joint opening went out of range. Then, the final load sequence was performed, during which the outside western support was tied down and the panels were loaded until failure. Like panels LJ-1&2, the panels over the inside western support due to negative moment. Results were comparable to panels LJ-1&2 and are shown in the following Table. Final failure occurred as flexural cracking in the panel, under negative moment at the support, developed into a shear crack.

### Summary of data recorded over sequential loading of panels LJ-3 and LJ-4

<table>
<thead>
<tr>
<th>Load</th>
<th>Joint Opening (in)</th>
<th>Vertical Deflection (in)</th>
<th>Top Joint Strain (microstrain)***</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Service</strong> (M-73 k-in/ft)</td>
<td>0.0012</td>
<td>0.021</td>
<td>-90</td>
</tr>
<tr>
<td>Test 1* Max</td>
<td>0.035</td>
<td>0.127</td>
<td>-411</td>
</tr>
<tr>
<td><strong>Service</strong> (M-73 k-in/ft)</td>
<td>0.0038</td>
<td>0.036</td>
<td>-86</td>
</tr>
<tr>
<td>Test 3 Max</td>
<td>0.0331</td>
<td>0.127</td>
<td>-362</td>
</tr>
<tr>
<td><strong>Service</strong> (M-73 k-in/ft)</td>
<td>0.0039</td>
<td>0.038</td>
<td>-98</td>
</tr>
<tr>
<td>Test 4 Max</td>
<td>0.19</td>
<td>0.766</td>
<td>-836</td>
</tr>
<tr>
<td><strong>Service</strong> (M-73 k-in/ft)</td>
<td>0.0097</td>
<td>0.0273</td>
<td>-23</td>
</tr>
<tr>
<td>Test 5 Max</td>
<td>0.18****</td>
<td>0.8****</td>
<td>-398</td>
</tr>
</tbody>
</table>

*Test 1 refers to initial loading. Test 3 refers to loading after cyclic loading. Test 4 refers to peak loading, and Test 5 refers to loading in which outside west support is tied down.

**Max refers to the peak load applied to the specimens in the test, service refers to equivalent service load level. Service load is based on unfactored HS-20 truck loading with impact.

***Negative value denotes compression strain.

****Measured with tape measure, not LVDT.

Panels LJ-5 and LJ-6 were loaded in the same manner as LJ-3&4. The Table below displays the joint displacements during the first series of cyclic loading as the grout cured.
Joint displacements measured during cyclic loading with grout curing on panels LJ-5 and LJ-6

<table>
<thead>
<tr>
<th>Time (hrs)</th>
<th>Joint Displacement (in)</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
<td>0.017</td>
</tr>
<tr>
<td>3</td>
<td>0.012</td>
</tr>
<tr>
<td>6</td>
<td>0.007</td>
</tr>
</tbody>
</table>

After the post-tensioning tendons were stressed and the ducts were grouted, a manual load of 20 kips was applied. The first visible cracks started forming at an approximate moment of 75.0 k-in/ft. Like the first four panels, cracking occurred in a gradual manner. Unlike the first four panels, panels LJ-5 and LJ-6 were loaded directly until the top LVDTs went out of range (after conditioning), which was at a joint moment of 173.2 k-in/ft. After this, the panels were again loaded until the instrumentation went out of range. This occurred at a joint moment of 156 k-in/ft.

Finally, a last load sequence was performed, during which the outside western support was tied down and the panels were loaded until failure. The failure mode for panels LJ-5 and LJ-6 was flexural cracking over the western inside support. Flexural cracks were first noticed on the top surface of the panel at a support moment of -56.3 k-in/ft and were measured to be 0.01 inch wide. Loading was terminated at a support moment of -101.0 k-in/ft. The following Table summarizes the results of these tests.

Summary of data recorded over sequential loading of panels LJ-5 and LJ-6

<table>
<thead>
<tr>
<th>Load</th>
<th>Joint Opening (in)</th>
<th>Vertical Deflection (in)</th>
<th>Top Joint Strain (microstrain)*****</th>
</tr>
</thead>
<tbody>
<tr>
<td>Test 1*</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Service** (M=73 k-in/ft)</td>
<td>0.0016</td>
<td>0.019</td>
<td>156</td>
</tr>
<tr>
<td>Max (P=21.5 k)</td>
<td>0.125</td>
<td>0.389</td>
<td>-1063</td>
</tr>
<tr>
<td>Test 3</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Service (M=73 k-in/ft)</td>
<td>0.022</td>
<td>0.042</td>
<td>-248</td>
</tr>
<tr>
<td>Max (P=24.5 k)</td>
<td>0.19</td>
<td>0.836</td>
<td>-1449</td>
</tr>
<tr>
<td>Test 4</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Service (M=73 k-in/ft)</td>
<td>0.0151</td>
<td>0.0386</td>
<td>-147</td>
</tr>
<tr>
<td>Max (P=28 k)</td>
<td>0.19</td>
<td>0.81</td>
<td>-1221</td>
</tr>
<tr>
<td>Test 5</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Service (M=73 k-in/ft)</td>
<td>0.0119</td>
<td>0.00038</td>
<td>-10</td>
</tr>
<tr>
<td>Max (M=375.6 k-in/ft)</td>
<td>.18****</td>
<td>.8****</td>
<td>-618</td>
</tr>
</tbody>
</table>

*Test 1 refers to initial loading, Test 3 refers to loading after cyclic loading, Test 4 refers to peak loading, and Test 5 refers to loading in which outside west support is tied down.

**Max refers to the peak load applied to the specimens, service refers to equivalent service load level. Service load is based on unfactored HS-20 truck loading with impact.

***Negative value denotes compression strain.

****Measured with tape measure, not LVDT.
3.2.3 Longitudinal Joint Summary:

For all of these tests, it was more beneficial to relate the point at which the joint first cracked to a moment per foot of width rather than a test load. It gives a better indication of the joint behavior in a real bridge application. The joint cracking moments are compared to the service load moment, plus impact of 1.3, based on an HL-93 truck (HS-20). The Table below displays the joint moments at which each panels started cracking.

Moments per foot of width at which first cracking occurs in the bottom of the longitudinal joint

<table>
<thead>
<tr>
<th>Test Panel</th>
<th>First Crack @ Bottom Joint Moment (k-in/ft)</th>
<th>Prestress in the joint grout across the joint (psi)</th>
<th>Longitudinal Joint Service Load Moment with Impact Factor (k-in/ft)</th>
<th>Factor of Safety for Joint Cracking Versus HS-20 Truck Moment</th>
</tr>
</thead>
<tbody>
<tr>
<td>LJ-1&amp;2</td>
<td>109</td>
<td>359</td>
<td>73.0</td>
<td>1.5</td>
</tr>
<tr>
<td>LJ-3&amp;4</td>
<td>91.6</td>
<td>338</td>
<td>73.0</td>
<td>1.25</td>
</tr>
<tr>
<td>LJ-5&amp;6</td>
<td>75.0</td>
<td>232</td>
<td>73.0</td>
<td>1.03</td>
</tr>
</tbody>
</table>

From all the tests, the following can be concluded:

Panels LJ-1 and LJ-2
- First cracking at the joint with moment of 109in-k/ft;
- Joint fully “softens” at approximately 20-21 kips vertical load, or a joint moment of 138.8 k-in/ft;
- Top surface cracking over west inside support occurs at a support moment of -94 k-in/ft;
- Shear cracking in the joint occurs at a load of 44 kips (~8k/ft of width);

Panels LJ-3 and LJ-4
- Slab became twice as stiff as grout hardened;
- Cracking developed in grout after one hour of set time;
- Bottom joint crack occurred at a moment of 91.6 k-in/ft (test load P=15 kips);
- Top surface cracking over west inside support occurs at a support moment of -84.6 k-in/ft;

Panels LJ-5 and LJ-6
- Slab became twice as stiff as grout hardened;
- First cracking in bottom of joint occurred at a joint moment of 75.0 k-in/ft;
- Some softening is apparent after cyclic loading;
- Upon initial loading, joint softens at a test load of 17 kips or 101k-in/ft, but after cyclic loading, softening develops at a test load of 14 kips, or a joint moment of 83.0 k-in/ft;
- Top surface cracking over inside west support occurs at a support moment of -56.3 k-in/ft.

3.2.4 Spring Constants for Joint Modelling

The moment-curvature relationships established during testing can be used to determine rotational spring constants of a joint at different post-tensioning levels. These constants can be
used to create a finite-element model of the longitudinal joint and the bridge structure as a whole, which will give a better understanding of the bridge behavior and can be compared to future load testing.

To determine spring constants in the linear elastic range, a procedure developed by Priestly (1992) was used to determine the “yield” curvature.

To determine spring constants in the linear elastic range, a procedure developed by Priestly (1992) was used to determine the “yield” curvature.

![Graph showing moment-rotation results with Priestley stiffness and initial slope stiffness.](image)

Figure 7. The moment-rotation results from LJ5&6 are shown with the Priestley stiffness and the initial slope stiffness. (Note: moment is k-in/ft width)

The Priestly method to determine spring constants is usually a conservative approximation, so a more realistic value was also calculated by taking the slope of the moment-curvature plot up to the design service moment as seen in Figure 7. The Table below shows the rotational spring constants calculated using the initial slope as well as the Priestly method.

<table>
<thead>
<tr>
<th>Longitudinal Joint Test</th>
<th>Prestress Across Joint (psi)</th>
<th>Rotational Spring Constant</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Initial Slope (k-in/ft/rad)</td>
<td>Priestly Method (k-in/ft/rad)</td>
</tr>
<tr>
<td>1&amp;2</td>
<td>359</td>
<td>437,414</td>
</tr>
<tr>
<td>3&amp;4</td>
<td>338</td>
<td>411,893</td>
</tr>
<tr>
<td>5&amp;6</td>
<td>232</td>
<td>367,000</td>
</tr>
</tbody>
</table>

The rotational spring constants shown in the Table are only for the linear elastic portion of the moment curvature plots. The design moment for the Door Creek Bridge is within the linear elastic range; however, it may be beneficial for finite-element modeling to allow the joint to act inelastically. For this, another rotational spring constant should be used and can be estimated by taking the slope of the inelastic portion of the individual moment-curvature plots. The next Table shows calculated spring constants for the inelastic behavior and Figure 8 compares the values.
with the actual response. In addition, the Table gives approximate moment values where the behavior of the joint begins to experience a loss of stiffness, denoted by $M_{\text{loss}}$. These values are a “best guess” judgment and should be used carefully when generating computer models.

### Rotational spring constants for inelastic range

<table>
<thead>
<tr>
<th>Longitudinal Joint Test</th>
<th>Prestress Across Joint (psi)</th>
<th>$M_{\text{loss}}$ (k-in/ft)</th>
<th>Inelastic Rotational Spring Constant</th>
</tr>
</thead>
<tbody>
<tr>
<td>1&amp;2</td>
<td>359</td>
<td>118</td>
<td>4230</td>
</tr>
<tr>
<td>3&amp;4</td>
<td>338</td>
<td>96</td>
<td>4104</td>
</tr>
<tr>
<td>5&amp;6</td>
<td>232</td>
<td>73</td>
<td>7738</td>
</tr>
</tbody>
</table>

Figure 8. Comparison of model stiffnesses (blue) with actual LJ5&6 behavior.

### 3.3 Transverse Joint Capacity

#### 3.3.1 Testing Procedures and Details

A simply supported single-span deck strip configuration was used for these tests with the female-female transverse joint occurring at mid-span as shown in Figure 9. Panels TJ-1 and TJ-2 were 8 inch thick slabs measuring 4’-0” x 8’-0”. Both specimens were cast using a self-consolidating (3/8” max. aggregate size) concrete mix.
Prestressing tendons for the slabs consisted of ½” diameter 270 ksi low-relaxation strands each stressed to 31 kips creating 0.65ksi of compression in the precast panel. These strands run in the 8’-0” direction of the slab, both top and bottom. While this prestressing is not needed in these panels for structural strength, they were constructed at the same time as the longitudinal joint panels and were cast in the same casting bed for ease of fabrication. Since the prestressing did not cross the joint, it had no impact on the measured joint behavior. Also, 6 x 6 W4 x W4 welded wire fabric was placed on both top and bottom for easy handling of the specimens. One post-tensioning duct was placed in the 8’-0” direction along the centerline to create prestress across the joint after the grout had hardened. One end of the slab had an anchorage assembly and the other had a female joint key cast into it with a block out to provide access to the duct.

During the test, two panels were placed end-to-end to create a 4ft wide female-female joint. A single-span configuration was used, with the joint occurring at the mid-span, and the load was placed at the edge of this transverse joint, just to one side. After the specimens were set, the joint was cast using Dayton Superior Non-Shrink grout. When the grout strength reached 4000 psi, the panels were post-tensioned together using two 0.6” diameter 270 ksi low-relaxation strands. The total force applied across the joint was 44.3 kips, creating a 154 psi stress across the joint.

There were two stages to the loading sequence. First, a conditioning stage loaded the panels monotonically and manually up to 10 kips, then 100 cycles ranging from 0 to 10 kips at 1 Hz were applied. After the cycles, one final monotonic and manual load up to 10 kips was applied. Then the slab was loaded to failure and the data acquiring instrumentation (LVDTs) went out of their measuring range.

3.3.2 Test Results

Visible cracking was first observed at a joint moment of 62.4 k-in/ft. The cracking in the bottom joint occurred in a gradual manner; 62.4 k-in/ft is an approximation of the force based on visual evidence and the loss of stiffness as noted on the load-deflection curve. Two LVDTs were placed across the bottom of the joint to measure joint opening. Displacements were first detected at a joint moment of 50 k-in/ft and the maximum joint opening measured 0.182” during test 1 (conditioning).
The cracks widths are plotted with applied load in Figure 10. It should be noted that very little residual crack opening remains after the load is removed – indicating that a bridge deck joint could be expected to close even if it received a large overload. On repeated reloading after the initial cracking, the crack returns to its starting position on release of the load.

Maximum vertical deflection was recorded during test 1 as well, with a value of 0.802”. The Table on the following page displays a summary of recorded test data.

3.3.3 Transverse Joint Test Summary

When post-tensioned to 153 psi the transverse joint can resist just over 60 in-k/ft along the length of the joint. The joint is initially rigid, but the stiffness drops to approximately 1660 in-k/rad after the joint cracks. The post-tensioning across the joint, however, is very effective at reclosing the joint again if an overload might cause initial cracking and opening.

| Summary of data recorded over sequential loading for transverse joint tests |
|---|---|---|---|
| Load (k-in/ft) | Joint Opening (in) | Vertical Deflection (in) | Top Joint Strain (με)** |
| Test 1 | Service* 73 | 0.048 | 0.107 | -310 |
| Test 1 | Max* 88.8 | 0.184*** | 0.802 | -823 |
| Test 2 | Service* 73 | 0.173 | 0.470 | -365 |

Figure 10. Plots of load vs. crack opening for the 3 sequences in the transverse test.
<table>
<thead>
<tr>
<th>Test 3</th>
<th>Max* 88.8</th>
<th>0.180***</th>
<th>0.790</th>
<th>-634</th>
</tr>
</thead>
<tbody>
<tr>
<td>Service* 73</td>
<td>0.178</td>
<td>0.570</td>
<td>-357</td>
<td></td>
</tr>
<tr>
<td>Max* 88.8</td>
<td>0.180***</td>
<td>0.800</td>
<td>-571</td>
<td></td>
</tr>
</tbody>
</table>

Note: Test 1 refers to initial loading, test 2 refers to loading after cycling, test 3 refers to peak loading.

Note: Max load refers to the peak load applied to the specimens during that test step; service load refers to equivalent service load level.

*Service load based on HS-20 truck with impact.

**Negative value denotes compression strain.

***Measured with tape measure, not LVDT.

3.4 Composite Deck/Girder Action

A final test was devised to determine if the deck could be attached to girders with composite fasteners at a spacing wider than AASHTO standards. The testing was conducted on a half-scale model of the Door Creek Bridge with the same material properties. The deck panels on one half of the beam span had shear stud block-outs spaced at 2’ center-to-center while the other side of the beam had shear stud block-outs spaced at 1’ c-c (4’ and 2’ spacing in a full scale bridge, respectively) as in Figure 11.

Figure 12 shows the top concrete “flange” of the composite girder with 4ft x 4ft by 4 inch thick precast panels. Each of the panels had female joint configurations to emulate normal transverse joints. The blockouts for the studs are apparent. The blockouts for connecting the longitudinal post tensioning ducts have already been grouted.

After grouting the transverse joints, two 0.6 inch 270 ksi strands were used to create prestress in the top flange or bridge deck. Only after the strands had been post-tensioned were the stud blockouts grouted to develop composite action between the deck and steel girder. Thus, no prestress was inadvertently applied to the steel girder. The joints were prestressed to a uniform compression level of 256 psi.

Prior to testing, the beam was supported on half-round rocker bearings. The completed girder, ready for loading at midspan, is shown in Figure 13.
Figure 11. Model steel girder without deck: a) stud groups at 2 ft. spacing, and b) stud groups at 1 ft spacing.

Figure 12. Top flange of composite girder after post-tensioning duct blockouts have been grouted, before grouting stud blockouts.

Figure 13. Completed composite model girder ready for midspan load testing.
3.4.1 Testing Procedure and Details

The loading for the composite girder testing consisted of two separate stages. The first stage was comprised of accelerated fatigue testing to 2 million cycles of load. The second stage involved loading the beam to its capacity.

**Fatigue loading:** AASHTO LRFD load factors, load distribution coefficients, and a LL impact factor of 15% for the prototype bridge were used to design the fatigue load level for the test model. The midspan point loading on the model bridge was selected to duplicate the stresses from the internal fatigue moment expected in the prototype bridge under truck loading. The correct fatigue stress simulation required a cyclic load of 9.1 kips at midspan. This load was superimposed on a static load of 6 kips, creating a load range of 6 to 15.1 kips.

The fatigue loading cycles were applied in five groups of 400,000 cycles each. At the end of each of these groups a static load was applied and girder behavior measured to determine whether any stiffness deterioration had developed.

**Capacity loading:** The planned sequence of capacity load testing was: 1) load to 25k and unload, 2) load to 50k and unload, 3) load to 75k and unload, 4) load to 100k and unload, then repeat loading to 100k a second time to see rebound strain. During the actual test the capacity of the actuator and load frame, 100k, was reached before a beam failure occurred, but the beam had a permanent deflection which indicating the yielding had occurred.

3.4.2 Test Results

**Fatigue tests:** The effect of the cyclic loading on the girder is best understood by inspecting the test results in plotted in Figure 14 for the static loading conducted after every group of 400,000 cycles of fatigue load. The response of the beam remains extremely consistent with no degradation of stiffness evident. Stiffness loss would be associated with fatigue damage or material deterioration. It appears that 2 million cycles of fatigue level loading has no effect on the girder, regardless of whether the composite spacing is the AASHTO expected distance or the double the AASHTO limit as used on one half of the beam span.

The maximum strain measured in the beam after 2 million cycles of load was 268 microstrain in Figure 14, very close to the value on the initial load cycle of 267 microstrain. The degree of composite action appears to stay perfectly constant based on these strain values.

The conclusion from this test is that 2 million cycles of loading has no effect on the behavior of the beam. The wider spacing of the stud groups does not affect the beam behavior in this range of cycles.
Figure 14. Comparison of the strain induced by loading at different stages of the 2 million load repetition fatigue testing sequence.

Figure 15. Strain in bottom flange of steel girder at midspan with composite girder tests.

*Capacity loading:* The beam was loaded and unloaded repeatedly to increasing load levels during this test. Based on static loading to 50 kips, the beam was clearly still in an
elastic range and the effective stiffness was calculated as 70k/inch of deflection. This stiffness would be associated with an effective moment of inertia of 5844in$^4$. This measured stiffness and inertia indicated that the girder had 95% of the theoretical fully composite section properties. Regardless of the stud spacing in the girder to deck connection, very good composite action is developed in the elastic range of the girder response.

Figure 15 shows how the response of the beam changes as the load was increased above the 50k level. Based on coupon testing of the beam steel, the yield strain was measured as 0.00158in./in. This measured yield strain agrees well with the strain data from the bottom flange of the girder when yield appears to start as indicated in Figure 15 during the 75k load sequence.

The actual strain measurements in the beam, taken 5 ft. on either side of midspan, show slightly different response conditions. This might indicate that the stud spacing is affecting the beam behavior, since the stud spacing was different in the two halves of the beam. The results, however, are counterintuitive. The side of the beam with closer stud group spacing had slightly higher steel strain, which would be associated with greater curvature and less stiffness. It would be expected that the closer AASHTO required spacing would provide better composite action and a higher stiffness.

Slip between the concrete deck and the steel girder was also measured. Figure 15 shows the load vs. slip behavior at the end of the beam where the stud groups were at the wider spacing. Increasing slip is apparent at a midspan load of 55k. This was judged to be a result of loss of shear bonding between the concrete haunch and the steel beam. This is the level at which the steel studs actually start working to transfer shear force. Actual “first slip”, however, is considered to be at a load of 65k when a significant length of the steel-concrete interface loses bond.

At the end of the beam with a closer stud group spacing (1 ft.), the beam appeared to retain composite behavior to higher loads. Slip measurements at this end of the beam showed the first movement and “first slip” to occur at a load of 75k, compared to 65k at the other end.

Rather than comparing the load at which slip occurs, it is wiser to inspect for the moment at which slip occurs and how this is related to moments in the prototype bridge. In the lab, slip was first measured at the east end of the girder span with an equivalent moment in the full scale bridge of 63,440 in-k. Slip at the west end developed at a moment equivalent to 73,200 in-k in the real bridge. The design service load moment in the full scale bridge is 26,062 in-k including impact with an HL-93 truck. The safety factor against slip from the east end results is 2.43. This shows that the shear stud spacing provides good composite action.
Upon completion of load testing with the composite girder, the girder flange was disassembled in an exploratory manner by creating saw cuts at critical locations. One cut was made down the centerline of the eastern-most concrete panel up to the first stud blockout. Then a transverse cut was performed across the full panel width allowing the condition of the blockout to be explored.

There was no visible deformation of the shear studs at this blockout, which would be expected if significant slip between the steel beam and concrete flange had developed.
When examining the non-shrink grout placed in the blockout we found a definite visible horizontal crack over the tops of the steel studs as in Figure 16. Having all the heads of the shear studs at a single level creates a plane of weakness.

3.4.3 Composite Girder Test Summary

The precast full depth bridge deck can be made composite with the steel girders using a connector spacing that is double the AASHTO maximum limit of 24 inches. After experiencing more than 2 million cycles of service level wheel loading, the deck/girder system exhibited virtually no change in its load resisting characteristics.

A high level of composite action is attainable, even with an unusually wide connector spacing between the deck and the steel girder. With loads in the elastic range of response, which is more than 2.25 times the service loading, the system displayed 95% of the theoretical full composite action with a plane section analysis assumption.

As the composite girder is loaded to capacity, horizontal shear slip initiates slightly earlier when the composite connectors are at a larger spacing. With the AASHTO maximum spacing, the beam reached 15% more capacity before slip starts. Still, slip does not start with either connector scheme until the loads reach nearly 2.4 times service load.

4. Prototype Bridge Construction

The project included specific tasks associated with the design and construction of a prototype bridge which can be summarized by the following research objectives.

1. Perform a constructability study and compare the construction of the innovative precast deck with a conventional cast-in-place system.
2. Evaluate structural performance of the structure as constructed.

In order to perform the constructability study and compare the construction of the innovative and conventional systems, direct observations were made on the construction process. Activities performed by each construction crew member as well as the tools and equipment required for each activity were recorded so that they could be tallied and compared for each structure.

In order to evaluate the structural performance of the structures as constructed, the panels and conventional bridge were instrumented and tested under load. Strain and displacement data were measured in order to evaluate the composite action between the deck and the girders and the transverse bridge deck bending behavior.
4.1 Description of Prototype Bridge

The bridge selected for implementation of the full depth precast deck system is located on westbound Interstate Highway 39/90 over Door Creek near McFarland, Wisconsin (Pleasant Springs Township, Dane County), as shown in Figure 17, and was constructed by WisDOT. There is a nominally identical companion structure on the eastbound roadway. The eastbound companion structure was reconstructed as part of the same project using traditional formwork and cast-in-place concrete. The conventionally constructed eastbound bridge is structure number B-13-160, and the westbound prestressed deck bridge is B-13-161.

![Figure 17. Location of the prototype bridge on I-90/39 westbound.](image)

Each bridge was an 83’-0” long single span structure with a 30° left skew. The existing bridges originally were 40’-2” wide, however, part of the project was to widen the bridges to 64’-6”. The bridges originally each had five 60” deep steel plate girders spaced at 8’-10” on center; three additional girders were added to each bridge at 7’-6” centers to accommodate the widening.

The haunch between the girders and the bridge deck varied between 1” to 3” to adjust for camber in the girders. Both bridges utilized headed shear studs to achieve composite beam action. Parapets for each structure are standard “LF” sloped face parapets constructed from conventionally field formed and steel reinforced concrete.

Both bridges were constructed in stages in order to maintain two lanes of traffic on the roadways at all times. This required that a longitudinal construction joint be present. In
the precast deck bridge the panels were post-tensioned together in place in both the longitudinal and transverse directions. The deck panels were designed to act compositely with the girders. Finally, an epoxy overlay was installed to provide a uniform driving surface and seal the deck for increased durability.

4.2 Precast Deck Bridge Characteristics

4.2.1 Precast Panel Layout
The overall precast panel layout for the bridge is shown in Figure 18. Each construction stage contained 10 trapezoidal shaped deck panels placed in a manner to keep half of the bridge open to traffic. The trapezoidal shape was selected to accommodate a 30º skew angle of the bridge while homogenizing panel shape. Stage 1 panels measured 34'-7 11/16" long, 6'-10 7/8" wide, 83/4" deep, and weighed 27,100 pounds. Stage 2 panels measured 39'-10 1/16" long, 6'-10 7/8" wide, 8 3/4" deep and weighed 31,100 pounds. The four end panels (two in each stage) had similar dimensions, but were different in that they had longitudinal post-tensioning anchorages and extra reinforcing bars to tie the panels to the end closure pours.

![Figure 18. Layout of precast deck panels for each construction stage.](image)

The longitudinal staging joint was located between girders (as opposed to over a girder). In the event cracking does occur at the joint, since it is located in a positive bending moment region, the cracks will form at the bottom of the slab. This should limit the ingress of salt solutions and leakage along the joint and will improve the durability of the deck. If the joint were placed over a girder, extra detailing would be required to avoid conflicts between the shear studs on the girder and the coupling hardware in the panels. This would create unwanted congestion at the joint and make the detail more difficult to
construct. A joint over the top of the girder might also allow undesirable salt intrusion to the girder itself if top deck cracking in the negative moment region developed.

The longitudinal joint was also the location selected for breaking the 2% cross slope of the deck, for drainage purposes, into two separate directions. The precast panels themselves were constructed with a flat top surface.

4.2.2 Joint Details
The transverse and longitudinal joints were both designed with the same detailing. A cross section profile through a joint is shown in Figure 19. The female-female joint type was selected based on favorable reviews of use on other similar projects. The contractor utilized expanding foam to seal the bottom of the joints prior to placing non-shrink grout in the joint. The foam was installed from the bottom side and typically penetrated and blocked off approximately the bottom 3/4-in of the joint.

![Figure 19. Cross section detail through the joints.](image)

4.2.3 Post-tensioning Systems and Prestressing
The overall bridge pre-and post-tensioning systems are depicted schematically in the plan view of Figure 20. The Door Creek Bridge had 185 psi of prestress applied across the longitudinal joint between deck panels and 250 psi across the transverse joints.

The stage 1 panels were constructed fully pre-tensioned with 12 prestressing strands per panel running in the transverse bridge direction as shown in Figure 21. The transverse prestressing was designed to resist panel stresses due to handling, transportation, and placement, as well as vehicle induced bending. The tendons were 0.6in. diameter 270 ksi low-relaxation 7 wire strand.

Half of the prestressing strands in the stage 1 panels were left with “tails” protruding from the panels in order to splice them to stage 2 transverse post-tensioning strands. These were the strands located directly above one another in Figure 21. In order to
accommodate longitudinal post-tensioning after placement in the field, the panels were constructed with 1” diameter plastic ducts running the length of the bridge.

Figure 20. Isometric view of the bridge with prestressing components shown.

Figure 21. Layout of transverse pretensioned strands in stage 1 panels.

Figure 22. Layout of transverse pre and post tensioned strands in stage 2 panels.
The stage 2 panels were constructed with only half the number of pre-tensioned strands as present in the stage 1 panels. In place of 6 of the 12 pre-tensioned strands, 1” diameter plastic post tensioning ducts were installed with metal end anchorages as shown in Figure 22. The prestressing strand “tails” left protruding from the stage 1 panels were spliced onto post-tensioning strands running through the corresponding transverse ducts in the stage 2 panels. An illustration of the coupling mechanism is shown in Figure 23.

![Figure 23. Detail of coupling splice between stage 1 strands and stage 2 strands.](image)

In addition to the transverse prestressing system, described above, the panels were also post-tensioned together longitudinally – in the direction parallel to the girders. The combination of transverse pre and post tensioning in addition to longitudinal post tensioning created a crowded interior for the deck panels.

The longitudinal post-tensioning ducts were located at mid-thickness of the panels in both stages. The transverse reinforcing, consisting of pre-tensioned strands in stage 1 and a
combination of pre- and post-tensioning, strands in stage 2, was located above and below the longitudinal ducts. Figure 24 shows the relative locations of the longitudinal and transverse reinforcing systems of a stage 2 panel. The only difference with the stage 1 panels would be that bonded pre-tensioned strands would be present in place of the transverse post-tensioning ducts.

![Figure 24. Deck section showing location of prestressing tendons in stage 2 deck.]

**4.2.4 Deck to Girder Connection**

The precast deck panels were intended to work in a composite fashion with the steel girders to resist vehicle live loading. This required a special connection between the panels and the girders.

The girders were not straight, but had sag or camber (old and new girders). In order for the flat deck panels to be supported on the girders, a masonry haunch was also placed between the two components to raise the deck to the correct road profile. Adjustable leveling bolts were used to temporarily hold the deck panels in position before the haunch masonry material was placed.

**4.2.4.1 Leveling Bolts and Haunches**

The precast deck panels were supported by leveling bolts when they were placed on the girders in the field. The leveling bolts could be adjusted to provide the required roadway profile while accounting for camber in the girders. Special hardware, depicted in Figure 25, was cast into the panels to hold the leveling bolts.

After adjusting the leveling bolts and setting the panels, the haunch thickness was found to vary from ¾” to 4½” in height. This area was filled with non-shrink grout. The contractor elected to use a tie wire stretched across the top of the girder to attach a 2x4 and plywood forms on each side to the bottom of the panels. No construction vehicles were allowed on the bridge until the haunch grouting was completed and the panels had a full bearing on the girders.
4.2.4.2 Shear Studs

The girders in the superstructure were designed to achieve their necessary stiffness and strength by acting compositely with the concrete deck panels. Headed shear studs were used to develop the composite action between the deck panels and the steel girders. For the precast deck panels, this method was adapted by casting blockouts, like windows, in the panels through which the studs could be welded to the supporting girders. A typical shear stud layout inside a blockout is shown in Figure 26.

![Figure 25. Leveling bolt system for setting deck panels in position.](image)

The spacing of the stud blockouts was selected as 4 feet. This exceeded the AASHTO specifications but the earlier testing proved the good composite action could still be achieved. The number of studs placed in each blockout depended directly on the amount of horizontal shear force that had to be transferred over that four foot spacing. At the 4 ft. spacing there were two blockouts placed above each beam in each panel.

The sequence of studs used was as follows:

- Starting from the abutment, the first two panels had 10 studs, $\frac{3}{4}$ inch diameter, per blockout as shown in Figure 25.
- The next two panels had 8 studs per blockout.
- The panel just before center span had 6 studs per blockout.
- The pattern above was then repeated in reverse from center span to the other abutment.
Due to the varying height of the haunch between the girders and the deck panels, two different length studs were used. The typical stud length was 6 in., however when the haunch was too deep, 8 in. studs were utilized. A minimum embedment depth of 2 in. into the deck panel was maintained for all shear studs.

![Figure 26. Group of steel studs welded to girder in a typical blockout hole.](image)

4.2.5 Parapet Wall

A Wisconsin standard sloped face parapet “LF” was utilized on this project. Epoxy coated parapet reinforcing bars were cast into the precast panels during production to make the parapet continuous with the deck panels. The same reinforcing type between the parapet and deck that is normally used in cast-in-place decks was used with the precast decks. Once the panels were placed, additional parapet reinforcing was tied in place in the field and the parapet was formed and cast in place using metal forms.

The use of a precast parapet would be preferred to meet the “rapid construction” goal of the project. A cast-in-place parapet was used, however, because a satisfactory crash tested precast parapet system was not found. Precast parapets are now being developed in Nebraska and the New England states.

4.2.6 Epoxy Overlay

Once the panels were in place, the top surface was ground down and an epoxy overlay was placed to both seal the deck for durability, and provide a uniform driving surface. The epoxy overlay was installed in 2 lifts. A temperature dependent cure time was
allowed between lifts. The 2-part epoxy supplied was Sikadur 22 Lo-mod, which was one of the pre-qualified manufacturers listed in the project special provisions. The aggregate used was Baraboo Quartzite supplied by Garnet Abrasives in Milwaukee, WI.

4.3 Precast Deck Panel Production

The precast prestressed concrete deck panes were manufactured by Spancrete Industries in Waukesha, WI. The stage 1 and stage 2 panels were cast at different times. The stage 1 panes were cast starting on June 14, 2005, which preceded the start of construction activity at the bridge site. The stage 2 panes were cast starting on August 1, 2005, which coincided with stage 1 construction activity at the bridge site.

At the manufacturing plant, the casting bed could accommodate up to three of the panels at any one time and was 12-feet wide. The actual casting sequence was 3-3-2-2 panels at at time for a total of 10 panels. The final two panels cast were to be placed at the east and west ends of the bridge. They were different from the eight interior panels because they had additional rebar, post-tensioning hardware, and blockouts. Spancrete felt that their experience casting the relatively simple interior panels first would be helpful with the casting the end panels, which had higher complexity because of the additional components.

4.3.1 Form Blockouts

Numerous blockouts had to be placed into the form beds prior to placing prestressing strands or concrete. Blockouts were used to form the stud pockets for composite action with girders, to form openings where post-tensioning ducts could be spliced, and to form spaces for post-tensioning anchorages. Placement of a blockout for a stud pocket is shown in Figure 27.

Figure 27. Gluing a stud pocket blockout to the bottom of the panel formwork.
4.3.2 Welded Wire Fabric

Welded wire fabric was placed at the top and bottom of the deck panel to control possible cracking during shipping and erection. The welded wire fabric was 6”x6” W4xW4. The steel area was not designed for strength. Fabric placement at the bottom of the form is shown in Figure 28. A similar layer was placed at the top after all inserts, ducts and strand were placed.

![Figure 28. Placing welded wire fabric into the deck panel formwork.](image)

4.3.3 Prestressing Strand, Ducts, and Reinforcing

Once the required blockouts, leveling bolt hardware, and bottom layer of welded wire fabric were in place, the prestressing strand was pulled into position. For stage 1
construction 12 lengths of pre-tensioned 270 ksi, 0.6” diameter, low relaxation strands were required for each set of panels. In the stage 2 construction 6 lengths of pre-tensioned strands were used and 6 post-tensioning ducts were set into place. Figure 29 shows placement of transverse bridge strand in the stage 1 panels.

The strands for pre-tensioning were stressed using a hydraulic jack. For a given strand an initial nominal tension of 3,000 lbs was applied to pull it straight. At this time the entire length of the strand was inspected to ensure the strand was clear of any obstructions and in the proper position. Once this was established, the stressing continued up to the required load level.

Once the strands were stressed, the post-tensioning hardware was put into position. In the typical interior stage 1 panes this included: plastic ducts for bridge longitudinal stressing, and cylindrical blockouts around the prestressing strand at the longitudinal joint where the strand would be spliced to post-tensioned strand in stage 2 pieces. In stage 2 panels this hardware included: plastic ducts for bridge longitudinal stressing, plastic ducts for bridge transverse post-tensioning, and anchorage hardware for the post-tensioning.

Steel rods were inserted inside the plastic post-tensioning ducts to keep them straight and in the correct position during the concrete pour. The cylindrical blockouts around the prestressing strand at the longitudinal joint location enabled the “tails” of prestressing strand to be unbonded over this length and give some working space for placement of the transverse couplers in the field. These blockouts with vertical ducts for subsequent duct grouting are shown in Figure 30.

For the stage 2 panels, only half as many transverse pre-stensioning strands were used. The remainder of the prestress was applied through post-tensioning strands placed in transverse plastic ducts. Strand was actually placed in these and used to hold the ducts in position during concreting. Anchorages for the post-tensioning were also required at the end of each duct along the outside edge of all stage 2 panels.

The final step in reinforcing the panels was to place epoxy coated steel bars into position to be used to connect the cast-in-place parapet wall to the bridge deck.

4.3.4 Prestress Transfer

Prestress was transferred to the deck panels after one day of cure by cutting the strands. Several panels were cast in line at the plant and there were gaps with exposed strand between the panels. The strands between the panels were cut simultaneously to prevent having panels suddenly sliding along the length of the bed due unsymmetric force. The 12 strands between panels were cut in sequence. The strands that were intended to subsequently be spliced to post-tensioning strand were protected from fraying during the cutting operation by use of cable clamps.
4.3.5 Storage and Shipping

After the forms were stripped away, the panels were lifted using an overhead crane and loaded onto the beds of semi-trucks, one panel per trailer. The trucks drove the panels into the precasting yard where they were unloaded for storage as shown in Figure 30. The panels had to be stored for a minimum of 30 days prior to placement at the bridge site. While in the yard the ends of the prestressing strands were removed flush with the surface of the panels except for the strands to be spliced in the field across the longitudinal bridge joint. The panel joint faces were blasted in order to roughen the surface for better joint bond in the field.

Figure 30. Completed stage 1 panel in the precast storage yard.

Figure 31. Proposed schedule for construction of twin bridges.
For shipping to the bridge site the panels were again loaded onto trucks, which were driven to the bridge approximately 65 miles away. The panels were lifted off the trucks and placed directly onto the bridge.

4.4 Bridge Construction Observations

Observation of the construction of bridges B-13-160 (with cast-in-place) and B-13-161 (with precast deck) was conducted for the purposes of:

- Comparing the schedule, labor time, equipment utilization, and cost of the innovative system to the conventional bridge system, and
- Identifying any problematic details that existed in the innovative system and noting suggestions for future applications.

The focus of the site observation and constructability study was on the deck systems. Other tasks such as roadway and approach work, substructure widening, grading, paving, and installation of new girders were not included other than for tracking the overall project schedule.

4.4.1 Proposed Project Schedule

The contractor submitted a construction schedule at a preconstruction meeting. The overall project start date was given as June 13, 2005. The start dates for each activity are shown for each structure in Figure 31.

The contractor apparently did not intend to take advantage of accelerated construction possible with the precast deck system. It can be inferred by examining the Figure that the intent of the contractor was to have work occurring on parallel tracks for each structure simultaneously. They also listed the shift from stage 1 to stage 2 as occurring on the same day. Similarly, the projects were planned to end on the same day on October 21, 2005.

The end date was important since one of the last steps of constructing the precast B-13-161 deck was the installation of the epoxy overlay, which would be difficult or impossible in colder weather.

4.4.2 Stage 1 Precast Deck Bridge

The stage 1 panels were placed successfully on August 3-4, 2005. As seen in Figure 32, each panel was shipped on a separate truck. The 100-ton capacity crane was used to lift the panel off of the truck. Each panel weighed much less than 100 tons, but the demand for a long crane reach required the 100 ton capacity. Prior to setting in place, the panel leveling screws (Figure 32) were rough-adjusted to the girder shape so the panel would
sit in a position to simultaneously meet the proposed roadway profile and account for camber in the girders.

Figure 31. Placement of stage 1 panel on the bridge girders.

Figure 32. Adjusting leveling bolts in panel before setting on girders.
4.4.2.1 Longitudinal Duct Splicing

Once all of the panels were in place, the longitudinal post tensioning strands were pushed through the ducts by hand. At each panel joint, the adjacent sections of ducts needed to be coupled together so that when the joints were grouted the duct would remain clear. A typical duct splice is shown in Figure 33. A heat shrink wrap was utilized; however this later proved ineffective in keeping the duct clear of grout. Use of a heat shrink tube was specified in contract documents, but was not employed in the stage 1 construction.

Figure 33. Heat shrink wrap utilized to splice longitudinal ducts.

Figure 34. Grouting transverse joints and duct splicing blockouts.
4.4.2.2 Grouting Transverse Joints

After the post-tensioning ducts were spliced, the panel joints, also called shear keyways, were grouted. The blockouts for the duct splicing were grouted in the same operation. The grouting operation is shown in Figure 34. The contractor utilized W.R. Meadows Sealight CG-86 non-shrink grout. Grouting required one day of work. The grout in the joints achieved sufficient strength (1000-psi, min.) to perform the longitudinal post-tensioning operations the next working day.

Figure 35. Schematic of the setup for testing stress in post-tensioned strand.

Figure 36. Red prestressing jack and a load cell placed on the strand end.
4.4.2.3 Longitudinal Post-tensioning Validation Test

The next step in construction was to post-tension the deck in the longitudinal direction, pulling all the individual panels into compression as a unit.

Prior to stressing longitudinal post-tensioning strands, the project special provisions called for the University of Wisconsin-Madison to perform a strand stress test. The purpose of the test was to measure the amount of friction loss from one end of the strand to the other, with some loss expected and 5% loss being specified as the maximum allowable.

The stress loss test was carried out by placing a load cell at each end of a strand to be tensioned in place as shown in Figure 35 & 36. The load cells were calibrated in the University lab before they were used in the field. The end with the jack was referred to as the “live” end and the end with the chuck was referred to as the “dead” end. The jack gauge pressure, strand elongation, and load cell forces were recorded in 10 increments up to 75% of the strand ultimate capacity (202 ksi in strand).

The results are shown in Figure 37. The applied load (x-axis) is plotted versus the load cell measured forces and the calculated strand force based on measured strand elongation. The intention of the measurement was to evaluate the strand-duct friction effect on inducing prestress loss.

Instead, the validation test showed that there was a major problem in the longitudinal ducts. Tests on a second strand showed similar results: a load of 45.4 k on the live end and 11.95 k at the dead end. The elongation in that strand was used to estimate a 28.7 k prestress level. This drastic loss of force from one end of the strand to the other, a 73% to 83% loss, indicated that the strand was locked up at some point along its length. Leakage of grout into the post-tensioning duct, from grouting the joints and blockouts, was suspected as the cause.

Fortunately, the elongation measurements combined with the load measurements at the two ends allowed a calculated estimate of where along the duct the “bonding” was likely to be. Figure 38 shows the problem when the grout in the blockout was hammered away and the ducts were examined. Indeed grout leakage into the duct had occurred. Twelve of the twenty three longitudinal ducts had such leakage. The leakage was the result of poor duct splicing, partly from using shrink tape rather than shrink tubing.

Before longitudinal post-tensioning operations could proceed, the ducts needed to be cleared of all the grout that had leaked in. The contractor attempted several techniques for accomplishing this. Some ducts were cleaned by attaching a masonry drill bit to a long reinforcing rod and drilling out the duct. The resulting delay caused by this construction error was 8 working days.
Figure 37. Loss of prestress in the longitudinal strands from various measurements.

Figure 38. Cleaned blockout, grout apparent in the center duct.

Figure 39. Strand anchorages ready for sealing (left), and filling with mortar (right).
Once the strands were freed, the University of Wisconsin – Madison research team again performed the load cell test on the first two strands to be post-tensioned. The calculated difference in measured readings for each strand from one end of the strand to the other was 2.23% and 1.94% respectively. This indicates that the friction loss was approximately 2%, well less than the 5% limit.

4.4.2.4 Longitudinal Post-tensioning Duct Grouting and Sealing

The final post-tensioning operation involved sealing the ends of the ducts and then fully grouting the ducts to provide protection for the strand. End caps were placed to cover the cut ends and the ends were dry-packed with a grout mixture. A bonding agent was used to coat the surfaces of the anchorage pockets prior to placing of the dry pack grout mix (see Figure 39). The contractor utilized Sika Grout 300 PT, which was mixed onsite by a technician employed by the supplier and then pumped into the ducts from one end with pumping continuing until the grout emerged from a port at the opposite end of the duct.

During the duct grouting operations, the technician reported having to pump with high pressure (as high as 250-psi) in order to get the grout to flow the length of the duct. In four ducts (#17, 8, 18, and 20), the grout was not able to be pumped for the entire length. He reported, however, that based on the volume of grout pumped that it was nearly at the end of the duct. He recommended pouring grout in form the other end, which was not allowed by the project special provisions. Despite this fact, his recommendation was followed and approximately 1 pint of grout was poured into the end of each duct.

Pouring grout into the duct from both ends was not allowed by the contract specifications because air pockets could be left at an intermediate location along the strand where no grout entered. The strand in such air pockets could be extremely susceptible to corrosion and failure if moisture (from grout mix water or other sources) was present.

As a result of the violation of the specifications a meeting was held at the bridge site on August 29, 2005 with representatives from WisDOT, FHWA, the contractor, the resident engineer, and the University of Wisconsin-Madison. The consensus of the meeting was that the panels should stay in place and the project should move forward since possible loss of the 4 compromised strands represented, at worst, a long term maintenance issue, not a structural capacity issue.

The wheel loads are carried in a transverse direction to girders, so the longitudinal prestress is not involved in the load resistance. The longitudinal prestress serves to keep transverse joints tightly closed. This would improve the durability of the bridge by preventing moisture entry and freeze-thaw damage. Loss of the suspect strand might compromise the joint seal in isolated locations and lead to durability problems.

4.4.2.5 Closure Pours

Casting of the closure pour areas at each end of the bridge was the next major activity to occur. Pour areas at each end of the bridge had been designed to accommodate
installation of the expansion joints. Setting the elevation of the expansion joint was a critical issue. The top surface of the panels as placed could not be used to set the elevation of the expansion joint since the panels would be ground down and have the epoxy overlay installed in the future. Since the closure pours were to be cast in place, conventional formwork had to be installed. This involved several small and unique form pieces due to the skewed ends of the bridge. A total of 4 working days was required to complete the formwork, reinforcing steel, and expansion joint installation in preparation for the closure pours.

4.4.2.6 Grouting Shear Stud Blockouts and Haunches

Once the deck panels were prestressed longitudinally to create a compressive pressure on the transverse joints, it was acceptable to then make the deck composite with the girders. This was accomplished by grouting the shear stud blockouts and haunch areas. It is important to recall that the deck must not be made composite until all prestressing is completed.

Light formwork was installed using a hammer-drill to drill a hole in the wood form and the concrete panel. Then a nail and length of tie-wire was hammered into the hole to hold the forms in place. For locations where the haunch depth was less than 1.5”, a 2x4 was utilized; where the haunch depth was greater than 1.5” plywood was cut to the required length to form the sides of the haunch.

Grout was transported using a wheelbarrow where it was poured into the shear stud blockouts. A vibrator was used to encourage the grout to flow along the length of the haunch and fill the void under the panels.

4.4.2.7 Grinding the Deck Surface

The surface of the bridge was rough due to slight variation in elevation of the precast deck panels, but particularly due to the unevenness in height of the grout at joints and blockouts. The surface of the deck was ground down using a diamond grinder. Finally, a shot blaster was then used to prepare the whole surface for application of the epoxy overlay.

4.4.2.8 Epoxy Overlay

Before application of the overlay, low spots in the deck were first filled by hand with a sand and epoxy mixture. The two-part epoxy overlay was mixed in buckets with a paddle wheel and spread around using squeegees. Aggregate was then broadcast by hand to completely saturate the epoxy. After the temperature-based cure time specified, the excess aggregate was swept up and the process repeated for the second lift as shown in Figure 40.

The completed stage 1 lanes are shown in Figure 41 after traffic had been switched and work was started on the stage 2 construction (past the right barrier).
Figure 40. Placing the second layer of epoxy and aggregate on stage 1.

Figure 41. Stage 1 construction complete on the precast deck bridge.
4.4.3 Stage 2 Precast Deck Bridge

The stage 2 precast deck panels were placed on September 28-29, 2005. Operations were slightly more complicated than stage 1 because the panels had to be lined up with the longitudinal joint as well as the transverse joint of the stage 1 panels. Most of the construction processes in stage 2 were identical to stage 1. The stage 2 operations that differed from stage 1 will be described here.

4.4.3.1 Transverse Strand Coupling

The primary difference between stage 1 and stage 2 panels was the presence of transverse post-tensioning ducts in place of half of the prestressing strands in the stage 2 panels.

The post-tensioning strands placed in the transverse ducts were spliced onto “tails” of stage 1 prestress strands left protruding from the stage 1 panels. The splice chucks were installed just prior to the panel placement. The post-tensioning strands were installed immediately following the panel placement.

Figure 42 shows the strand splicing at a blockout in the edge of the stage 2 deck panel. The stage 1 deck is on the left with the epoxy overlay visible. The top strands from both deck panels are shown with the strand coupler. The bottom strands have already been coupled and are enclosed in a sealed tube that allows the coupler to move when the strands are tensioned. After tensioning, grout will be pumped into the duct from the opposite (far right) end until it flows out of the tube at the left in the Figure, in the stage 1 piece.

4.4.3.2 Grouting Joints and Post-tensioning

The panels were set in place and the transverse strands were coupled, as described above. The longitudinal post-tensioning strands were placed in the ducts next. The transverse joints of the stage 2 construction were then grouted and the longitudinal strands were post-tensioned the following day. No problems were encountered in this stage with grout leaking into ducts.

After the deck was post-tensioned longitudinally, the next day the longitudinal joint between the stage 1 and stage 2 decks was grouted. The final joint, after grouting, is shown in Figure 43. With one day of grout cure sufficient to provide the desired strength, the deck was post-tensioned transversely.

Grouting of both the longitudinal and transverse post-tensioning ducts was completed the day after the transverse strands were stressed.

The remainder of the deck construction, including installation of steel studs, grouting stud pockets and haunches, end closure pours, grinding and application of the epoxy overlay were all accomplished by the same process as described for the stage 1 deck.
Figure 42. Splice of transverse strands at the longitudinal joint between stage 1&2 decks.

Figure 43. Longitudinal joint, to right of barrier, after grouting of joint and splice pockets.
4.4.4 Conventional Cast-in-place Deck

Construction of the conventional concrete deck was accomplished in the normal fashion with the following specific operations.

a) Hanger devices for supporting deck forms were installed on the girders.
b) Wood walers were hung from the hangers to span between the girders.
c) The elevation of the walers was adjusted to compensate for varying girder camber or sag.
d) Wood support timbers, 4in. x 4in. by 8 ft., were placed above the walers running parallel to the steel girders.
e) Plywood form boards were installed above the walers and timbers. Figure 44 shows the form construction with walers, timbers and plywood.

f) Angle brackets were installed to support deck overhang forms.

g) Identical longitudinal timbers and plywood decking was installed on the overhangs.

h) Headed studs were welded to the girders as composite action connectors.

i) Steel expansion joint hardware was installed at the ends of the deck formwork.

j) Side deck forms were placed.

k) Deck reinforcing bars were placed and tied together to form two separate top and bottom layers of orthogonal reinforcement.

l) Rails were installed on the side forms, running the length of the bridge, to support the concrete finishing machine.

m) The screeding and finishing machine was set on the rails and a dry run of the machine across the rails was used to check rail height and adjust for the proper deck elevation.

n) Deck concrete was poured using a concrete pump truck from one end of the bridge.

o) The deck was wet cured for 7 days.

p) TK-290 deck sealing compound was sprayed over the entire deck surface.

The planned construction schedule for both bridges was shown in Figure 31. Delays, problems encountered, and increased efficiency from familiarization all contributed to significant changes in the schedule. The actual construction timeline as recorded in the field is shown in Figure 45 and might be contrasted with Figure 31.

5 Prototype Bridge Evaluation

In addition to the method of construction of the precast deck system, the cost of the system, time to construct, and difficulties or lessons learned are also of paramount importance. The construction study particularly focused on special issues encountered during the fabrication process and involved interviews with many of the key personnel. In order to evaluate the cost of the system the bid unit price comparison, working hours recorded in the field, specialized equipment, and sub-contractor utilization were examined.

5.1 Cost Analysis

A good starting point for the cost analysis is the original contractor bid documents. Two competing companies bid on this job: Zenith Tech, Inc., and Lunda Construction Company. Zenith Tech was the low bidder at $1,807,945.46. The second bid was 9.3% higher.
The primary bid items that include labor on the B-13-160 (cast-in-place) bridge were:

<table>
<thead>
<tr>
<th>Item:</th>
<th>Unit</th>
<th>Quantity</th>
<th>Bid A</th>
<th>Bid B</th>
</tr>
</thead>
<tbody>
<tr>
<td>concrete masonry - bridges</td>
<td>cub. yard</td>
<td>171</td>
<td>68,400</td>
<td>72,675</td>
</tr>
<tr>
<td>coated reinforcing steel</td>
<td>lb.</td>
<td>36,370</td>
<td>31,278</td>
<td>28,368</td>
</tr>
<tr>
<td>other: deck sealer, studs, …</td>
<td></td>
<td>6,374</td>
<td>6,374</td>
<td>9,233</td>
</tr>
<tr>
<td>total, per square foot</td>
<td></td>
<td></td>
<td>$18.44</td>
<td>$19.17</td>
</tr>
</tbody>
</table>

The primary bid items that include labor on the B-13-161 (precast deck) bridge were:

<table>
<thead>
<tr>
<th>Item:</th>
<th>Unit</th>
<th>Quantity</th>
<th>Bid A</th>
<th>Bid B</th>
</tr>
</thead>
<tbody>
<tr>
<td>concrete masonry - bridges</td>
<td>cub. yard</td>
<td>26</td>
<td>10,400</td>
<td>11,050</td>
</tr>
<tr>
<td>coated reinforcing steel</td>
<td>lb.</td>
<td>3,606</td>
<td>3,101</td>
<td>2,813</td>
</tr>
<tr>
<td>other: deck sealer, studs, …</td>
<td></td>
<td>4,011</td>
<td>4,011</td>
<td>5,085</td>
</tr>
<tr>
<td>precast deck panels</td>
<td>sq. ft.</td>
<td>5,160</td>
<td>335,400</td>
<td>247,680</td>
</tr>
<tr>
<td>deck grinding</td>
<td>sq. ft.</td>
<td>5,254</td>
<td>5,254</td>
<td>11,821</td>
</tr>
<tr>
<td>epoxy overlay</td>
<td>sq. ft.</td>
<td>5,254</td>
<td>27,583</td>
<td>27,583</td>
</tr>
<tr>
<td>total, per square foot</td>
<td></td>
<td></td>
<td>$67.10</td>
<td>$53.21</td>
</tr>
</tbody>
</table>

Bidder A (Zenith) was awarded the project based on lower costs for other components. For the whole project Bidder A’s precast estimate was 264% of the conventional, bidder B was 178%. An analysis of the bids shows that the precast deck system is considerably more expensive than a cast-in-place bridge deck. It is also clear, however, that there is considerable variation in just the cost of the precast panels, one being 35% higher. This variation is most likely attributable to uncertainty involved in construction with the new system.

Bidder A provided a breakdown of their bid for the “panels” item which is shown in Figure 46. Nearly 60% of the cost was for materials. The panel materials cost was $39 per square foot. This high cost of precast panels in Wisconsin will make them attractive only where there is a significant other incentive for their use such as need for rapid deck replacement. In contrast, precast panel cost in other portions of the country, such as Nebraska, have been reported as much as 50% lower than the Wisconsin cost.

![Figure 46. Breakdown of bid for “precast panels” installed in bridge.](image-url)
5.2 Labor Hours Required

During the course of observing the construction process, the number of personnel, the tasks they performed, and the time and duration of these tasks was recorded.

The Table below summarizes the labor required for the bridge deck construction of both structures. The deck for the precast structure B-13-161 required more man-hours to install in the field than the conventional B-13-160. Overall (including the delay observed on B-13-161 in stage 1) 1,337.0 man-hours were required to construct the B-13-161 deck, where as B-13-160 required only 948.5 man-hours; this is a difference of 41%.

<table>
<thead>
<tr>
<th></th>
<th>B-13-161 (man-hours)</th>
<th>B-13-160 (man-hours)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Overall</td>
<td>1,337.0</td>
<td>948.5</td>
</tr>
<tr>
<td>Stage 1</td>
<td>704.5</td>
<td>432.5</td>
</tr>
<tr>
<td>Delay</td>
<td>169.5</td>
<td>0.0</td>
</tr>
<tr>
<td>Stage 1 (no delay)</td>
<td>535.0</td>
<td>432.5</td>
</tr>
<tr>
<td>Stage 2</td>
<td>632.5</td>
<td>516.0</td>
</tr>
</tbody>
</table>

Man-hours required for construction of the bridge decks.

Looking at each stage of construction, B-13-161 required 63% and 23% more man hours than B-13-160 in stage 1 and stage 2, respectively. In stage 1, however, there was a delay due to a construction error of 8 working days, during which 169.5 man-hours were expended. Taking this into account, the precast B-13-161 deck still required more man-hours than B-13-160, however the difference was only 24%. If the man-hours expended during the delay are discounted from the overall total, then the B-13-161 deck required 1,167.5 man-hours to construct in the field, which is 23% higher than the B-13-160 deck.

More man-hours were expended during stage 2 than in stage 1 for both structures. On B-13-161, the sub-contractor for the post tensioning activities maintained more of a presence on site in stage 2 than in stage 1. This was because there was additional work (the transverse coupling and post-tensioning) in stage 2 that was not present in stage 1 construction. This sub-contractor also wanted to avoid the difficulties they experienced with the duct sealing that they had in stage 1.

The prime contractor, also, had additional work in stage 2 (longitudinal joint forming and grouting) that contributed to the greater stage 2 man-hour expenditure. For B-13-160, the primary reason for additional stage 2 man-hour expenditure is that all of the time spent stripping the deck forms was included in the stage 2 totals. All of the forms, both from stage 1 and stage 2, were left in place until after the entire deck was complete and the forms were removed at the end.
It should be noted that the man-hours discussed above were limited to the time spent constructing the bridge decks in the field. The precast panels were manufactured ahead of time in the precasting plant and shipped to the site. The man-hours required for the panel manufacturing are not included in the above man-hours totals.

### Precast – B-13-161 Bridge: Task Construction Time for Stage 1

<table>
<thead>
<tr>
<th>Stage 1 Activities</th>
<th>Maximum number of workers used</th>
<th>Days over which work occurred</th>
<th>Total number of worker hours</th>
<th>Percent of hours in stage</th>
</tr>
</thead>
<tbody>
<tr>
<td>delay, free and replace strands, grout joints</td>
<td>8</td>
<td>8</td>
<td>109.5</td>
<td>24.00%</td>
</tr>
<tr>
<td>forming and casting closure pour at deck ends</td>
<td>7</td>
<td>8</td>
<td>155.0</td>
<td>32.00%</td>
</tr>
<tr>
<td>forming and grouting transverse joints</td>
<td>6</td>
<td>3</td>
<td>65.5</td>
<td>14.6%</td>
</tr>
<tr>
<td>epoxy overlay</td>
<td>8</td>
<td>2</td>
<td>60.0</td>
<td>13.2%</td>
</tr>
<tr>
<td>forming for launcheans</td>
<td>4</td>
<td>3</td>
<td>52.0</td>
<td>11.8%</td>
</tr>
<tr>
<td>grouting hanch and shear stud blockouts</td>
<td>5</td>
<td>2</td>
<td>51.5</td>
<td>11.4%</td>
</tr>
<tr>
<td>splicing/sealing longitudinal ducts</td>
<td>2</td>
<td>3</td>
<td>40.0</td>
<td>9.1%</td>
</tr>
<tr>
<td>erecting precast panels</td>
<td>4</td>
<td>2</td>
<td>31.0</td>
<td>7.1%</td>
</tr>
<tr>
<td>grinding and preparing deck surface</td>
<td>3</td>
<td>3</td>
<td>25.0</td>
<td>5.6%</td>
</tr>
<tr>
<td>inserting longitudinal strand</td>
<td>3</td>
<td>1</td>
<td>17.5</td>
<td>3.9%</td>
</tr>
<tr>
<td>grouting longitudinal ducts</td>
<td>3</td>
<td>1</td>
<td>17.0</td>
<td>3.7%</td>
</tr>
<tr>
<td>stressing longitudinal strands</td>
<td>2</td>
<td>1</td>
<td>10.0</td>
<td>2.2%</td>
</tr>
<tr>
<td>installing shear studs</td>
<td>2</td>
<td>2</td>
<td>7.0</td>
<td>1.5%</td>
</tr>
<tr>
<td>cutting and sealing ends of longitudinal ducts</td>
<td>2</td>
<td>1</td>
<td>5.5</td>
<td>1.2%</td>
</tr>
</tbody>
</table>

Shaded cells are activities relating to post-tensioning

Stage 1 Total = 704.5 100.00%

### Precast – B-13-161 Bridge: Task Construction Time for Stage 2

<table>
<thead>
<tr>
<th>Stage 2 Activities</th>
<th>Maximum number of workers used</th>
<th>Days over which work occurred</th>
<th>Total number of worker hours</th>
<th>Percent of hours in stage</th>
</tr>
</thead>
<tbody>
<tr>
<td>forming and casting closure pour at deck ends</td>
<td>7</td>
<td>4</td>
<td>147.5</td>
<td>32.32%</td>
</tr>
<tr>
<td>erecting precast panels</td>
<td>7</td>
<td>2</td>
<td>74.0</td>
<td>16.70%</td>
</tr>
<tr>
<td>forming for launcheans</td>
<td>3</td>
<td>3</td>
<td>53.0</td>
<td>11.9%</td>
</tr>
<tr>
<td>forming and grouting transverse joints</td>
<td>5</td>
<td>1</td>
<td>46.5</td>
<td>10.6%</td>
</tr>
<tr>
<td>installing transverse coupling hardware</td>
<td>6</td>
<td>3</td>
<td>44.5</td>
<td>9.94%</td>
</tr>
<tr>
<td>splicing/sealing longitudinal ducts</td>
<td>4</td>
<td>1</td>
<td>38.5</td>
<td>8.61%</td>
</tr>
<tr>
<td>forming and grouting longitudinal joint</td>
<td>4</td>
<td>1</td>
<td>31.0</td>
<td>6.90%</td>
</tr>
<tr>
<td>epoxy overlay</td>
<td>5</td>
<td>2</td>
<td>25.0</td>
<td>5.65%</td>
</tr>
<tr>
<td>grinding and preparing deck surface</td>
<td>3</td>
<td>2</td>
<td>24.5</td>
<td>5.47%</td>
</tr>
<tr>
<td>stressing transverse strands*</td>
<td>5</td>
<td>1</td>
<td>22.5</td>
<td>5.05%</td>
</tr>
<tr>
<td>grouting hanch and shear stud blockouts</td>
<td>2</td>
<td>2</td>
<td>20.5</td>
<td>4.52%</td>
</tr>
<tr>
<td>inserting longitudinal strand</td>
<td>3</td>
<td>1</td>
<td>17.5</td>
<td>3.87%</td>
</tr>
<tr>
<td>installing shear studs</td>
<td>2</td>
<td>2</td>
<td>17.0</td>
<td>3.77%</td>
</tr>
<tr>
<td>inserting transverse strand*</td>
<td>4</td>
<td>1</td>
<td>16.0</td>
<td>3.51%</td>
</tr>
<tr>
<td>stressing longitudinal strands</td>
<td>2</td>
<td>1</td>
<td>11.0</td>
<td>2.45%</td>
</tr>
<tr>
<td>cutting and sealing ends of transverse ducts*</td>
<td>5</td>
<td>1</td>
<td>10.5</td>
<td>2.27%</td>
</tr>
<tr>
<td>strip hanch and closure pour forms*</td>
<td>3</td>
<td>2</td>
<td>10.0</td>
<td>2.21%</td>
</tr>
<tr>
<td>grouting transverse ducts*</td>
<td>3</td>
<td>1</td>
<td>9.0</td>
<td>1.94%</td>
</tr>
<tr>
<td>grouting longitudinal ducts</td>
<td>3</td>
<td>1</td>
<td>9.0</td>
<td>1.94%</td>
</tr>
<tr>
<td>cutting and sealing ends of longitudinal ducts</td>
<td>2</td>
<td>1</td>
<td>5.0</td>
<td>1.09%</td>
</tr>
</tbody>
</table>

*Task specific to stage 2 (no stage 1 equivalent)

Stage 2 Total = 632.5 100.00%

Shaded cells are activities relating to post-tensioning

The specific tasks required to construct the precast bridge deck B-13-161 in each stage were tracked and are summarized in the Tables above. Several observations can be made by examining the Tables.
• First, in general, any task requiring forming, especially the closure pour areas, required a relatively large number of worker-hours.
• Second, the delay experienced in Stage 1 as a result of construction error (not properly sealing the longitudinal ducts) represented 24% of the total worker-hours expended in that phase.
• Third, the erection of the precast panels in stage 2 required a notably larger number of worker-hours than in stage 1 because of increased difficulty in aligning the panels and because the post-tensioning subcontractor maintained a presence on site during the placement of the panels.
• Finally, the number of worker-hours required to carry out the post-tensioning activities is highlighted in the tables. In Stage 1, 90.0 of the 704.5 total worker-hours, or 12.78%, was taken up by the post-tensioning activities. The same tasks repeated in Stage 2 required only 81.0 worker-hours. However, the additional tasks involved in installing the transverse post-tensioning system required an additional 102.5 worker-hours. Thus in Stage 2, 183.5 out of the 632.5 total worker-hours, or 29.01%, was taken up by all post-tensioning activities.

5.3 Specialized Equipment

The largest piece of special equipment required for placing the panels was the Manitowoc Model 222 Lattice Boom Crawler Crane (100-ton capacity). This size crane was required to lift the panels into place. The stage 1 and 2 panels weighed 27,100 lb and 31,100 lb each, respectively. Even with the 100-ton crane, it was necessary for the contractor to construct a temporary mat on top of the girders for the crane to achieve the necessary reach to set the middle panels. This crane was brought on site for panel placing, and would not have been required had the entire job been conventionally constructed. The next largest crane that was used on the project was an American 5299 (50-ton capacity) crane.

For the remainder of the panel grouting and post-tensioning operations there were several smaller pieces of equipment required. These included: 2 calibrated hydraulic jacks for tensioning the strands, mechanical shears to cut the ends of the strands, a grout mixer and pump unit, and a mortar mixer.

For the deck grinding and preparation a separate sub-contractor was engaged to perform the diamond grinding. They brought 3 trucks carrying their equipment: one truck carried the diamond grinder, a second truck carried additional equipment and a pickup, and the third truck was the wastewater truck, which pumped and held the tailings from the grinder during operation. Additional deck preparation required the use of a small mill and a shot blaster, which were supplied by another sub-contractor. These were brought to the site only for the day which they were in use and were immediately removed from the site upon completion of the work.
5.4 Sub-Contractors

The bids for constructing the precast deck bridge were higher than the conventional deck. One contributing factor may be the number of sub-contractors required. As shown in the Table below, the prime contractor had to utilize an additional 5 sub-contractors during the course of constructing the precast B-13-161 than on B-13-160. It should also be noted that 3 of the 5 additional sub-contractors were required due to the grinding and epoxy overlay application.

<table>
<thead>
<tr>
<th>Sub-contractor</th>
<th>Scope of Service</th>
<th>B-13-160</th>
<th>B-13-161</th>
</tr>
</thead>
<tbody>
<tr>
<td>Red Cedar Steel Erectors</td>
<td>Steel Girders and Stud Connectors</td>
<td>X</td>
<td>X</td>
</tr>
<tr>
<td>Wingra Stone Company</td>
<td>Redi-Mix Concrete</td>
<td>X</td>
<td>X</td>
</tr>
<tr>
<td>CGC, Inc.</td>
<td>Concrete Testing</td>
<td>X</td>
<td>X</td>
</tr>
<tr>
<td>Chilstrom Erecting</td>
<td>Tie Rebar, Post-tensioning</td>
<td>X</td>
<td>X</td>
</tr>
<tr>
<td>Spancrete</td>
<td>Precast Panels</td>
<td>X</td>
<td></td>
</tr>
<tr>
<td>Dywidag Systems International</td>
<td>Grout Ducts</td>
<td>X</td>
<td></td>
</tr>
<tr>
<td>Diamond Surface Inc</td>
<td>Deck Grinding</td>
<td>X</td>
<td></td>
</tr>
<tr>
<td>Surf Prep</td>
<td>Shot Blast and Mill Deck</td>
<td>X</td>
<td></td>
</tr>
<tr>
<td>Truesdell Corp.</td>
<td>Epoxy Overlay</td>
<td>X</td>
<td></td>
</tr>
</tbody>
</table>

A breakdown of the 582 hours of subcontract work is shown in Figure 47. Clearly the post-tensioning demanded a high number of man-hours. This sub-contractor expended 71% of all the sub-contractor man-hours. The next highest was for the epoxy overlay, which accounted for 15% of sub-contractor man-hours.

![Figure 47. Distribution of sub-contractor man-hours on the precast bridge.](image-url)
5.5 Contractor Submittals

The special provisions for this project required that the contractor make several submittals to the State’s representative, referred to as the resident engineer (RE) for the project. Many of the submittals were required because the methods are not commonly used in Wisconsin bridge construction and the design team desired that the contractor focus special attention on those aspects of the process.

As a courtesy, copies of the contractor’s submittals relating to the bridge deck were provided to the research team and are listed in the Table below. Notably, the items relating to the duct grouting were never submitted and are highlighted in the Table. If a grouting operation plan had been created and submitted, then the difficulties encountered while grouting the ducts may have been avoided.

<table>
<thead>
<tr>
<th>Spec. Section</th>
<th>Item/Copy Requested</th>
<th>Status</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td><strong>Deck Panels Submittals</strong></td>
<td></td>
</tr>
<tr>
<td>B.1</td>
<td>Concrete Mix</td>
<td>Ed Pioja (Ayres) submitted collection of individual reports to RE. Copy rec’d 2/1/2006</td>
</tr>
<tr>
<td>B.1</td>
<td>Final Concrete Report</td>
<td></td>
</tr>
<tr>
<td>B.1</td>
<td>“Coulomb Test” Results</td>
<td>Copy obtained from RE 12/30/2005</td>
</tr>
<tr>
<td>B.1</td>
<td>“Slab Test” (Chloride Ion Permeability) Results</td>
<td>Tests completed (not required by contract). Roger Becker (Spaacrete) provided results via email 2/21/2006</td>
</tr>
<tr>
<td>B.8.2</td>
<td>Grouting Operation Plan</td>
<td>RE reports it was never submitted to him</td>
</tr>
<tr>
<td>C.5.2</td>
<td>Tendon Modulus of Elasticity Test Results</td>
<td>Included in individual reports submitted to RE from Ed Pioja (Ayres) listed at top</td>
</tr>
<tr>
<td>C.5.4</td>
<td>Jack Calibration Charts</td>
<td>Copy obtained from RE 12/30/2005</td>
</tr>
<tr>
<td>C.5.5</td>
<td>Stressing Record (Elongations and Pressures)</td>
<td>Copy obtained from RE 12/30/2005</td>
</tr>
<tr>
<td>C.7.4</td>
<td>Duct Grout Placing Record</td>
<td>RE reports that his field notes were the basis for acceptance of work, but no official form was kept</td>
</tr>
<tr>
<td></td>
<td><strong>Epoxy Overlay Submittals</strong></td>
<td></td>
</tr>
<tr>
<td>B.2</td>
<td>Aggregate Gradation</td>
<td>Copy obtained from RE 12/30/2005</td>
</tr>
<tr>
<td>B.2</td>
<td>Hardness Test</td>
<td>RE reports it was never submitted as separate item. Material Certifications list an acceptable value.</td>
</tr>
<tr>
<td></td>
<td><strong>Other Information</strong></td>
<td></td>
</tr>
<tr>
<td>-</td>
<td>Level runs for bolt elevations</td>
<td>RE provided half of stage 2 data. The rest was misfiled or missing.</td>
</tr>
<tr>
<td>-</td>
<td>Inspector’s Daily Reports</td>
<td>Copy obtained from RE 12/30/2005</td>
</tr>
</tbody>
</table>
Informally, a list of required submittals and deadlines was compiled and given to the project superintendent at the preconstruction meeting in an early attempt to increase awareness of the unusual nature of this project. Still, the contractor was not pleased with the unusual number of submittals required. All of the required submittals were eventually made except for the two relating to the grouting operations. These turned out to be critical given the difficulties encountered with the duct grouting in stage 1.

5.6 Problems/Difficulties Encountered with the Precast Deck Bridge

The primary difficulties encountered were:

- The originally proposed concrete mix design did not meet the specification requirements with respect to the rapid chloride ion permeability test.
- Shear studs were improperly placed within some blockouts.
- The stage 1 longitudinal post-tensioning ducts were not properly sealed prior to grouting the surrounding joints.
- Longitudinal post-tensioning ducts were difficult to pump full of grout.

5.6.1 Concrete Mix Design for Precast Deck Panels

The special provisions required that three separate rapid chloride permeability tests be conducted in accordance with ASTM C1202, on 4”x8” concrete cylinders prepared from the same materials to be used in casting the panels. The samples were to be mixed in conformance with ASTM C 31. Also, that all three rapid chloride permeability tests should be performed on cylinders that were 56 days old. The ASTM C 1202 estimates chloride penetration based on the amount of electrical current passing through the specimen in a 6 hour period.

After bridge work was started, the University and the design engineers realized that the C1202 test was quite controversial and results were questionable. Documentation obtained from the Precast Concrete Institute showed widely varying C1202 results for specimens that had actual chloride ion content that did not widely vary.

ASTM C 1543 is a more direct measurement of the permeability of the concrete. The test involves ponding a sodium chloride solution on top of the test specimen and periodically taking samples from specified depths for chemical analysis. Test results are reported as chloride content of the specimen as a function of depth. This test is generally referred to as the “Ponding Test”. Unfortunately this requires a much longer test time and was not chosen.

Just prior to casting the stage 1 panels the results of the specified C1202 Coulomb Test were reported. The charge passed for each of the three specimens tested was reported as 3,278, 3,769, and 3,341 Coulombs respectively. These values were classified as “moderate” according to the ASTM specification and exceeded the maximum allowable value of 1,500 Coulombs given by the contract special provisions.
WisDOT and the University recognized that the Coulomb test was probably not a good measure of acceptance, but that was what was required by the awarded contract. The precaster chose to revise their mix by adding 5% silica fume and kept all other quantities the same.

The addition of 5% silica fume to the concrete produced acceptable Coulomb Test results of 796 and 894 Coulombs for the two specimens tested; this was below the maximum value allowed of 1500 Coulombs. The SCC concrete was observed to be somewhat thicker than usual, but still performed adequately during panel casting.

The precaster also decided to run C1543 ponding tests to prove the behavior of the mix. The Ponding Tests were completed in February 2006 on specimens that had sodium chloride solution ponded on them for 6 months. The test report showed that for the four slabs tested the chloride ion concentration measured between 0.031 to 0.100% by weight of sample in the 0.06”-0.5” depth range and between 0.030 to 0.036% by weight in the 0.5”-1.0” depth range. No standard acceptance criteria exist for the results of this test. It should also be noted that no samples were tested without the addition of the silica fume, so no comparison can be made between the standard SCC mix and the SCC mix with silica fume added.

5.6.2 Shear Stud Placement

When the sub-contractor started installing the shear studs on the beams for the composite precast deck, they did not use a template or any other means to locate the actual shear studs as per plan. The result was that the shear stud arrangement did not match the specified spacing dimensions and pattern shown in the contract documents. Typical examples of the improper shear stud arrangement are shown in Figure 48.

For the remainder of stage 1 construction the resident engineer laid out a sample arrangement of studs in a single pocket and requested that the remainder of the shear studs be installed as specified in the plans. With this guidance the remaining studs were applied correctly.

In stage 2 construction, the prime contractor utilized a cardboard template to mark the locations to grind off the paint in preparation for the shear stud installation. This resulted in less grinding work for the prime contractor and much better compliance with the dimensions specified in the contract documents.
5.6.3 Sealing of Post-tensioning Ducts at Splices

In order to post-tension longitudinally, each panel had a set of ducts at mid-depth that ran across the panel width. Such ducts in adjacent panels lined up so that a continuous post-tensioning strand could be fed the length of the bridge through all of the panels. The joints were to be grouted to form a solid joint.

Before this could be done the ends of the post tensioning ducts protruding into the joint blockouts needed to be coupled and sealed. This was to provide a continuous open duct for the longitudinal post tensioning strands, which were to be stressed once the grout in the joints hardened. The methods employed to splice the ducts in the stage 1 panels proved to be ineffective.

The construction documents showed that the splices were to be protected with a shrink tube. Unfortunately a change was made in the shop drawing to a heat shrink wrap product and the change was not noticed by the designers. Still, heat shrink tubes were delivered by the supplier to the job site. The constructor sliced the tubes and converted it to a shrink wrap, as shown in Figure 49, which did not adequately seal the splices.
An additional modification that may have contributed to the duct leakage problem was in a change of grout materials. The contractor ran short of the CG-86 construction grout before the joint grouting was finished. They then substituted the Sika 300 PT grout intended for pumping into the post-tensioning ducts. This was a much less viscous grout meant to flow through tight ducts and may have found an easy path to enter the splices.

Finally, as a result of grout entering the ducts and hardening, the ducts had to be subsequently drilled clear before new strand could be inserted and post-tensioned. Due to residual grout in the ducts, the grout pumping operation to fill the ducts was very difficult and grout could not be pumped from one end to the other.

The errors in this process might be tracked back to the contractor’s negligence in not submitting a grouting plan (per Specifications) and the resident engineer not carefully insuring that the project plans and specifications were followed.

An alternate duct splicing method was used in the stage 2 construction. Lengths of PVC pipe were sleeved over the duct ends to be joined; longer PVC sections were used where a relatively short duct end protruding from the panel necessitated it. A silicone caulk was then squirted into the gap between the PVC sleeve and the duct. Once the silicone cured, the entire assembly was wrapped with tape. The tape seams were then squirted with a spray on rubber product. From observations it appeared that the tape product alone would have been equally effective in sealing the ducts, and be quicker and easier to install.

5.6.4 Summary

Most of the problems listed above could have been avoided. An important recommendation for future projects would be to closely follow the specifications contained in the contract documents.

The responsibility falls to the project inspector or resident engineer to insist on compliance with the project specifications. In this case the resident engineer did not demand compliance and there was no consequence or penalty for non-compliance. The specifications provided procedures to follow in the situations encountered in the field; however lack of review ahead of time caused inappropriate action to occur.

5.7 Post Construction Feedback

Throughout the course of the construction comments and feedback were elicited from the construction workers and material suppliers. Some of the comments were recorded informally, others were collected in structured interviews with single or multiple participants in a group.
5.7.1 Comments from Precast Deck Manufacturer’s Employees

1) Post tensioning was not a familiar process for the manufacturer. Correct placement of many of the post-tensioning components in the panels depended some on familiarity with the system so production planning was more difficult than normal.

2) A lot of time was spent in coordinating with the post-tensioning material supplier to develop shop drawings and determine what materials were needed.

3) They recommended that the State specify a single supplier/manufacturer for the post-tensioning materials so that the use of those materials could be standardized at a precasting plant.

4) The large number of “blockouts” in the deck panels meant more labor in forming.

5) Single flat ducts, that could accommodate 3 strands, would have been easier to work with compared to the three separate single strand ducts used.

6) The biggest factor in the panel cost was the size and number of pieces. Increasing the piece size, for instance full width panels without a longitudinal staging joint, would be more economical. Pieces to 80,000 lbs and 16 ft. wide could be moved and shipped.

7) The parapet should be cast on the panels at the plant before shipping.

8) Due to the congestion in the panels, with steel mesh, pretensioned strands, and three layers of post-tensioning ducts, the panels could not have been cast with quality results if SCC concrete had not been used.

5.7.2 Comments from the Field Construction Team

1) Due to unfamiliarity with post-tensioning, the contractor had to bring in additional sub-contractors who would not be used on a normal job.

2) The addition of sub-contractors complicated the prime contractor’s scheduling.

3) The contractor preferred joints that were not post-tensioned.

4) Use of larger precast pieces would improve production economy and reduce the number of joints in the structure.

5) Design deck elevations were given at 1/8 points along span, however setting the leveling bolts on the precast panels required more elevations. Elevations should be provided at the edge of each panel or at the leveling bolt locations.

5.7.3 Comments from WisDOT Personnel

1) There is concern about long term seepage through the deck joints.

2) A joint key system (male-female) that would allow the joints to lock together during placement might make setting panels easier during construction.

3) Installation of expansion joints on a deck that is to receive grinding of the surface creates a problem in correct elevation location of the joints so they are not damaged. Joints should be designed to be below the deck surface.
5.7.4 Design Engineer’s Comments

1) Possible degradation of joints and degradation of the epoxy overlay should be closely monitored.
2) This technology should be used in a truly “fast” construction rather than at a normal bridge construction pace.
3) A joint that does not need post-tensioning should be developed.
4) A decision matrix should be developed to identify projects where precast decks should be used.
5) The requirements for permeability and chloride ion penetration as used in this project should be modified since the test listed may not be reliable.
6) Some kind of leak test should be specified to check the sealing of duct splices.

5.7.5 Summary

Based on the comments from the participants and the bid cost, accelerated construction and use of post-tensioning represents a major paradigm shift for the construction team. This introduction to new systems requires a large degree of up-front planning and coordination. Contractors may not be accustomed to such preplanning.

For the State to take advantage of more efficient structural systems and the benefits of rapid construction, an alternate method of awarding contracts may be needed. The contract needs to specifically award a contractor for rapid construction or construction of more durable structures and/or penalize for long construction duration by charging a “rent” for the time the site is under construction.

6. Bridge Testing

In addition to laboratory tests, described earlier, a series of tests were conducted on the actual bridge components. In order to evaluate the system’s performance in-situ, seven tests were conducted:

- At the precast plant during strand detensioning of a panel
- Load cells during 1st attempt at post-tensioning in the field
- Load cells during 2nd attempt at post-tensioning in the field
- Longitudinal strains during stage 1 post-tensioning
- Load test of B-13-160
- Load test of B-13-161
- Relative displacements of longitudinal joint on B-13-161

The results of only the bridge load tests and the longitudinal joint test will be discussed here.

In order to measure material strains during the load tests, two types of resistive strain gauges were utilized. The first type was used for measuring strain in metal surfaces of
the girders and on steel reinforcing bars. The second type of gauge utilized was a 60mm long gage for measuring strain on concrete surfaces (the top and bottom of the concrete deck). The exact locations of the gages are shown in the Appendix.

A differential transformer (LVDT) was also used to measure the relative displacements at the longitudinal staging joint. The LVDT is shown in Figure 50 with the stage 2 deck on the right and the stage 1 on the left. The LVDT is measuring the relative vertical displacement at the joint.

![Figure 50. LVDT used to measure relative vertical displacement at stage joint.](image)

A differential transformer (LVDT) was also used to measure the relative displacements at the longitudinal staging joint. The LVDT is shown in Figure 50 with the stage 2 deck on the right and the stage 1 on the left. The LVDT is measuring the relative vertical displacement at the joint.

6.1 Load Testing of Conventional Bridge

For purposes of comparing the structural performance of the conventional bridge to the precast deck bridge, a load test was carried out on B-13-160. The bridge was instrumented with strain gauges. The truck utilized for the load test was provided by Huml Contractors. The truck weight and dimensions are shown in Figure 51.

![Figure 51. Truck wheel locations and weights for conventional bridge loading.](image)
The truck was positioned in two different locations on the bridge. Position 1 was intended to produce the maximum transverse deck strain. The center of the rear wheels on the driver side of the truck was centered between girders 2 and 3. Position 2 was intended to produce the maximum longitudinal girder strains. The truck was centered over the top of the girder.

Gages placed on girder 2 and the deck above girder provided information on the composite girder bending behavior. The strain profile through the depth of the girder with the truck in position 2 is shown in Figure 52. (Note: $\mu e$ is strain x $10^{-6}$)

Using AASHTO LRFD lane load distribution factor (0.47) and the AASHTO suggested effective concrete flange width, the theoretical bending strains in the girder under the truck loading were also calculated. The theoretical strain profile is also shown in Figure 52. The AASHTO prediction produces higher strain estimates than actually measured.

![Figure 52](image)

Figure 52. The measured girder strains (diamonds) are compared with theoretical results for girder 2 below the conventional deck. The solid line is a least squares linear fit for the measured strains.

With the truck in the first loading position, the transverse strains in the concrete deck and on the top and bottom surfaces were measured. The top transverse deck strain at deck midspan was 57$\mu e$ in compression. The strain at 1.5 in. above the bottom was -50 $\mu e$ in tension. The theoretical predicted values using the AASHTO effective strip width were 50 $\mu e$ at the top and -34 $\mu e$ near the bottom.
Over the deck support, the top of girder 2, the top deck transverse strain was measured at -134 με in tension. The theoretical strain was predicted as -64 με. It was not apparent why the measured exceeded the predicted by a factor of two. The measured strain is very near to the expected concrete cracking strain of -132 με.

6.2 Load Testing of Precast Deck Bridge

A different truck was used for the B-13-161 precast deck load testing. The dimensions and weight of the truck are shown in Figure 53. Three different truck positions were used in testing of this bridge.

Position 1 was intended to produce the maximum transverse deck strain. For transverse alignment the truck was placed so that the center of the rear wheels on the driver side of the truck was centered between girders 6 and 7. Position 2 was intended to produce the maximum joint opening of the transverse panel joint at the midspan of the bridge. The truck pulled straight forward from Position 1 until its rear axle was centered over the panel joint. Position 3 was intended to produce the maximum longitudinal girder strains. For transverse alignment, the truck was centered over the top of the girder.

![Figure 53](image-url) Truck wheel locations and weights for precast bridge loading.

![Figure 54](image-url) The measured girder strains (diamonds) are compared with theoretical results for girder 6 below the precast deck. The solid line is a least squares linear fit for the measured strains.
With the truck in position 3, causing maximum girder bending, the strain profile in the girder and deck above was measured and is plotted in Figure 54. Again, theoretical predicted strains using the AASHTO LRFD lane load distribution factor on the truck and the effective flange width are shown with the measured strains.

In this case the peak girder strain is about 2/3 of the value measured for the conventional deck bridge. Naturally the prestressed precast deck should be expected to be stiffer since cracking is less likely. The higher stiffness should result in the truck load being distributed more uniformly to all girders – which would result in less strain in the heavily loaded girder.

The peak strains are also lower than predicted using the AASHTO distribution factor, by 62% in compression at top and 51% in tension at bottom. Still the neutral axis location is consistent with the predicted location. Thus the deck provides better load distribution than expected by AASHTO.

Based on rough survey elevation measurements taken at the top of the deck surface with the truck in position, the load distribution to the girders would be:

Girder 5 … 22%  Girder 6 … 57%
Girder 7 … 22%  Girder 8 … 0%

The AASHTO factor for girder 6 was 44%.

With the truck moved to the first loading position the transverse bending strains in the deck were measured. At the center of the deck span the top strain was 29 \( \mu e \) in compression and the bottom strain was -34 \( \mu e \) in tension. Theoretically predicted strains, using the AASHTO LRFD effective strip width, were 30 \( \mu e \) at the top and -30 \( \mu e \) at the bottom.

The transverse flexural strain over the top of the support, girder 6, was also measured with the truck in the same position. The measured strain was -6 \( \mu e \) in tension, the theoretical predicted value was -31 \( \mu e \) in tension. In this deck the measured strain was not near the cracking level and was 81% smaller than the theoretical value.

With the truck in position 2 the opening across the transverse joint below the wheel load was also measured using a Demec dial gage. Directly under the wheel load (~8.5k) the bottom of the joint opened 0.0125 inches.

6.3 Longitudinal Joint Vertical Movement during Grout Hardening

One of the concerns during the design phase had been the ability of the wet grout in the longitudinal joint to harden with a tight joint while it was exposed to vibration from traffic being carried on half of the bridge deck. In an effort to simulate this effect in the laboratory a cyclic load was applied to the test specimens as the grout cured. It was observed that under a simulated static wheel load of 9.9 kips, the joint displacments...
lessened over time as the grout cured, with specific readings of 0.015”, 0.011”, and 0.008” at 0, 3, and 6 hours after the grout was poured.

In an effort to evaluate how the previous lab tests correlated to actual vehicle induced movement, an LVDT was set up to measure the relative vertical longitudinal joint displacement under service loads on the stage 1 portion of the bridge. The LVDT was set to record 20 readings per second. Data was recorded for three 15-minute intervals after the grout was poured and is displayed in Figure 55.

![Figure 55. Vertical displacement of longitudinal joint during grout curing.](image)

The relative displacement did not reduce with curing time as noted in the laboratory tests. 95% of the peak readings were within the range of +0.05/-0.05 in. or over three times the size of displacement simulated in the laboratory testing.

Despite the larger than expected joint displacements, the grout in the field appeared to satisfactorily cure. Cracks in the surface of the grout, parallel to the longitudinal joint, developed in a manner similar to what was observed in the laboratory tests, but did not appear to have any deleterious effects on the overall joint tightness. The serviceability of the joint will rely on the post-tensioning to hold the joint tight since it was clearly pre-cracked.

### 6.4 Crack Mapping at Completion of Construction

Once the deck had been ground down in preparation for the installation of the stage 1 epoxy overlay, cracks were observed at some of the shear stud blockouts. The cracks were visible to the naked eye (eg without the use of a die penetrant). The lengths were measured using a tape measure and the widths established by comparing to a crack width gauge. The typical orientation of these cracks, as shown in Figure 56, is roughly parallel with the skew angle of the bridge and propagated from the top, left (TL) and bottom, right (BR) corners of the shear stud blockouts.

20 of the 80 stage 1 blockouts were observed to have one or more cracks present as shown by the shaded blockouts in Figure 57. In three of the 20 cracked blockouts the crack appeared to be related to the pick point for the girder. The average crack length
was 6.25 in. but the size varied from 1 in. to 26 in. complete crack descriptions can be found in Emhke (2006). Interestingly, this cracking did not appear in stage 2 panels.

Figure 56. Typical appearance of cracks at blockouts.

Figure 57. Blockout locations in stage 1 having more than 1 crack.

7. Analytical Modelling

The primary objective of finite element modeling was to examine the joint and deck behavior under load. Finding the critical bridge deck stress values in the longitudinal and transverse directions would allow identification of a minimum amount of prestressing required to limit joint opening to an acceptable amount and avoid tension in the deck.

Issa (1998) had done previous FEM work on precast deck panel bridges. A secondary objective, therefore, is to reevaluate the two bridges described by Issa. First the actual bridges will be described, the general modeling procedures and assumptions will be discussed, followed by a description of the results from the models. Finally, overall conclusions and design recommendations will be provided.

7.1 Description of Bridges Analyzed

Three different bridges: the Culpeper Bridge, Welland River Bridge and Door Creek Bridge were modeled and analyzed using SAP 2000. Linear elastic analyses were performed on the Culpeper and Welland River Bridge and both linear elastic and non-linear analyses were performed on the Door Creek Bridge.
The Culpeper and Welland River Bridges had been previously modeled and analyzed by Issa (1998). These two bridges were re-analyzed in order to verify the accuracy of the current SAP modeling method and to identify the minimum required pre-stressing level across the joints under AASHTO LRFD (2007) service loads including impact.

Then, the Door Creek Bridge was analyzed to evaluate the minimum required pre-stressing level across the longitudinal joints and transverse joints and to predict the structural behavior of the bridge under several overloading cases.

7.1.1 Culpeper Bridge

The Culpeper Bridge is a simply supported bridge 54.5 ft. in length and 30 ft. in width. This bridge is located in Virginia and maintained by the Virginia Department of Transportation. The deck is supported by five beams at 6.25 ft spacing. Two exterior beams are W33×125 with 2.5 ft. deck overhangs and the interior beams are W33×132.

7.1.2 Welland River Bridge

The Welland River Bridge carries two southbound lanes and is located near the City of Niagara Falls and maintained by the Ontario Ministry of Transportation. As described by Issa, the bridge consists of 18 continuous spans 48 ft. long and 43.5 ft. wide. Only 3 spans were constructed using the precast concrete deck panels with 8.85 inches depth. The deck is supported by four lines of steel bridge girders with sizes of W33X125 for the exterior girders and W33X150 for the interior girders. Deck span is 12 feet.

7.2 Bridge Modelling Procedures and Assumptions

Each of the bridges was modeled in 3 dimensions using the finite element method. The prestressing in the bridges was not modeled, rather the effects of live load alone were examined on the bridges to determine the level of tension stress that would develop. This then would serve as the basis for selecting the amount of prestress needed to prevent tension.

Since complete prevention of tension stress in the bridge decks may be unnecessary, further analysis was conducted using special non-linear finite element modeling techniques with the Wisconsin B-13-161 Door Creek Bridge to see what effects limited prestress and joint opening might have on the bridge system.

A brief description of the general modeling procedures will be followed by specific details. A complete description of the modeling procedures and results has been provided by Chi (2007).
7.2.1 *General Modelling Procedures*

All of the bridges examined were composed of precast deck panels supported on steel girders. In each case the individual components of the bridges were modeled. The models were all 3-dimensional representations of the actual bridges.

Two-dimensional beam elements were used to simulate the steel girders and were located at the girder centroids. Shell elements were used to represent the deck, but each deck panel was modeled as a separate unit. Truss like link elements were used to connect the panels. The deck was joined to the steel beams by short stiff beam elements to represent the shear studs. All deformations of the shear studs were considered rigid except for bending and axial extension. The parapet walls were modeled using shell elements.

7.2.2 *Culpeper Bridge Model*

The deck of the Culpeper Bridge was modeled using 16 in. by 15 in. shell elements. That sizing allowed the corner nodes of elements to match the locations of the composite shear connectors between the deck and girder.

Taking advantage of the single span bridge’s symmetry, only half of the simply supported bridge was modeled. The bridge was split longitudinally along its centerline. The model then actually simulated a case where two trucks would be placed on the bridge, one in each lane and in identical positions. Figure 58 shows the model with the parapet on the close right edge and the line of symmetry on the far left edge.

To particularly study truck wheel load effects on the joints, the truck middle axle was positioned over the transverse joint at midspan of the bridge. One wheel is centered directly between two of the girders. The truck position is also shown in Figure 58.

7.2.3 *Welland River Bridge Model*

The deck of the Welland River Bridge was modeled using 16 in. by 11.1 in. shell elements. That sizing again allowed the corner nodes of elements to match the locations of the composite shear connectors between the deck and girder.

This was a three span continuous bridge. Again half of the bridge was modeled taking advantage of a line of symmetry down the longitudinal centerline of the bridge. No information was available on the exact location of transverse joints in the bridge. As a conservative examination of possible joint opening effects under negative moment, a transverse joint was assumed to exist directly over the top of one of the interior piers.

Figure 59 shows the bridge. Truck loads were placed in various positions longitudinally on this bridge to induce maximum longitudinal bending and possible transverse joint opening.
7.2.4 Door Creek Bridge Model

Modelling of the Door Creek Bridge was slightly different because of the intent to conduct overload analysis and allow the joints to open. The panels of the Door Creek bridge were connected at the transverse and longitudinal joints with non-linear link elements that could allow the joints to open if desired. The nonlinear link’s stiffness and strength were defined by the laboratory test results described earlier in Section 3.2.

The deck was simulated using 12 in. by 13.3 in. shell elements that had a trapezoidal shape because of the skew of the bridge and deck panels. This sizing allowed the element nodal locations to match the actual shear connector locations in the bridge.

A full 3-D model of the bridge was created for the finite element method (FEM) analysis because the longitudinal joint eliminated the possibility of assuming transverse symmetry. The bridge model is shown in Figure 60.

Considerable effort was directed toward identifying the most conservative truck position to induce joint opening. The skew of the panels and joints made selection of critical truck positioning much more difficult than in the other bridges.
Figure 58. Model of ½ the Culpeper Bridge showing the truck location.

Figure 59. Model of ½ of the Welland River Bridge.

Figure 60. Model of the full Door Creek Bridge.
7.3 Bridge Modelling Results

The Primary objective of the FEM analyses was to evaluate the tension stress level in either the transverse joint or the longitudinal joint under service loading conditions for the three different bridges, considering the dynamic load allowance factor and design truck based on the AASHTO LRFD Bridge Design Specifications (2007).

The secondary objective was to determine how much overloading would be required to cause joint opening with the amount of pre-stress that actually exists in the Door Creek Bridge.

The third objective was to examine how joint opening in the Door Creek Bridge would affect the overall performance of the bridge and how the loads are redistributed as joints crack open while the loading is increased slowly toward the factored strength design loads.

7.3.1 Verification of Modelling Accuracy

Before the three primary objectives were addressed, an essential initial step was to verify the quality of the FEM modeling procedure. This was accomplished by comparing results obtained in this work with previous results reported by Issa (1998).

In the verification process the trucks were positioned in the same manner that Issa used. In some instances Issa’s loadings did not match the prescribed AASHTO LRFD or Standard Specifications design loadings. His loadings often had multiple design trucks in a lane simultaneously. Still, first these positions were matched in the present analysis for verification, then the correct truck positioning was implemented.

7.3.1.1 Culpeper Bridge Verification

The stresses calculated in the bridge deck at the maximum moment location were compared with stresses reported by previously. Issa listed the maximum longitudinal bending stress as approximately 100 psi. The present SAP analysis produced a result that was only different by 2 psi.

7.3.1.2 Welland Bridge Verification

Issa predicted a maximum longitudinal stress in the transverse joint above the pier as 272 psi of tension, without the dynamic magnifier. The SAP model predicted a stress of 250 psi. Issa used a non-standard loading with two AASHTO HS20 trucks spaced only 24 ft apart. The difference in results may be due to incorrect truck positioning in the SAP model since Issa’s position had to be estimated from his drawings.
7.3.1.3 Verification Summary

The closeness of the maximum stress results, as well as comparisons of stress variation across the bridges and moment variation across the bridges, served to verify the accuracy of the SAP modeling procedures. The SAP models could then be used with the correct AASHTO truck loadings to estimate required design prestress in the bridge decks.

7.3.2 Predicted Required Prestress Levels

An AASHTO LRFD HL-93 truck was placed on each of the bridges, with wheels in the critical location, to calculate the peak flexural tension stress that would develop in the deck joints. A single HL-93 truck was used on each bridge.

The AASHTO LRFD dynamic load allowance DLA, or impact factor, of 1.33 was also applied to the truck load. AASHTO specifies that the normal DLA should be taken as 33%, but that 75% should be used for deck joints. The interpretation of “deck joints” in AASHTO was taken to mean expansion type joints, and not joints in longitudinally post-tensioned precast deck systems. Thus the 33% factor was used here.

Though the Welland Bridge was a multi-span structure and the AASHTO truck train might be considered for creating the peak negative moment over an interior pier, with an AASHTO specified truck-to-truck spacing of 50 ft and spans of only 48 ft it was clear that the single truck loading would control.

7.3.2.1 Culpeper Bridge Tension Stress

Figure 61 shows a contour plot of the variation of top surface longitudinal stress in the deck of the Culpeper Bridge. The bottom surface stress would vary in a similar manner but with lower values.

The critical section was directly beneath the truck wheels and two elements away from the midspan of the bridge. Figure 62 plots the variation of the longitudinal deck stress along a longitudinal section taken directly beneath the wheel. Both the top surface stress and the bottom surface stress are shown. Because of the composite girder action the stresses are due to combination of deck bending and axial compression the girder top flange.

The maximum longitudinal tension stress due to truck live load in the Culpeper bridge was 120 psi without the dynamic load allowance and 160 psi with the DLA. Issa’s maximum reported stress was 100 psi without DLA. Issa, however, had the truck located in a different position that did not cause peak tension in the deck.

7.3.2.2 Welland Bridge Tension Stress

Longitudinal stress variation, perpendicular to the transverse joint, on the top of the Welland Bridge due to HL-93 truck loading is shown in Figure 63. The critical truck
location for inducing high stress over the interior fell in the first span of the bridge with
the truck back axles closer to the pier.

Figure 61. Contour plot of the deck’s longitudinal top surface stress in the Culpeper
Bridge due to AASHTO HL-93 truck loading. (+ stress = tension)

Figure 62. Variation of longitudinal deck stress along the critical section of Figure 61.

Figure 63. Contour plot of the deck’s longitudinal top surface stress in the Welland River
Bridge due to AASHTO HL-93 truck loading.
The tension stress across a transverse joint in the multi-span Welland Bridge was highest over the interior pier where deck bending was combined with deck tension since the deck acted as a top flange of the composite girder. The peak deck tension reached 150 psi under the truck loading and 199 psi when the DLA was included.

Figure 64. Longitudinal deck stress contours and truck location: Door Creek.

Figure 65. Transverse deck stress across the longitudinal joint: Door Creek.
7.3.2.3 Door Creek Bridge Tension Stress

The Door Creek Bridge was examined to determine peak stresses which could develop in both the transverse and longitudinal joints. As noted earlier, the skewed nature of the transverse joints in this bridge made identification of the critical truck load position more challenging than in the other bridges.

Figure 64 includes an indication of the critical truck position for deck stress at a transverse joint as well as a contour plot of the longitudinal stress variation on the deck top surface. The peak deck tension stress developed in the transverse joint below one of the wheel loads. The joint tension stress was 189 psi due to the HL-93 truck and 251 psi with the DLA included.

A second separate truck load position was used to predict the maximum stress in the longitudinal deck joint. In this case one line of the truck wheel loads was placed directly over the joint. Figure 65 shows the variation of stress across the longitudinal joint along the bridge length. The peak tension stress was 180 psi due to the HL-93 truck and was increased to 239 psi with inclusion of the DLA.

The actual prestress in the Door Creek Bridge was designed to provide 250 psi of compression across the transverse joints and 185 psi across the longitudinal joint. Comparing these values to the peak predicted tension values it appears that there is likelihood that the longitudinal joint could experience some degree of opening.

7.3.3 Door Creek Bridge – Joint Opening with Overload

The initial opening of any post-tensioned joint will occur as soon as the compression stress induced by the prestress is overcome by tension stresses from structural loading. The analysis results show that the longitudinal joint of the Door Creek Bridge should experience some opening under the design truck service loading (with impact).

The results of the laboratory testing of both the transverse and longitudinal joints indicated that the prestress induced compression in joints could be exceeded before the joint stiffness actually decreased significantly.

Figure 66 shows the measured laboratory behavior for a longitudinal joint with 360 psi of prestress. The major softening moment appears to be at about 135 in-k/ft. A calculation of the moment required to just overcome the 360 psi prestress gives 46 in-k/ft. Clearly the overall deck behavior really doesn’t exhibit softening with the first opening, but rather after the opening has developed.
With half as much prestress, 185 psi, the Door Creek Bridge is expected to soften earlier. The prestress would be overcome with 27.3 in-k/ft of moment. Overall “softening”, however, would not be expected until the moment reached 80 in-k/ft. This behavior is exhibited in the modeled stiffness relationship shown in Figure 67 for the analyses.

The behavior of the transverse joint under the higher prestress of 250 psi was stiffer and is also shown as modeled in Figure 67 based on the laboratory testing.

The amount of overload to cause “softening” in the Door Creek Bridge was calculated in SAP using the non-linear joint model. An increase of load to 162% of the service plus impact load was required to bring the transverse joints to the “softening” level shown in Figure 67. An increase to 159% of the service plus impact load brought the longitudinal joint to its softening condition.

Additional overloading cases were then examined to identify to what extent joint opening would develop if this bridge was overloaded. The non-linear SAP model was again used with joints allowed to open and rotate as the internal moments in the joints increased. Three cases were examined: 150%, 200% and 500% of the moments that caused first softening of the joints as listed above. The full set of load cases is described in the list
below with the two different truck locations (5&6) needed to cause opening of transverse and longitudinal joints

<table>
<thead>
<tr>
<th>Load Case</th>
<th>Input Load Factor</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Truck Loading Case 5</td>
</tr>
<tr>
<td>Service + Impact (SL+IM)</td>
<td>1.33</td>
</tr>
<tr>
<td>Overloading (OL)</td>
<td>1.5×(SL+IM)</td>
</tr>
<tr>
<td>150% Overloading (150%OL)</td>
<td>1.5×OL</td>
</tr>
<tr>
<td>200% Overloading (200%OL)</td>
<td>2.0×OL</td>
</tr>
<tr>
<td>500% Overloading (500%OL)</td>
<td>5.0×OL</td>
</tr>
</tbody>
</table>

As expected, when the wheel load amplitude increases a longer and longer portion of a joint begins to exhibit stiffness softening associated with increased crack opening.

7.3.3.1 Transverse Joint Opening

Some of the analysis results for an overloaded transverse joint are shown in Figures 68 and 69. Figure 68 illustrates softening of the joint directly under the wheel and redistribution of load transfer over a wider portion of the joint as the joint opens. The plots show the lateral distribution of the moment in the deck, but as the joint opens more moment also flows to the adjacent girders. The arrowed lines show the portion of deck that has reached the “softened” state at each load level.

The curves in Figure 68 suggest that the joint is behaving like a yielding member with the resisting internal moment staying nearly constant once the yield deformation has been exceeded. In reality the joint behaves as a non-linear elastic connection. When the loads creating the forces in Figure 68 were removed the prestress subsequently reclosed the joint rather than having a permanent offset as in a yielding member.

Figure 69 shows how the overloading and joint softening affect the vertical deflection of the bridge deck. The softening and joint opening does not appear to have a significant affect on the deflection until 200% of the softening load (or 325% of the service plus impact load) is exceeded. The ratio of displacement to load stays virtually constant, indicating constant bridge deck stiffness, until the 200% level is exceeded. Again, the stiffness is recovered with unloading – no residual deflections.
Figure 68. Effect of wheel overload on internal moments in a transverse joint.

Figure 69. Effect of wheel overload on deflection – shown across the bridge width.

Figure 70. Effect of three overloaded axles on the longitudinal joint – Door Creek.
7.3.3.2 Longitudinal Joint Opening

Overloading might be expected to have a more serious effect on the Door Creek Bridge longitudinal joint than on the transverse joint. The longitudinal joint prestress is less than the amount needed to compensate for tension stress caused by service plus impact load bending.

The degree of crack opening of a joint is directly proportional to the rotation of the joint. Longitudinal joint rotation is plotted along the length of the joint for the various load cases in Figure 70. This joint is more sensitive to overloading. The ratio of joint rotation to load stays constant until the 150% overload condition. Beyond that level the joint rotation starts increasing more rapidly. The arrowed lines in the Figure indicate the length of the joint that has reached the “softening” condition for each overload.

Figure 71 shows the deck’s vertical deflection along the length of the longitudinal joint as the overload increases. The non-linearity in stiffness is again evident. Fortunately the deflections under overload are again recovered when the loads are removed.

The length of the portion of the longitudinal joint that reached the softened condition is compared with the total joint length in the following Table.

<table>
<thead>
<tr>
<th>Load Case (text)</th>
<th>Length of Softened Longitudinal Joint (inch)</th>
<th>Bridge Span (inch)</th>
<th>Ratio (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>SL+IM</td>
<td>0</td>
<td>996</td>
<td>0</td>
</tr>
<tr>
<td>OL</td>
<td>0</td>
<td>996</td>
<td>0</td>
</tr>
<tr>
<td>150% OL</td>
<td>108</td>
<td>996</td>
<td>10.8</td>
</tr>
<tr>
<td>200% OL</td>
<td>241</td>
<td>996</td>
<td>24.2</td>
</tr>
<tr>
<td>500% OL</td>
<td>640</td>
<td>996</td>
<td>64.3</td>
</tr>
</tbody>
</table>

The longitudinal joint, with low prestress, reaches the “softening” condition with less overload than the transverse joint. It is still unlikely, however, that significant crack opening or “softening” will develop in the joint since a wheel overload equal to 160% of the design service plus impact would be required.

7.4 Summary of Modelling Results

The analyses of the Culpeper, the Welland River and the Door Creek bridges may be used to arrive at a set of observations and conclusions regarding design needs and behavior of precast deck bridges.

7.4.1 Stress in Tranverse Joints

The tension stress developed in transverse joints depends primarily on whether a bridge is single simple span or multi-span continuous. Secondary dependence is related to the deck span between girders, but the stress is less sensitive to this parameter. The stress in the transverse joints of the bridges studied is listed in the Table below along with the AASHTO LRFD specified minimum required prestress.
### 7.4.2 Stress in Longitudinal Joint

The Door Creek Bridge is the only structure that was analyzed with a longitudinal deck joint. The peak tension stress was calculated as 239 psi. None of the other bridges had a longitudinal joint and the transverse post-tensioning was not required.

### 7.4.3 Effect of Designing with Reduced Prestress

Preliminary design analysis suggested that a prestress of 350 psi would be desirable in the transverse direction. The actual prestress provided in the panels was nearly equal to that amount. The prestress provided across the longitudinal joint, however, was less than the amount needed to completely balance tension stress induced by the truck loading. The longitudinal joint prestress was only 185 psi.

Examining the effect of overload on the deck presented above, with joint behavior modeled in a non-linear elastic fashion, it is apparent that design to exactly balance the amount of tension stress induced by service plus impact loading may not be strictly necessary. Both joint rotation and vertical deflection of the bridge decks remains linearly related to load even after the joint precompression is overcome. Designing to avoid any tension under service plus impact load may be overly conservative.

Under positive deck bending moment, with the bottom of the deck in tension, it might be reasonable to aim at a design that allows the bending moment from loads to approach the “softening” moment adjusted by a safety factor. Since opening of bottom cracks may not critically affect the deck durability, small crack opening at the bottom may be tolerated.

The longitudinal joint of the Door Creek Bridge, at the middle of the deck span with positive moments, had 185 psi of prestress and a safety factor of 1.59 against softening. It had a safety factor of 0.77 against developing tension in the joint, i.e. tension opening would be expected under service plus impact loading. If the minimum prestress for transverse joints, 250 psi, had been used for the longitudinal joint design a factor of 1.04 would have existed against joint tension developing.

The transverse joints of the Door Creek Bridge had 250 psi of prestress and a safety factor of 1.62 against “softening” and 1 against developing tension stress under service plus impact positive moment bending.

Under negative deck bending moment, with the top of the deck in tension, it is more reasonable to aim at a zero tension design when selecting the prestressing. Top cracking
could allow fluid into cracks and enhance freeze-thaw deterioration. Negative moment and top cracking might occur if the longitudinal joint was placed above a girder, rather than mid-deck span between girders. The prestress in such a case should equal the expected tension stress in the deck under service plus impact loading adjusted by a safety factor.

Tension may also develop at the top of a transverse joint in the deck when the deck is part of a composite girder and the girder is experiencing negative moment, as in over a pier. Though the bending moment in the deck may be positive, the deck acting as a tension flange of the girder may experience cracking. Here again the prestress should balance the tension stress to prevent cracking.

Issa recommended design prestress levels of 450 psi in transverse joints above piers where girder negative moments could cause cracking. That value seems high in that the analysis of the Welland Bridge showed that only 199 psi of tension would develop under live plus impact loading. Issa’s high recommended value might be due to the double truck loading used, which is very conservative compared to the AASHTO LRFD loading. AASHTO LRFD specifies that a minimum prestress of 250 psi be used in transverse joint design.

AASHTO LRFD presently has no specification limits for prestressing across longitudinal joints, regardless of whether the joints are in positive or negative bending.


In many respects the design of a full depth prestressed precast deck panel system is simpler and more straightforward than cast-in-place bridge deck design, but some details are unusual and specific to prestressing or precasting. This chapter provides a draft of text that should serve as a basis for developing a section of the Wisconsin Bridge Design Manual (LRFD) focused on full depth precast bridge decks. Reference is provided to a new Manual section 17.11. An accompanying set of suggested “Special Provisions” for contract documents is included following the 17.11 Manual text.

8.1 Suggested Bridge Design Manual Text

17.11 Design of Full-Depth Precast Concrete Bridge Decks

17.11.1 General

An alternative to using cast-in-place decks is the use of full-depth prestressed precast concrete deck panels. These panels are constructed off-site under controlled
conditions and brought to the site ready for installation. Construction with precast concrete deck panels takes less time and thus creates less interruption to motorists.

Economically, full depth precast panels should be considered when there are extreme constraints on construction time or conditions of limited access to the bridge. In most situations, a bridge deck could be replaced using only night closures, or in certain circumstances staged construction can be implemented to maintain traffic in both directions. In stage construction, half of the bridge is replaced while the other half is kept open to traffic.

Other advantages include increasing work zone safety by reducing the number of and exposure time of workers operating near moving traffic, and reducing environmental impacts by minimizing the site access footprint.

The increased cost of full depth prestressed precast decks results particularly from the need for specialty sub-contractors during erection and also the generally higher cost of precast components compared to cast-in-place. The precast deck panels are joined together by field post-tensioning. This operation may require a specialty prestressing contractor and/or a specialty grouting contractor. Since the resulting road surface will generally not be smooth enough to meet requirements because of differential heights at the joints, grinding and/or placement of an overlay may also be needed.

Overall the precast deck system operates in a manner similar to a cast-in-place deck. The deck is designed to span between girders and is made composite with the girders to resist bridge bending.

A set of standardized special provisions (STSP) for the use of full depth precast bridge decks is available from the Bureau of Highway Construction, Standards Development Section.

17.11.2 Precast Full Depth Deck Layout

The overall bridge deck must be modularized into a set of precast deck panels. The primary goal is to select a panel length that will span the entire width of the bridge, avoiding the need for a longitudinal joint. If staged construction is necessary, then a longitudinal joint must be used and the panel length will be defined by the staging requirement.

Given a panel length, for best economy the panel width should be sized as large as possible. The limitations on the overall panel size are:
1) limit panel weight to 40,000 lb for truck shipping;
2) aim for panel weight ~30,000 lb to allow erection with 100 ton crane;
3) width of 12ft or less to allow shipping.

The panel width may also be affected by the total bridge length and a desire to have all panels the same width. In skewed bridges the panels should be parallelogram shaped with their ends matching the edge of the bridge deck and their sides matching the angle of the abutment. A 1 ft. gap of cast-in-place concrete, or closure pour, may be used at the deck ends near the abutments to simplify combining post-tensioning end anchor blocks with the need for expansion joint hardware as shown in Figure 17.11-1.

Design elevations for the deck should be specified in the plans at the transverse joint locations to simplify placement of the deck panels in the field.
17.11.3  Prestressing for Full Depth Precast Decks

Transverse prestressing will be created through pretensioned strands placed into the deck panels at the precasting plant. The longitudinal prestress will be created by post-tensioned strand that are inserted in ducts and tensioned after the deck panels are set in place and the joints are grouted. All joints are designed to be prestressed.

The prestressing is through low relaxation 270 ksi strands. Seven wire strand in 0.5 inch and 0.6" diameters is normally used. Strand stresses are limited by AASHTO LRFD 5.9.3 and 5.9.4 but no tension shall be allowed across joints.

The transverse pretensioned strand should be provided with 2-5/8 inch of cover. Equal prestress should be provided in top and bottom layers to resist bending with no eccentricity to avoid inducing camber into the panels.

The longitudinal post-tensioning should be provided using flat ducts that can carry three strand. Each duct should be placed at mid-depth of the deck section and should run parallel to the girders. Ducts should not be located above girders due to conflicts with connectors for developing composite deck-girder action. Special panels with end anchorages for the strands must be designed at each end of the deck. Ducts must satisfy AASHTO LRFD 5.4.6.

All post-tensioning ducts will have a minimum of two ports exiting the top of the deck. One port must be placed at each end of the duct. A third port at mid-length of the duct is desirable if the duct length is greater than 60 ft. Grout for the duct will be pumped into one port until grout is observed exiting from the next port. All post-tensioning anchorages will be recessed to allow placement of a minimum of 2 inches of cover grout for protection.

17.11.4  Precast Full Depth Deck Panel Design

The precast deck panels are designed using the same HL93 loading and deck strip widths as described in Section 17.5.1 for normal deck design. In calculating the resisting mechanism of the deck, the transverse joints are ignored and the deck is considered as a solid deck spanning across girders and broken into design strips.
The basis for transverse design of longitudinal joints is to prohibit concrete tension under Service loading. The Service 1&3 loading conditions are used for the deck and it is designed with transverse pretensioned prestress strand to limit the tension stress caused by the combination of vehicle live load and dead load to the prescribed limits [AASHTO LRFD 5.9.4]. The moments described in Section 17.5.3.1 for a cast-in-place deck are the moments used in design, but at the Service 1 and 3 states. The design sections described in Section 17.5.3.1 are also used for defining the design moments. Prestress loss must be considered in the design.

When the bridge is skewed, the panels follow the skew angle of the bridge; the transverse prestressing strands also follow the skew angle. The total prestress in the strands should be modified so that the component perpendicular to the girders still provides the required precompression in the concrete perpendicular to the girders.

In addition to the design live and dead load forces developed when the panels are supported on girders, the design should be checked to insure that the tension cracking stress of the panel [LRFD Table 5.9.4.2.2-1] is not exceeded when it is lifted using the lifting hooks for movement and placement.

Deck panel thickness should be minimized to reduce shipping weight but the minimum thickness is 8 inches and a minimum 8.5 inches should be used if the surface is to be ground after erection for smoothness.

If the bridge is built with staged construction or has a longitudinal joint, then the needed transverse prestress must be applied across that joint using post-tensioning after the panels have been placed and the joint grout has hardened.

Longitudinal distribution steel in the bottom layer, per LRFD 9.7.3.2, is not required, but is replaced by a separate requirement for longitudinal post-tensioning to maintain the integrity of the transverse joints. The prestress across all transverse joints, provided by post-tensioning, shall not be less than 0.250 ksi as required by LRFD 9.7.5.3.

The longitudinal post tensioning, however, may need to be increased above 0.250 ksi in multi-span bridges where the negative composite girder bending over a pier creates high tension stress in the deck. The longitudinal continuity deck reinforcement [LRFD 6.10.1.7] is not used. The deck is designed with prestressing to offset tension caused by composite beam action, but with a minimum of 0.250 ksi.

The normal requirements for temperature and shrinkage steel are not followed since the prestressing requirements control cracking. A light epoxy coated steel mesh, 6 x 6 W4 x W4 WWF, is desirable on the top and bottom surfaces of the panel to control stresses during shipping and handling.

Deck panels should be checked for flexural capacity under the Strength 1 limit state as required in AASHTO LRFD 5.7.3. Design for shear is not required.

17.11.5 Special Design Details

Transverse joints, and longitudinal if used, should be of a female-female form as shown in Figure 17.11-2. Joints are filled with high strength non-shrink grout. Where post-tensioning ducts are spliced at panel joints, blockouts must be provided for splicing access.
Normal mild steel reinforcing, as used in cast-in-place decks, is used at the edges or overhang of the deck to connect the parapet or other rail system to the deck.

Two elevation leveling bolts, such as detailed in Figure 17.11-3, should be placed in the panels above each girder to provide even support of the panels during construction. The elevation of the girders will likely vary significantly and adjustment for bearing is essential.

Tapered blockouts, as shown in Figure 17.11-4, are placed in the precast panels to allow connection to the girders for composite action. The blockouts are placed above the girder lines and at a preferred maximum spacing of 4 ft. to reduce the number of penetrations through the panel, but not at a spacing less than 2 ft. Shear studs are placed in the blockout when the panels are used over steel girders, rebar ties will be used in the blockouts over concrete girders.

Temporary forming will be placed below the deck panels and adjacent to the beam sides to create a grouted haunch between the deck panels and the supporting girders. The haunch will be grouted with the composite shear connector blockouts.
17.11.6 Construction Sequence

The design process and construction sequence are inter-dependent. In order to insure that all prestress is applied to the deck to compensate for load induced tension, the full deck prestressing sequence must be completed before the deck is made composite with the girders. The construction sequence should follow the steps listed below.

1) Fabricate precast deck panels and release the pre-tensioned strands once the deck has achieved the initial assumed design compression strength.
2) Check for proper fit up of panels and alignment of post-tensioning ducts and duct splicing blockouts. Allow deck panels to cure 30 days before placement.
3) Ship the pretensioned panels to the job site. Measure elevations of the bridge girders. Preset the leveling bolts on each panel before installing to meet the girder based on the measured girder elevations.
4) Set the panels in place. Adjust leveling bolts to approximately the same torque (bearing load) on each bolt and to obtain the desired deck surface profile.
5) Splice the longitudinal post-tensioning ducts at the transverse joint blockouts. Grout the transverse keyway joints between the panels and duct splicing blockouts. Do not place any haunch grout below the panels. Cure to a minimum 1ksi compression strength.
6) Apply the required post-tensioning to the longitudinal tendons. Post-tension the strands at the center of the panel width first. Then post-tension strands to either side alternately and work out towards the ends of the panels.
7) Seal the end anchorages of the post-tensioning. Grout the longitudinal post-tensioning ducts by pumping grout from one end.
8) Install the shear studs (if on steel girders). Grout the shear connector blockouts and the haunch below the panel in one operation. Place grout into a shear connector blockout until grout appears at next blockout, then move placement to that blockout.
9) Construct end expansion joints and closure pours, if used.
10) Grind and clean the deck, if required. Place deck overlay, if required.
11) If a longitudinal joint is used, steps 4-7 should then be followed for the second portion of the deck and then the following steps are completed.
11a) Couple the transverse prestressing strands from the first panels to post-tensioning strands inserted in the transverse ducts of the second panels. Cover the strands and coupling with a sleeve to prevent bonding to joint grout and to allow it to move slightly.
11b) Grout the longitudinal joint and splicing blockouts. Do not place any haunch grout. Allow cure to 1ksi compression strength.
11c) Post-tension the transverse tendons. Start at the bridge midspan and alternately to each side until reaching the ends of the span.
11d) Grout the transverse post-tensioning ducts.
11e) Follow steps 8-10 above.

8.2 Suggested Special Provisions for Full Depth Precast Decks

A. Description. This work consists of the manufacture, transportation, storage, erection and grouting of precast prestressed/post-tensioned concrete deck panels and the furnishing, installation, post-tensioning and grouting of tendons in accordance with this specification and in reasonably close conformity with the lines, grades, design, and dimensions shown on the plans. All applicable portions of Wisconsin Department of Transportation Standard Specifications (“Standard Specs”) Section 501, 502, and 503 shall apply to this Item. All post-tensioning related materials and procedures shall be as per the manufacturer’s recommendations, which shall conform to all requirements of the most recent editions of the American Association of State Highway and Transportation Officials (AASHTO), the Post-Tensioning Institute (PTI), and the American Society of Testing and Materials (ASTM). Precast materials shall meet the specifications of the PCI Manual for Quality Control for Plants and Production of Precast Prestressed Concrete Products, MNL-116. The work governed by this specification shall also include the furnishing and installation of any appurtenant items necessary for the particular post-tensioning system used, including but not limited to ducts, couplers, anchorage assemblies, supplementary steel reinforcing bars, and grout used for keyways, shear stud pockets, girder haunches and pressure grouting of ducts.

A.1 Potential Post-Tensioning Suppliers. The Contractor shall engage the services of a DOT pre-approved post-tensioning company to furnish, install, test, and prepare reports for the complete post-tensioning systems on this project. The post-tensioning system shall be from one manufacturer only.

A.2 Terms. Interpret the following terms wherever in this specification or in other contract documents:

PRESTRESSING – The application of compressive force to the concrete by stressing the strands.

POST-TENSIONING – The application of compressive force to the concrete by stressing tendons after the concrete has been cast and cured. The force in the stressed tendons is transferred to the concrete by means of anchorages.

PRE-TENSIONING – The application of compressive force to the concrete by stressing the strands before the concrete has been cast. The force in the stressed strands is transferred to the concrete by releasing this stressing after the concrete has cured.

POST-TENSIONING LAYOUT – The pattern, size, and locations of post-tensioning tendons provided in the plans.

POST-TENSIONING SYSTEMS – A proprietary system where the necessary hardware (anchorages, confining reinforcing, wedges, strands) is supplied by a particular manufacturer of post-tensioning components.

LONGITUDINAL – Applying to the direction of traffic (e.g. “longitudinal post-tensioning”, “longitudinal joint”).
TENDONS – A high strength steel member made up of a number of prestressing strands or bars in a metal or plastic duct.

TRANSVERSE – Applying to perpendicular to the direction of traffic (e.g. “transverse post-tensioning”, “transverse joint”).

STRAND – An assembly of high strength steel wires wound together. Strands usually have six outer wires helically wound around a single straight wire of a similar diameter.

WIRE – A single, small diameter, high strength steel element and, normally, the basic component of strand, although some proprietary post-tensioning systems are composed of individual or groups of single wires.

ANCHORAGE – An assembly of various hardware components, including confining reinforcement, which secures a tendon and its ends after it has been stressed and imparts the tendon force into the concrete.

WEDGES – A set of small conically shaped steel components placed around a strand to grip and secure it by wedge action in a tapered hole through a wedge plate.

WEDGE PLATE – A steel component of the anchorage containing a number of tapered holes through which the strands pass and are secured by conical wedges.

SET (ALSO ANCHOR SET OR WEDGE SET) – Set is the total movement of a point on the strand just behind the anchoring wedges during load transfer from the jack to the permanent anchorages. Set movement is the sum of slippage of the wedges with respect to the anchorage head and elastic deformation of the anchor components.

ANTICIPATED SET – Anticipated set is that set which was assumed to occur in the design calculation of the post-tensioning forces immediately after load transfer.

MEMBER – Member signifies the concrete that is to be post-tensioned.

A.3. Deck Panel Manufacturing. Fabricate deck panels to the following tolerances:

Length = +/-1⁄8 in
Width = +/-1⁄4 in
Depth = +1⁄4, -1⁄8 in
Cover = +1⁄4, -0 in
Sweep = +/-1⁄4 in
Horizontal location of shear connector block-outs = +/-1⁄4 in
Size of block-outs = +/-1⁄4 in
Variation from specified plan end squareness or skew = +/-1⁄4 in
Differential camber between adjacent members = +/-1⁄4 in
Location of handling device parallel to length of member = +/-1 in
Location of handling device transverse to length of member = +/-1 in
Bearing area (deviation from plane surface when tested with a straight edge through middle half of unit) = +/-1⁄8 in

A.3.1 Ducts. Tie in position all ducts, anchorage assemblies, and reinforcing steel. Support ducts at a spacing as specified by the manufacturer, carefully inspected and repaired before placing of the concrete. Exercise care during the placing of the concrete to avoid displacing or damaging the ducts. Duct supports shall be spaced at maximum 24” intervals, and have the capacity to hold the duct in its correct profile, prevent flotation if the duct is empty and, when appropriate, support the weight of the completed tendon assembly. Duct support, as noted above, is not required if a pretensioned strand is
carrying the duct during placing of the concrete. Duct support for longitudinal ducts, as noted above, is not required if an internal steel bar with a minimum diameter equal to 125% of the tendon is carrying the duct during placement of the concrete. The contractor at contractor’s expense shall supply any additional mild reinforcing required to support post-tensioning ducts. The tolerance on the location of the ducts shall be ±1/4” at the ends of the panel and ±1/2” within the panel. After installation in the forms, seal at all times the ends of the ducts to prevent entry of water and debris. Construction debris or other contaminants may damage the duct walls, intensify corrosion, and obstruct the placing of prestressing steel or increase friction during stressing.

If conflicts exist between the reinforcement and post-tensioning duct, in general, the position of the post-tensioning duct governs and the reinforcement is adjusted locally per the engineer’s approval.

Ducts used for post-tensioning are vulnerable to damage by crushing, bending, or impact. Such damage may constrict the area of the bore, puncture the duct or open seams such that grout is able to leak into the duct during concrete placement, and inhibit strand placement. Protect the integrity of the ducts from such damage at all times.

When cutting or joining duct components or forming holes for vents and drains, remove all protrusions and burrs, which could snag and/or scratch the prestressing steel during threading of strands or stressing. When assembling and placing reinforcement, precautions shall be taken to avoid damage to ducts which are already in place in the forms. Do not weld within 10’ of any tendons already placed in the forms.

A.3.2 Anchorage. The anchoring devices shall be recessed so that the ends of the prestressing steel and all parts of anchoring devices will have at least two inches of cover from the panel’s surface. Following post-tensioning and grouting, fill the recesses with non-shrink grout.

Anchorage pockets will be required to accommodate the anchorage assemblies for any post-tensioned longitudinal and transverse tendons. Install the anchorage assemblies on site at no additional cost.

Do not pre-place permanent post-tensioning strands during curing, storage, and shipping of precast panels. Contractor may pre-place temporary strands to support ducts during concrete placement only. Do not install post-tensioning strands until contractor erects and levels all panels.

A.3.3 Corrosion Inhibitor. Protect pre-tensioned strands that will subsequently be spliced to transverse post-tensioning strands, from unraveling at transfer by using suitable de-tensioning procedures or temporary clamping devices on the strands. Provide the strands with corrosion protection consisting of an approved oil coating immediately after transfer of pretension. Approved corrosion protection oils include: Dromus ABD (Shell/Texaco), Emulsified Cutting Oil (Shore Chemical Co.), and RustBan 310 (Esso).

A.3.4 Handling, Storage, and Shipping. Precast panel prestressing and reinforcement, as detailed in the plans, were designed to avoid cracking during storage, handling or shipping. It is the responsibility of the contractor to transfer the prestress, handle, store, ship, and erect the as-designed panels in a crack-free manner. “Crack-free” is defined as not having more than one crack every 25 square feet, the width of which does not exceed 0.008”. The contractor may provide additional reinforcement to ensure crack-free transfer of prestress and/or panel installation, which is incidental to the cost of the panels.

All storage of precast deck units either before shipment to the bridge site or at the bridge site shall be such that they are supported in a manner that will minimize development of
camber or deflection but also in a manner that will not induce forces sufficient to cause cracking. Consider supporting the precast deck units at the same locations as the lifting inserts on the opposite face.

The engineer shall inspect the finished panels for cracking and evaluate the severity of the cracks prior to panel on-site placement. Contractor at contractor's expense shall repair cracks as directed by the engineer. This inspection is independent of the inspections required by Sections 502 and 503 of the Standard Specifications.

A.3.5 Yard Fit-up. Contractor shall ensure that the precast deck panels will fit-up and align properly before shipping from the precast plant. Verification of alignment of post-tensioning ducts and block-outs is particularly critical. If pre-assembled in the yard, use blocking to simulate the support of the deck panels on the beam top flanges including differences in elevation. Verify the construction of all deck panel units in compliance with all plan requirements.

A.4. Shop Drawings. The Contractor shall submit detailed shop drawings, which include, but are not limited to:

1. Complete description of the details covering each of the precast panels and subsequent post-tensioning system to be used for permanent tendons. This shall include:
   - Designation of the specific post-tensioning steel, anchorage devices, couplers, duct material, and accessory items to be used.
   - Complete geometric layouts for each precast segment, including all prestressing strand and mild reinforcement layout, anchorage "local zone" and "general zone" reinforcement, and subsequent post-tensioning tendon configuration.
   - Properties of each of the components of the post-tensioning system.
   - Procedure and operations for prestressing, securing strands, and releasing the strands at force transfer.
   - Details covering assembly of each type of post-tensioning tendon.
   - Equipment to be used in the post-tensioning operation.
   - Step by step erection procedure of precast panels, and detailed sequence of operations for post-tensioning and securing tendons.
   - Parameters to be used to calculate the typical tendon force such as expected friction coefficients, anchor set, and post-tensioning steel relaxation curves. This should include calculations indicating all expected short-term and long-term losses, and short-term and long-term stresses.
   - Sample calculations and procedures for review of field-obtained elongations.
   - Details and sealed calculations of special reinforcing at "Local Zones" of anchorages.
   - Safety procedures to be followed.

2. Table detailing the post tensioning jacking sequence, jacking forces, and elongation of each tendon at each stage of erection for all post-tensioning.

3. The operation of grouting shear-keyways, haunches and block-outs, and end anchorages, the materials and proportions for grout, details of equipment for mixing and placing grout, and methods of mixing and placing grout.

4. The operation of grouting post-tensioning tendons, the materials and proportions for grout, details of equipment for mixing and placing grout, and methods of mixing and placing grout.

5. Calculations for supplemental reinforcement for handling, erection, and operation.
6. A complete sequence of work conforming to the maintenance of traffic as indicated in the plans.

The contractor has the right to modify the panel size, pocket spacing, or joint details from that listed in the plans. However, the contractor must submit shop drawings accurately portraying these revisions for approval, complying with the below requirements:

- The amount of prestress required in the panels shall be as specified in the plans (along the skew) per foot length of bridge.
- The shear stud block-out spacing must be greater than 2 feet and less than 4 feet.
- The shear stud intensity along the girders may be divided up into zones as indicated on the plans. Any modifications must follow this intensity layout.
- All modifications must take into account revisions to handling, storage, shipping, and erection stresses, and consequently possible revisions in the mild steel reinforcement required.
- The materials, anchorages, devices, systems, and operations shall comply with all conditions in this special provision and the design criteria as indicated on the plans.
- If the design does not comply with the above requirements, calculations and correspondence prepared by a registered Professional Engineer in the State of Wisconsin may be submitted to the engineer for approval justifying the areas of non-compliance.
- The modification shall result in no net increase in cost to the Owner, or result in an extension of the construction schedule.

Contractor shall submit all submittals sufficiently in advance of the start of construction to allow the engineer an average 45-calendar day review period, but not less than a 30-calendar day review period. The review period shall begin on the day of receipt of the submittal in the office of the engineer. All submittals not approved and requiring resubmittal shall be subject to the above review time periods, with the review time beginning anew for each such submittal. The contractor shall coordinate all submittals between his various subordinates (contractors, suppliers, and engineers) to allow for a reasonable distribution of the review effort required by the engineer at any given time. Contractor shall receive final approval before any fabrication begins. Contractor shall furnish all shop drawings as per all applicable requirements of Standard Specification Section 506.3.2. Supply manufacturer's literature where applicable. All shop drawings are to accurately detail the actual methods, materials, equipment, etc., that the contractor will be using in the field on the project. Do not deviate unless approved by the engineer.

B. Materials. The contractor shall make all arrangements to fabricate, supply, and install the precast deck panels and the post-tensioning system including all necessary incidentals for construction of the proposed superstructure.

B.1 Concrete. The contractor/supplier shall submit a concrete mix design to the engineer for approval. The contractor/supplier may use self-compacting concrete (SCC). The contractor shall receive approval of the mix design prior to the production of the panels. The contractor/supplier shall allow for a 14-day review period by the engineer after the submittal of the concrete mix design.

In addition to the requirements of the Standard Specifications, the requirements for the concrete are:
- 6000-psi, average 28-day strength.
- 5500-psi, minimum final design strength at 28 days.
- 4000-psi average strength at transfer.
- 3/8” maximum aggregate size.
- 6%-8% air entrainment.
- 0.40 maximum w/c ratio.
- Do not ship a panel until it is at least 30 days old.

The top surface of the panels is to be screeded and trowelled.

Compression test cylinders shall conform to American Standards for Testing Materials (ASTM) C 31 and the test results shall be as required by ASTM C 39. The cylinder size used for the compression tests shall be either 6”x12” or 4”x8”. The curing method used for cylinder curing shall be that of the actual production concrete panels.

After all concrete and grout work and testing has been performed, the contractor shall collect all concrete and grout test data and provide to the engineer a final concrete report containing all information discussed above and all other project concrete and grout test results including:

- The mix designs for all concrete and grout.
- Slump test results.
- Curing procedures.
- Cylinder strengths.
- Air entrainment.

B.2 Mild Reinforcing Steel. All mild reinforcing steel shall be deformed bars or welded wire fabric conforming to ASTM A615 or ASTM A185 and epoxy coated per ASTM A775 or ASTM A884.

B.3 Prestressing and Post-Tensioning Strands. Prestressing and post-tensioning strands shall be uncoated seven wires Grade 270 low-relaxation strands manufactured in accordance with AASHTO M203. Diameter of each strand shall be 0.5 or 0.6 inches as called out in plans, having an area of 0.153 or 0.217 square inches respectively. Do not splice or couple pretensioning strands/or wires.

Protect the prestressing and post-tensioning strands against physical damage and rust from the time of manufacturing to grouting. Fabricate post-tensioning strands with sufficient length beyond anchor bearing plates to allow stressing, coupling, and anchorage device installation.

B.4 Post-Tensioning Couplers. Use strand couplers only at locations specifically shown on the plans or approved by the engineer. A strand coupler shall develop at least 100 percent of the minimum specified ultimate tensile strength of the strand, tested in an unbonded state without exceeding the anticipated set. The coupling of bars shall not reduce the elongation at rupture below the requirements of the strand itself. Perform testing of couplers using samples of the prestressing bar on the project.

Couple strand couplers onto strands per the manufacturer’s directions.

B.5 Post-Tensioning Ducts. Post-tensioning duct material shall meet the requirements of AASHTO and Post-Tensioning Institute (PTI) for bonded tendons. Support the ducts as required in Section A.3.1. The Contractor shall provide additional support bars as necessary to achieve this requirement. The Contractor shall coordinate the placement of the post-tensioning ducts with the mild reinforcing steel as shown in the plans. Preference for location placement of the post-tensioning ducts shall have the following priority, first
longitudinal then transverse ducts. Protect all ducts and accessories such as clamps, bolts, nuts, screws, etc. with galvanization per ASTM A653 or an approved plastic material.

All duct material shall be sufficiently rigid to withstand loads imposed during placing of concrete and internal pressure during grouting while maintaining its shape, remaining in proper alignment, and remaining watertight.

Duct system, including duct couplers and joints shall effectively prevent entrance of cement paste or water into the system and shall effectively contain pressurized grout during grouting of the tendon. The duct system shall also be capable of withstanding water pressure during flushing of a duct in the event the grouting operation is aborted.

The interior diameter of ducts for tendons consisting of more than one strand, bar or wire shall be large enough to provide an interior area not less than 2.5 times the net area of the prestressing steel.

The contractor shall provide polypropylene plastic duct. Do not use ducts manufactured from recycled material. Use seamless fabrication methods to manufacture ducts.

The duct system and components and accessories shall be designed to transfer a force, through the duct into the surrounding concrete, equal to 40 percent of the ultimate tensile strength in a length of 2'-6". The contractor shall provide to the engineer certified test reports verifying that the duct meets specification requirements in regard to force transfer.

Furnish duct with end caps to seal the duct interior from contamination. Ship ducts in bundles. Cap and cover ducts during shipping and storage. Protect ducts against ultraviolet degradation, crushing, excessive bending, dirt contamination, and corrosive elements during transportation, storage, and handling. After incorporating the duct into the bridge component, remove the end caps supplied with the duct. Store duct in a location that is dry and protected from the sun. Storage must be on a raised platform and completely covered to prevent contamination. If necessary, wash duct before use to remove any contamination.

Construct mechanical couplers with stainless steel, plastic or a combination of these materials. Use plastic resins meeting the requirements for plastic ducts to construct plastic couplers. Use ASTM 314 stainless steel to make metallic components.

Use shrink sleeves manufactured specifically for the size of the duct being coupled consisting of an irradiated and cross linked high density polyethylene backing with an adhesive that will withstand 150 degrees F operating temperature. Install heat shrink sleeves using procedures and methods in accordance with manufacturer’s recommendations.

**B.6 Grout Vents.** Provide all ducts and anchorage assemblies for permanent post-tensioning with grout vents or other suitable connections at each end and at each side of the couplers for the injection of grout after post-tensioning. Provide a minimum of one grout vent at each end of the ducts. Vents shall be ½-inch minimum diameter standard pipe or suitable plastic pipe. Connect all connections to ducts with plastic structural fasteners. Consider applying waterproof tape at all connections including vent and grouting pipes. Plastic components shall not react with concrete or enhance corrosion of the post-tensioning steel, and shall be free of water-soluble chlorides. The vents shall be mortar-tight, taped as necessary, and shall provide means for injection of grout through the vents and for sealing the vents.

Remove vent ends after the grout has set.
All grout injection and vent pipes shall be fitted with positive mechanical shut-off-valves. Vents and injection pipes shall be fitted with valves, caps, or other devices capable of withstanding the pumping pressures. Do not remove or open valves and caps until the grout has set.

B.7 Anchorage Assemblies. Secure all post-tensioning tendons at the ends by means of permanent type anchoring devices. The contractor shall supply special reinforcement such as spirals or grids for the longitudinal and transverse tendons. The anchorage device shall be capable of developing 100% of the specified ultimate strength of the post-tensioned tendons. All anchorages, wedge plates, and bearing plates shall be galvanized per ASTM A653, or plastic encapsulated. Anchorage system, block-outs, and their details shall be compatible with the post-tensioning system used. The Contractor shall determine the required dimensions, adjust the mild steel reinforcing and panel details, and include the proposed details for approval with the shop drawings. The end zone reinforcing shall be epoxy coated in accordance with ASTM A775.

Proportion confinement reinforcement or bursting steel for the longitudinal and transverse post-tensioning anchorages in accordance with Section 5.10.9 of the AASHTO LRFD Specifications.

Secure all post-tensioned tendons at the ends by means of permanent type anchoring devices. Do not use dead end anchorages. Do not use two-part wedges for anchoring prestressing strands.

Contractor shall submit two sets of post-tensioning systems, (including anchorages, couplers, sleeves, ducts and strands), for quality assurance testing of post-tensioning anchorage devices. Assemble the test specimen in an unbonded state, and during testing the anticipated set specified by the manufacturer shall not exceed the maximum anchor set by more than 20%.

The plans do not indicate the necessary reinforcing steel required to resist “Local Zone” bursting and splitting stresses imposed in the concrete by the post-tensioning anchorage. The manufacturer shall, at his expense, design the “Local Zone” reinforcing steel to resist local bursting stresses imposed on the concrete by the selected anchorage system. The contractor shall furnish any additional steel required at no additional cost. Perform and sign the design by a Registered Professional Engineer in the State of Wisconsin who is experienced in concrete post-tensioned bridge design and construction. The “Local Zone” reinforcing shall be epoxy coated in accordance with ASTM A775.

Arrange anchorage devices for verification of the post tensioning force prior to removal of the stressing equipment.

Post-tensioning anchorage devices shall effectively distribute prestressing loads to the concrete and shall conform to the following requirements:

- The bearing stress in the concrete created by the anchorage plates shall not exceed the values per Section 5.10.9.7.2 of the AASHTO LRFD Specifications.

- Bending stresses in the plates or assemblies induced by the pull of the prestressing steel shall not exceed the yield point of the material in the anchorage plate when 100 percent of the ultimate strength of the tendon is applied, the load shall not cause visual distortion of the anchor plate as determined by the engineer.
Provide to the engineer certified test reports from an approved independent testing laboratory, verifying compliance with this requirement, for each type and/or size of anchoring device.

B.8 Post-Tensioning Grout. The grout to be used to fill the voids in tendons shall consist of Portland Cement, water and admixtures which impart low water content, flowability, minimum (anti- or low) bleeding, non-shrink and, when necessary, set retarding properties to the grout. The grout shall be PTI Class "C" (prepackaged, non-shrink) grout designed for use in an “aggressive” environment. Furnish a grout with thixotropic properties that has material, placing, and testing in compliance with the “2003 PTI Guide Specification for Grouting of Post-Tensioned Structures.”

B.8.1 Grout Components. Portland Cement in the grout shall conform to the requirements for ASTM C-150 Type I. The cement shall be fresh and not contain lumps or other indications of hydration or “pack set”. Type II cement will not be permitted without written recommendation of the grouting specialist or the subcontractor for grouting when accompanied by acceptable tests of grout and grouting procedures.

Water shall be potable, clean, and free of injurious quantity or substance (chloride, nitrates, sulfites) known to be harmful to Portland cement or prestressing steel.

Admixtures, if used, shall consist of chemicals which, when incorporated into the grout mixture, impart the properties of low water content, good flowability, minimum bleeding (sedimentation of cement), expansion or non-shrink and, when necessary, increase in setting time. Do not use admixtures containing chlorides (as CL in excess of 0.5 percent by weight of admixture assuming one pound of admixture per 94 pounds of cement) sulfites, fluorides, and nitrates. Clearly stamp the date of manufacture on each container. Do not use an admixture for which the shelf life recommended by the manufacturer has expired. Use all admixtures in accordance with the instructions of the manufacturer.

B.8.2 Grout Procedure. The contractor shall determine the specific admixtures and proportions of materials to be used to meet the requirement set out in Section B.8.3 below, and which, from prior documented experience with similar materials, equipment and placing conditions, will result in a grout which does not bleed excessively and can be effectively placed. The quantity of water in the grout shall be as low as possible, consistent with the fluidity needed for placing.

At least six weeks before grouting of the post-tensioning ducts commences, the contractor shall submit to the engineer for review and approval a “Grouting Operation Plan.” Written approval from the engineer for the proposed plan is required before grouting begins. The Grouting Operation Plan shall cover the material, placement, and related construction procedures for the post-tensioning duct grout.

The Grouting Operation Plan shall address the following:

- The names of the grouting crews and the Supervisor.
- The experience of the crewmembers and the Supervisor.
- The training/demonstrations to be provided prior to beginning operations.
- The type of equipment to be used, including capacity relative to demand.
- Working conditions of equipment, back-up equipment, and spare parts.
- Types, brands, and certifications of all materials.
- The identity of independent laboratories providing material certification.
- The procedures for the production of grout, including testing and controls.
- An estimate of the grout quantity required per group of tendons.
- The methods of controlling the rate of flow for filling of the ducts.
• The logic for providing the locations, types, and sizes of the inlet and outlet grout vents.
• The means of sealing and protecting tendons and ducts prior to grouting.
• The grout mixing and pumping procedures.
• The direction of the flow of grout and the sequence of using inlets and outlets and order for closing the vents.
• The procedures for handling blockages, including flushing of ducts.
• The procedures for possible duct repair and post-grouting repair.
• The contractor’s QC forms that are to be signed daily by the grout supervisor.

Before duct grouting operations commence, a meeting shall be held with the contractor, grouting crew, resident engineer, and engineering inspection team to discuss and understand the grouting operation plan, required testing and corrective procedures.

At least 30 days prior to beginning the first duct grouting operation, the contractor shall furnish to the engineer the results of tests performed by a laboratory approved by the engineer, demonstrating that all grout mixtures proposed for use meet the requirements of these specifications.

This information shall include a graph relating compressive strength of the grout to the age of the grout, covering ages from 3 hours to 7 days.

**B.8.3 Required Properties.** Grout shall have the following physical properties:

<table>
<thead>
<tr>
<th>Property</th>
<th>Test Value</th>
<th>Test Method</th>
</tr>
</thead>
<tbody>
<tr>
<td>Water Cement Ratio</td>
<td>Max 0.45</td>
<td>--</td>
</tr>
<tr>
<td>Fine Aggregate</td>
<td>Max # 50 Sieve</td>
<td>ASTM C33 (non-react. per App.)</td>
</tr>
<tr>
<td>Compressive Strength at 28 Days</td>
<td>Min 6,000 PSI</td>
<td>ASTM C942**</td>
</tr>
<tr>
<td>(Ave. of 3 Cubes)</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Volume Change</td>
<td>0% at 24 hrs, 0.3% at 28 days</td>
<td>ASTM C940</td>
</tr>
<tr>
<td>Initial Set of Grout</td>
<td>Min 90 Minutes</td>
<td>ASTM C953**</td>
</tr>
<tr>
<td>Bleeding</td>
<td>Max 0% at 4 hrs</td>
<td>ASTM C940</td>
</tr>
<tr>
<td>Efflux Time from Flow Cone</td>
<td>Min 11 Seconds</td>
<td>ASTM C939 ***</td>
</tr>
<tr>
<td>Freshness</td>
<td></td>
<td>AASHTO M 85</td>
</tr>
</tbody>
</table>

** The test specimen shall be prepared using the material and in the proportions which are to be used in production of grout.

*** The flow cone test shall not apply to grout that contains an admixture imparting a thixotropic consistency to the grout.
B.9 Shear-Keyway, Haunch & Block-Out, and End Anchorage Grout/Mortar. Mortar or grout for keyways, haunches, block-outs, and recesses shall be a non-shrink mortar. The mortar shall be prepared, placed, and cured per manufacturer's specifications. The mortar material shall meet the requirements of B.8.3, except that grout does not have to meet the efflux time specified. Minimum strength of the mortar shall be 1000-psi prior to applying post-tensioning longitudinally or transversely. Meet the strength requirements of the grout by conducting compressive strength tests on sample test cylinders taken during the pouring of the grout. Conduct the sample tests in accordance with ASTM C942 for the testing of the grout. Curing method(s) shall be similar to the curing method used during the construction.

At least 30 days prior to beginning the first grouting operation, the contractor shall furnish to the engineer the results of tests performed by a laboratory approved by the engineer, demonstrating that all grout mixtures proposed for use meet the requirements of these specifications.

This information shall include a graph relating compressive strength of the grout to the age of the grout, covering ages from 3 hours to 7 days, as well as the curing method used to attain these strengths.

B.10 Haunch Forming. Propose methods of forming the haunch prior to construction by the contractor. The engineer may approve these methods provided the construction procedure ensures proper consolidation of the grout, and maintains the required profile of the deck. Submit the proposed methodology no later than 4 weeks prior to delivery of the precast deck panels to the site.

B.11 Manufacturer’s Lots. The manufacturer of prestressing steel, prestress anchorages, and bar couplers shall assign an individual number to each lot of strand, wire, bar or devices at the time of manufacture. Identify each reel, coil, bundle, or package shipped to the project by tag or other acceptable means as to manufacturer’s lot number. The Contractor shall be responsible for establishing and maintaining a procedure by which all prestressing materials and devices can be continuously identified with the manufacturer’s lot number. Do not incorporate items into the construction that at any time are not positively identified by the Engineer as to lot number.

Clearly identify low relaxation strand as required by AASHTO M203. The engineer will not accept any strand not so identified.

The Contractor shall furnish manufacturer’s certified reports covering the tests required by this specification. Furnish a certified test report stating the guaranteed minimum ultimate tensile strength, yield strength, elongation, modulus of elasticity, cross-sectional area, and composition for each lot of prestressing steel. Furnish typical stress-strain curves for prestressing. Furnish a certified test report stating strength during testing, using the type of prestressing and post-tensioning steel in the construction, for each lot of post-tensioning anchorage devices and couplers.

B.12 Sampling and Testing. Do all testing in accordance with the AASHTO Specifications, unless specifically noted in these Special Provisions.

The contractor at his expense shall furnish to the engineer the following samples of materials and devices selected at locations designated by the engineer:

- One five-foot long sample of prestressing strand (for pre-tensioning and post-tensioning) from each reel or pack of strand.
- Two sets of units of each post-tensioned anchorage system.
Furnish samples at least 60 days in advance of the time they are to be incorporated into the work. The engineer reserves the right to reject any material or device that is obviously defective or damaged subsequent to testing.

C. Construction.

C.1 Sequence of Work.
1. Fabricate the deck panels. Release the pre-tensioned strands in the transverse direction once the deck panels have achieved 4000-psi minimum compressive strength, and treat extended strands with corrosion inhibitor as required in Section A.3.3.

2. Check for proper fit-up and alignment as per Section A.3.5. Verify that all precast deck panels have been constructed in compliance with all plan requirements. Allow the deck panels to cure for a minimum of 30 days prior to shipping.

3. Ship and erect the precast deck panels. Use leveling bolts to achieve the required profile grade. The tolerance to flush condition of adjacent deck panel edges shall be +/- 1/8”. Adjust leveling bolts to anticipated positions based on girder elevations prior to setting panels in place. At no time will construction equipment be allowed on deck panels until construction of the deck is complete, and the grout in the haunches and transverse joint keyways have achieved a minimum compressive strength of 1000-psi.

4. Grout the transverse shear keyways and adjoining duct block-out, after sealing the bottom of this joint with a form. Do not pre-place the haunch grout prior to stressing the tendons.

5. Stress and grout the longitudinal tendons once the shear keyway grout has achieved 1000-psi minimum compressive strength.

6. Install the shear studs (if on steel girders), and grout the haunch and shear connector pockets. Remove the protrusions of all leveling bolts and grout vents, and grout the subsequent voids.

7. Construct the end closure pours if required by plans.

8. Grind and clean the deck if required by special provisions.

9. Seal deck with an overlay if required per the special provisions. Place barriers prior to the overlay. Do not open the bridge construction stage to traffic until overlay is complete.

10. Repeat steps 3 through 6 for a second stage of construction if required, including the longitudinal tendon stressing.

11. Couple the transverse strands of second stage panels to the prestress strand of the first stage, at the longitudinal joint. Cover the strands and coupler at the longitudinal joint with a duct sleeve.

12. Grout the longitudinal joint and the adjoining tendon block-outs.

13. Post-tension and grout the transverse tendons after the longitudinal joint grout has achieved a minimum 1000-psi compressive strength.

14. Install the shear studs (if on steel girders), and grout the haunch and shear stud pockets of the second stage construction. Remove the protrusions all leveling bolts and grout vents, and grout the remaining voids.
15. Complete the end closure pour for stage 2, if required by plans.

16. Form and cast the barriers of the second stage.

22. Grind and clean the deck of the second stage if required per the special provisions.

23. Seal the second stage deck panels with an overlay per the special provisions, if required. Place barriers prior to the overlay. Do not open the bridge to traffic until overlay is complete.

Before the precast deck panels are set in place, adjust the leveling bolts to the appropriate expected extension based on elevation data for the girders and desired roadway profile. After placement of the panel, adjust the bolts to give the correct roadway profile, then torque each leveling bolt to approximately the same amount. Each bolt should have a resulting torque within 20% of the average of all bolts in a panel.

The work listed above is not all-inclusive. The Contractor is to develop the detailed sequence of work tasks to be performed and shall submit them with the shop drawings. The engineer shall obtain the work plan and all project related approvals prior deck construction.

C.2 Shear-Keyway Surfaces. The vertical and horizontal keyway surfaces shall be intentionally roughened to expose coarse aggregate by high-pressure water cleaning or similar methods immediately after removing from forms but no more than 2 days after casting. Prior to delivery on site, thoroughly clean the keyways of all dirt, dust, and other foreign matter by means of high pressure washing using a pressure of at least 1000-psi and a delivery rate of not less than 4 gallons per minute. Prior to placing non-shrink mortar, thoroughly clean the shear keyway surfaces by air blasting on site. Treat exposed concrete surfaces at block-outs and recesses with a bonding agent prior to filling. The Contractor shall maintain a sufficient quantity of expanding foam or other material on hand to plug any leaks that may occur during grouting of the shear keyways.

C.3 Protection of Post-Tensioning Steel and Hardware. Protect all post-tensioning steel against physical damage or rust at all times (from time of manufacture to grouting or encasing in concrete). The engineer shall reject post-tensioning steel that has sustained physical damage at any time. Protect post-tensioning hardware from rust and corrosion at all times (from time of manufacture until completion of the project).

Post-tensioning steel shall be packaged in containers or shipping forms for protection of the steel against physical damage, and corrosion during shipping and storage. Place a corrosion inhibitor, which prevents rust or other results of corrosion, in the package or form, or incorporate in a corrosion inhibitor carrier-type packaging material. Do not apply a corrosion inhibitor directly to the steel, except as specified in Section A.3.3. The corrosion inhibitor shall have no deleterious effect on the steel or concrete or bond strength of steel to concrete. Inhibitor carrier-type packaging material shall conform to the provisions of Federal Specification MIL-P-3420F-87. Immediately replace or restore to original condition packaging or forms damaged from any cause.

Post-tensioning steel shall be stored in a manner which will at all times prevent the packing material from becoming saturated with water and allow a free flow of air around the packages. If the useful life of the corrosion inhibitor in the package expires, immediately rejuvenate or replace it.

At the time of post-tensioning steel installation, it shall be free from rust, loose mill scale, dirt, paint, oil, grease, or other deleterious material. Removal of tightly adhering mill scale
will not be required. The engineer may reject any post-tensioning steel, which has experienced rusting that the contractor cannot remove.

The shipping package or form shall be clearly marked with the heat number and with a statement that the package contains high-strength post-tensioning steel, and care is to be used in handling. The type and amount of corrosion inhibitor used, the date when placed, safety orders and instructions for use shall also be marked on the package or form.

The contractor shall install, stress, and grout the tendons in less than a seven consecutive calendar daytime period. If the period between installation and stressing of the post-tensioning steel and grouting of the tendon will exceed seven consecutive calendar days, the contractor shall completely remove the post-tensioning steel from the duct and the engineer shall inspect for corrosion. If the engineer finds corrosion other than light surface rust (which can be completely removed by rubbing), the contractor will replace the existing pulled strand with new strand.

Wrap projecting ends of post-tensioning steel with a corrosion protection material until the final placement of cover material. Install pipes on each duct to serve as injection or vent ports during grouting. The contractor may install the transverse tendons in the ducts immediately prior to or during placement of the deck panels.

C.4 Post-Tensioning Operations.

C.4.1 Installing Tendons. Push or pull post-tensioning strands through the ducts that make up a tendon using methods that will not snag on any lips or joints in the ducts. Strands which are pushed, should have a rounded off end or be fitted with a smooth protective cap. During the installation of the post-tensioning strand into the duct, any mechanical device shall not intentionally rotate the strand.

Alternatively, consider assembling and pulling strands through the duct to form the tendon using a special steel wire sock ("Chinese finger") or other device attached to the end. Do not weld the ends of the strands together for this purpose. Round the end of the pre-assembled tendon for smooth passage through the duct. Cut strands using an abrasive saw or equal. Do not flame cut.

C.4.2 Testing of Post-Tensioning Tendons by the Contractor. For accurately determining the friction loss in a strand tendon, the contractor shall test, in place, the first two longitudinal tendons and measure the tendon force at the end opposite the jack end. Submit the test results to the engineer. Apparatus and methods used to perform the tests shall be proposed by the contractor and be subject to review of the engineer.

The contractor shall submit certified Modulus values from strand manufacturers. Submit two test reports of the "Tendon Modulus of Elasticity Test" to the engineer at least 30 days prior to installing the type of tendon. Testing by the contractor is incidental to the price paid for the precast post-tensioned deck panels.

C.4.3 Stresses in Tendons. The design of the structure is based on the assumed friction and wobble coefficient shown in the plans. The post-tensioning forces shown are for jacking forces.

Post-tension all post-tensioning steel by means of hydraulic jacks so that the force of the post-tensioning steel shall not be less than the value shown on the approved shop drawings. The maximum temporary tension stress (jacking stress) in post-tensioning steel shall not exceed 80 percent of the specified minimum ultimate tensile strength of the post-tensioning steel. Anchor the prestressing steel at initial stresses in a way that will result in the ultimate retention of permanent forces of not less than those shown on the approved
shop drawings. However, in no case shall the initial stresses, after anchor set, exceed 70 percent of the specified minimum ultimate tensile strength of the post-tensioning steel, and immediately after seating shall not exceed 74 percent of the minimum specified ultimate strength. Permanent force and permanent stress will be considered as the force and stress remaining in the post-tensioning steel after all losses. These losses include creep and shrinkage of concrete, elastic shortening of concrete, relaxation of steel, losses in post-tensioning steel due to sequence of stressing, friction and take-up of anchorages, and all other losses peculiar to the method or system of stressing have taken place or have been provided for.

The contractor shall design his post-tensioning system and installation procedures such that flushing of the ducts is not necessary. The engineer may approve flushing only in cases of emergency. If the engineer approves flushing, such as to reduce friction, the contractor may use water-soluble oil or graphite with no corrosive agents as a lubricant subject to the approval of the engineer. Flush lubricants from the duct as soon as possible after completion of stressing by use of water pressure. Water used to flush ducts may contain slack lime (calcium hydroxide) or quick lime (calcium oxide) in the amount of 0.1 pounds per gallon. These ducts shall be flushed again just prior to the grouting operations. Each time the ducts are flushed, immediately blow dry the ducts with oil-free air, and grout immediately to prevent the formation of rust.

C.4.4 Stressing Jacks. Each jack used to stress tendons shall be equipped with a pressure gauge having an accurate reading dial at least six inches in diameter for determining the jack pressure. Within 30 days prior to use for stressing on the project, calibrate each jack and its gauge as a unit by a testing laboratory approved by the engineer. Perform the calibration with the cylinder extension approximately in the position that it will be when applying the final jacking force and with the jacking assembly in an identical configuration to that used at the job site (i.e., same length hydraulic lines). Perform the calibration with the jack applying load to the testing machine. Furnish certified calibration calculations and a calibration chart, both in English units of measure, to the engineer for each jack. These certifications shall state that calibration testing was performed in accordance with specifications for this project. Pressure gauge readings are to be within three percent of the actual applied force during calibration. If pressure gauge readings are not within three percent of the applied force, determine and correct the source of error and recalibrate the gauge.

Recalibrate each jack as requested by the engineer (six-month minimum interval). The contractor may calibrate subsequent to the initial laboratory calibration, by the use of a master gauge. The contractor shall supply the master gauge in a protective waterproof container capable of protecting the calibration of the master gauge during shipment to the laboratory. The contractor shall provide a quick-attach coupler next to the permanent gauge in the hydraulic lines, which enables the quick and easy installation of the master gauge to verify the permanent gauge readings. The engineer shall possess and calibrate the master gauge for the duration of the project. If the contractor repairs or modifies the jack in any way, such as replacing the seals or changing the length of the hydraulic lines, then the contractor shall recalibrate the jack by the approved testing laboratory. Jacks and gauges shall not be interchanged without recalibration or proof loading using load cells, master gauges or other methods approved by the engineer. No extra compensation will be allowed for the initial or subsequent jack calibrations or for the use and required calibration of a master gauge.

C.4.5 Stressing of Tendons. Prior to post-tensioning any member, the contractor shall demonstrate to the satisfaction of the engineer that the post-tensioning steel is free and unbonded in the duct.
Do not apply post-tensioning forces until the concrete and/or grout has attained the specified compressive strength as evidenced by tests on representative samples of the concrete.

Stress the tendon at the anchorage(s) shown on the plans.

All post-tensioning tendons shall be tensioned by the use of equipment allowing actual elongation to be measured directly and using a hydraulic ram equipped with a method of determining the tensioning force applied using either a gauge measuring the internal hydraulic pressure in the ram or force exerted by the ram, or a load cell. Convert readings from any one of these gauges to actual tensioning forces using calibrated values from a calibration chart. All gauges shall be of sufficient size and adequately made to allow readings to an accuracy of 100-psi. Record the jack force and elongations at a minimum of five intervals (20 percent, 40 percent, 60 percent, 80 percent, 100 percent). Keep a permanent record of gauge pressures and elongations at all times and submit to the engineer.

Jacks shall be equipped with proper ports or windows for adequate visual examination and measurement of tendon movement. They shall also be capable of slow release of stress to allow relaxation from temporary overstress to the proper seating force.

Tension all tendons to a preliminary force as necessary to eliminate any take-up in the tensioning system before starting elongation readings. This preliminary force shall be between 5 and 25 percent of the final jacking force. Measure the initial force by a dynamometer or by other approved method, to use as a check against elongation as computed and as measured. Each strand shall be marked prior to final stressing to merit measurement of elongation and to ensure that all anchor wedges set properly.

Elongation tolerances for individual tendons based on friction coefficients determined in accordance with these special provisions, and based on material properties determined from laboratory tests, shall be within 7 percent (plus or minus) of the theoretical value, or as deemed necessary by the engineer.

The engineer may verify the post-tensioning force as deemed necessary. If the tendon force, measured by gauge elongation or by anchor lift-off, varies by more than five percent for one duct of strands, or 10 percent for one individual strand, check the entire operation and determine the source of error and remedy to the satisfaction of the engineer before proceeding with the work. Measure elongations to the nearest 1/16-inch.

Stress tendons in accordance with the sequence shown on the approved shop drawings. The stressing sequence shall be such that not more than one tendon will be eccentric about the centerline of a member at any time.

If strands must be pulled from both ends, as per section C.4.2, do not stress strands simultaneously by jacks at each end.

The engineer may accept multi-strand post-tensioning tendons, having wires that fail, by breaking or slippage during stressing, provided the following conditions are met:

• The completed structure must have a final post-tensioning force of at least 98% of the design total post-tensioning force.
• For precast construction that has members post-tensioned together across a common joint face, at any stage of erection, the post-tensioning force across a mating joint must be at least 98% of the post-tensioning required for that mating joint for that stage of erection.
• Any single tendon must have no more than a 5% reduction in cross-sectional area.
of post-tensioning steel due to wire failure.

If these conditions are not met, remove and replace the strand(s). Do not use previously
tensioned strands.

Cut stressing tails of tendons by an abrasive saw within ¾-inch to 1-½ -inches away from
the anchoring device, (pocket formers shall be sized to perform this process). Do not flame
cut prestressing steel. Do not cut off stressing tails of tendons until approval of the
stressing records by the Engineer.

Perform all stressing operations in accordance with AASHTO and PTI specifications under
the control of a person experienced in this type of work. The post-tensioning manufacturer
shall inspect all stressing operation work. No person shall ever stand behind an operating
jack or in front of either end of a stressed and ungrouted tendon.

Conduct the release of the transverse prestressing at the fabrication plant a minimum of 7
days after casting the precast deck panels, and after the deck panel concrete has achieved
a minimum compressive strength of 4000-psi. The post-tensioning operation for the
longitudinal and transverse post-tensioning shall be conducted a minimum of 30 days after
casting the precast deck panels, after the non-shrink grout between the panels has
achieved a strength of 1000-psi in transverse or longitudinal joints, and after the deck panel
concrete has achieved a minimum compressive strength of 6000-psi.

C.5 Post-Tensioning Supervision. A qualified representative of the post-tensioning
manufacturer who is skilled and experienced in the proposed work shall be on-site during
all stressing operations, including the transverse and longitudinal post-tensioning. The
representative shall be available for (a) inspecting and approving all post-tensioning
hardware installation at the plant prior to concrete placement, (b) stressing and anchoring
of tendons and (c) grouting operations.

C.6 Grouting. Grout the space between the duct and the tendons after completion of
post-tensioning tendon stressing and anchoring. Within 4 hours after stressing and prior to
grouting, protect the tendons against corrosion by using a plug at each end of the duct to
prevent the passage of air. Leave the plugs in place until the duct is grouted. Begin
grouting immediately after the stressing is complete and approved by the project Engineer.
All grouting operations, equipment, mixing, and material for grouting shall be in accordance
with AASHTO, PTI, and the post-tensioning manufacturer’s specifications, subject to
approval by the engineer.

Supervise, inspect, and document all grouting operations by the contractor’s supervisors
and technicians who have experience and training in grouting operations for post
tensioning. All supervisors and technicians performing the grouting work shall have
completed the American Segmental Bridge Institute (ASBI) training and certification
program for grouting “Grouting Certification Training” (GCT) Program. The grouting
supervisor shall be on site and directly oversee all grouting operations at all times during
grouting operations for both the transverse and longitudinal post-tensioning. Cost of
training and certification is incidental to the cost of precast prestressed/post-tensioned
concrete deck panels.

Successfully grout each tendon before the construction proceeds to the point where access
to the anchorages for tendon replacement becomes impractical.

C.6.1 Equipment. Equipment for batching component materials shall be capable of
accurately measuring the materials. The mixer shall be capable of continuous mechanical
mixing of the ingredients to produce a grout free of lumps with ingredients thoroughly
dispersed. Equipment for thixotropic grout shall have two identical charging/holding tank
units. Each unit alternates between duties as either a blender or a holding tank. The tank shall have a high-shear (colloidal) mixer and pump and the placing pump shall have exact pressure control capabilities. Feed the tank from the holding tank. In addition, a pressure-filter-type grout test kit is required. The contractor shall have immediate access to standby equipment as necessary to ensure that the work on the critical path is performed on schedule. The grouting equipment shall contain a screen having clear openings of 0.125-inch maximum size to screen the grout prior to its introduction into the grout pump. If a grout with thixotropic additive is used, a screen opening of 3/16-inch will be satisfactory. This screen shall be easily accessible for inspection and cleaning.

Grout pumps shall be capable of pumping the grout in a manner that complies with this special provision. Pumps shall be a positive displacement type capable of producing an outlet pressure of not less than 300 pounds per square inch and shall have seals that are adequate to prevent introduction of oil, air, or other foreign substance into the grout and to prevent loss of grout or water.

The contractor shall provide necessary standby equipment on hand during all grouting operations in the event of equipment malfunctions.

Place a pressure gauge having a full-scale reading of not greater than 300 pounds per square inch at some point in the grout line between the pumping outlet and the duct inlet.

The grouting equipment shall utilize gravity feed to the pump inlet from a hopper attached to and directly over it. The hopper must be kept at least partially full of grout at all times during the pumping operation to prevent air from being drawn into the post-tensioning duct. Under normal conditions, the grout equipment shall be capable of continuously grouting the longest tendon on the project in not more than 20 minutes.

Provide pipes or other suitable devices for injection of grout and to serve as vent holes during grouting. The material for these pipes shall be at least ½ inch inside diameter and may be either metal or a suitable plastic which will not react with the concrete or enhance corrosion of the post-tensioning steel and is free of water-soluble chlorides. These pipes shall be fitted with positive mechanical shut-off valves capable of withstanding grouting pressures. Connect all connections between a grout pipe and a duct with metal or plastic structural fasteners and taped with a waterproof tape, as necessary, to assure a watertight connection. Vents shall be marked to verify which vent is associated with a specific duct.

C.6.2 Mixing Grout. First, add water to the mixture followed by cement and fine aggregate (if used), and finally the admixture. Mix the grout in mechanical mixing equipment of a type capable of continuous mixing, which will produce a grout that is well dispersed and free of lumps. The engineer will not permit grout re-tempering. Grout shall be continuouslyagitiated until pumping. Place the grout within 30 minutes following the introduction of the admixture to the grout mixture, unless approved otherwise by the Engineer.

C.6.3 Cleaning and Flushing Tendons. Do not flush tendons and ducts with water or any other substances on this project unless ordered by the engineer.

C.6.4 Placing Grout. The Contractor shall establish a numbering system and a form for recording the grouting of each tendon. The form shall include a checklist of all applicable specifications for grouting. The form shall also include the date, a comment section for difficulties encountered, and a signature location for the inspector to indicate satisfactory grouting of the tendon. Submit to the engineer a completed and signed form for each tendon before receiving payment.
Grouting shall start at the lowest vent port with all vent holes open. The pumping pressure through the pipe shall be maintained until grout is continuously wasted at the next vent hole and until no visible slugs or other evidence of water or air are ejected and the grout being ejected has the same consistency as the grout being injected. Then close the vent valve, hold the pumping pressure momentarily, and close the valve at the injection port. Collect and dispose of the expelled grout and wastewater in a manner that will prevent contamination of the structure and environmentally sensitive areas. Remove expelled grout that has created contamination prior to payment for post-tensioning.

The pumping pressure at the tendon inlet shall not exceed 150 pounds per square inch. However, perform normal operations at 75 pounds per square inch. If the actual grouting pressure exceeds the maximum recommended pumping pressure, grouting may be injected at any vent that has been or is ready to be closed as long as one-way flow of grout is maintained. When one-way flow of grout cannot be maintained, immediately flush the grout out of the duct with water.

Do not open the shut-off valves on the pipes serving as injection ports or vent ports until the grout has taken its final set.

When it is anticipated that the air temperature will fall below 32 degrees F, verify that all ducts are free of water to avoid freeze damage to ducts. Do not grout when the temperature of the grout is below 45 degrees F. Maintain the temperature of the concrete or air surrounding the tendon at 35 degrees F or above from the time of grout placement until the compressive strength of the grout, as determined from tests on two-inch cubes cured under the same conditions as the in-place grout, exceeds 800 pounds per square inch.

Under hot weather conditions, grouting shall take place early in the morning when daily temperatures are lowest. Do not grout when the temperature of the grout exceeds 90 degrees F. It may be necessary to chill mixing water or take other special measures to lower the temperatures of the grout.

The engineer shall be physically probe or visually inspect all anchorages by 48 hours after grouting to insure that there is no bleed water. The engineer shall conduct spot inspections on one or more selected anchorages per span as long as no voids are found. Fill all voids immediately with a grout conforming to this special provision.

Reopen grouting vents at high points 10 minutes after completion of grouting and record any escape of air, water, or grout. Within 30 minutes of grouting and before the grout has hardened, check all open vents for voids. At locations where voids are observed, top off grout throughout the outlet, or perform a regrouting operation using an injection port and outlet vent. Not less than 24 hours after the completion of grouting, the level of grout at all injection port and outlet vent locations shall be inspected and topped off as necessary with freshly mixed grout. This process will continue until the engineer is assured that there are no bleed water or subsidence voids. The engineer may conduct subsequent spot inspections on one or more selected anchorages as long as no voids are found. If voids are found, then check all tendons for voids until the engineer is assured that the voids are not occurring.

After the grout has set, cut off pipes used as injection or vent ports. Cut off metal pipes two inches below the surface of the concrete. Cut off plastic pipes one inch below the surface of the concrete. Patch the resulting void with a suitable material approved by the engineer.

The contractor shall have equipment on hand that is capable of completely flushing the duct out with water in case of an aborted grouting operation. The equipment shall utilize a different power source than that used for the grouting equipment.
C.7 Protection of Post-Tensioning Anchorages after Stressing. Protect exposed ends of installed post-tensioning steel against corrosion at all times until final cover placement. As soon as possible, but not exceeding 14 days after tensioning is completed, check exposed end anchorages, strands, and other metal accessories for voids and filled, then cleaned of rust, misplaced mortar, grout, and other such materials. Immediately following the cleaning operation, the entire surface of the anchorages recess (all metal and concrete) shall be thoroughly dried and uniformly painted with epoxy bonding compound complying with AASHTO M235, Class II, in accordance with the manufacturer’s recommendations.

Immediately following application of the epoxy-bonding compound, install a tight fitting form and fill the anchorage recess with non-shrink cement based grout and coated with two coats of tar-based emulsion meeting the requirements of Federal Specification MIL-P-23236.

D. Measurement. The department will measure Precast Prestressed/Post-Tensioned Concrete Deck Panels by the per square foot for each precast deck panel completed and accepted.

E. Payment. The department will pay for measured quantities at the contract unit price under the following bid item:

<table>
<thead>
<tr>
<th>ITEM NUMBER</th>
<th>DESCRIPTION</th>
<th>UNIT</th>
</tr>
</thead>
<tbody>
<tr>
<td>SPV. ___</td>
<td>Precast Prestressed/Post-Tensioned Concrete Deck Panels</td>
<td>S.F.</td>
</tr>
</tbody>
</table>

Payment is full compensation for all labor, materials, and equipment required to detail, fabricate, construct, erect, and stress the proposed precast panels. Payment also includes shop drawings and any supplemental or alternate calculations, including “local zone” reinforcement design, shipping, and placing; post-tensioning materials such as ducts, anchorages, reinforcing steel, “local zone” reinforcing steel, and post-tensioning steel; and post-tensioning and grouting operations such as inserting, grouting, stressing, grouting and testing. All materials and work shall meet the requirements detailed in the contract plans and in this special provision.

9. Conclusions

This project has clearly shown that full depth precast concrete bridge decks can be constructed in Wisconsin. Furthermore, the fully prestressed nature of the deck, combined with the quality precast concrete, is expected to provide a deck that be more durable and will remain crack free with a longer life than conventional cast-in-place concrete decks.

The use of full depth precast panels to form a deck has the potential to achieve more rapid construction than conventional methods. The speed of erection was not proven in the prototype bridge of this project because the contractor had no incentive to complete the project in a rapid manner.
While the full depth precast system is expected to be more durable and can provide a rapid method of construction, the cost should be expected to be higher than normal construction. The system will be attractive where speed is given special weight in awarding a contract, rather than a low bid approach. A cost premium of 30% to 100% may be expected.

The prototype bridge in this project had a high cost, 270% of conventional construction, because of contractor unfamiliarity with post-tensioning, heavy use of additional subcontractors, and the requirement for staged construction with a longitudinal joint. The variation in precast panel cost alone from 2 bidders was $17/ft² or 32% of the accepted bid cost. A contractor who was not awarded the project had the lower panel cost.

A bridge with staged construction was selected as a demonstration project. The staging, however, required a longitudinal joint which alone increased the contractor’s labor time by 52% compared to the conventional bridge. A deck without a staging joint would compare much more favorably in cost with a conventional deck.

Precast deck panels can be used for partial replacement of existing bridge decks with overnight construction and traffic opening during the day. Laboratory testing proved that the panels were able to carry full truck loading at their edges without secondary support such as an end diaphragm if additional prestressing was added to the deck edges. That will allow portions of existing decks to be removed and replaced with precast deck panels and traffic to be re-introduced on a partially completed precast deck.

The AASHTO maximum spacing between shear connectors used to make decks composite with girders can be increased. The spacing was doubled in this project, to 4 ft., and in laboratory tests with no detrimental effects. The girder displayed 95% of the ideally fully composite stiffness.

The AASHTO LRFD required prestress for transverse joints of 0.25ksi is satisfactory for single span bridges. AASHTO has no specification for prestress in longitudinal joints but the 0.25ksi was also proven to be satisfactory for that location as well. When the precast deck is used on continuous multi-span bridges, however, the prestress in transverse joints will need to be increased above the AASHTO specified amount and should be calculated by the design engineer assuming the deck acts as a tension flange of the composite girder resisting the girder’s negative moment over the piers.

Laboratory testing combined with analytic results proved that it is not necessary to design the prestressing in panel joints to completely offset load induced tension stresses. As much as 62% overload, beyond the zero tension load, can be sustained without detrimental effects on the bridge deck’s performance under service loading. The AASHTO minimum prestress level may be unnecessarily high, resulting in added cost.

When the State uses post-tensioning on bridge projects it is essential that the State obtains a qualified project engineer with prestressing experience to be responsible for the project. On the demonstration bridge a series of problems were encountered because of contractor
errors that were often in violation of contract specifications. The errors arose from unfamiliarity with construction methods and an inexperienced project engineer. The project engineer needs to be an advocate for the State in ensuring that project specifications are met.

Based on the experience with this project, the WisDOT should strongly consider sponsoring a training workshop for bridge construction contractors in the State to develop familiarity and skills in prestressing and post-tensioning. Prestressed concrete has been proven to have enhanced durability and can serve as a basis for achieving the 100 year bridge. Post-tensioning will be an essential skill when the State starts using spliced precast girder technology to span longer distances. At present the State contractors are not sufficiently familiar with these techniques. In the long run the cost of construction to the State can be lessened through a proactive initiative to develop skills of the contractors.
References


Appendices

Appendix A: Original plans for Door Creek Bridge, 1961
Appendix B: Plans for new B-161 bridge.
GENERAL NOTES

DRAWINGS SHALL NOT BE SCALPD. BAR STEEL REINFORCEMENT SHALL BE EMBEDDED 2' CLEAR UNLESS OTHERWISE SHOWN OR NOTED.

THE FIRST OR FIRST TWO BOGS OF THE PLAIN, BAR MARK SHOULDS NOT BE USED. ALL SHOULDS ARE BASED ON THE EXACT ORIGINAL STRUCTURE PLANS.

SEQ EXISTING NAME PLATE FOR YEAR OF ORIGINAL CONSTRUCTION. FOR NEW NAME PLATE:

AT THE SURFACE OF ASSESSMENT ALL VOLUME WHICH CANNOT BE IN PLACE STRUCTURE SHALL BE SCALED WITH STRUCTURE BASED.

ALL FIELD CONNECTIONS SHALL BE MADE WITH X DIAMETER ARMS HIGH-FREQUENCY WIRE. ALL UNLISTED SHOPFOLDS SHOWN ON NOTES.

BENCH MARKS

<table>
<thead>
<tr>
<th>NO.</th>
<th>STATION</th>
<th>DESCRIPTION</th>
<th>ELEV</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>DSK, S.W. CORNER 0-13-130</td>
<td>868.37</td>
<td></td>
</tr>
</tbody>
</table>

LIST OF DRAWINGS

1. DECK REPLACEMENT AND WIDENING
2. DETAILS
3. WEST ALIGNMENT
4. WEST ALIGNMENT DETAILS
5. EAST ALIGNMENT
6. EAST ALIGNMENT DETAILS
7. BEARING DETAILS (WEST ALIGNMENT)
8. BEARING DETAILS (EAST ALIGNMENT)
9. FRAMING PLAN
10. STEEL DETAILS
11. SUPERSTRUCTURE CROSS SECTION
12. DECK PANEL DETAILS
13. PANEL 1 DETAILS
14. PANEL 2 DETAILS
15. DECK PANEL DETAILS
16. POST-TENSIONING DETAILS
17. SUPERSTRUCTURE DETAILS
18. EXPANSION DECK
19. SLOPE FACE PAVING
20. EXISTING GREEN

EXPANSION BEARING REMOVAL

FOR NEW BEARING DETAILS SEE 00-03-01

BENESCH
<table>
<thead>
<tr>
<th>BID ITEM</th>
<th>UNIT</th>
<th>WEST</th>
<th>EAST</th>
<th>TOTALS</th>
</tr>
</thead>
<tbody>
<tr>
<td>REMOVING OLD STRUCTURE STA 208-73,36</td>
<td>CY</td>
<td>1440</td>
<td>58</td>
<td>1900</td>
</tr>
<tr>
<td>EXPANSION FOR STRUCTURES BRIDGES STA 125-3,36</td>
<td>CY</td>
<td>26</td>
<td>24</td>
<td>50</td>
</tr>
<tr>
<td>PROTECTIVE SURFACE TREATMENT STA 208-73,36</td>
<td>CY</td>
<td>26</td>
<td>24</td>
<td>50</td>
</tr>
<tr>
<td>RUSSELL STRUCTURE EAST 100,140</td>
<td>CY</td>
<td>26</td>
<td>24</td>
<td>50</td>
</tr>
<tr>
<td>CONCRETE MASONRY BEAMS STA 125-3,36</td>
<td>CY</td>
<td>26</td>
<td>24</td>
<td>50</td>
</tr>
<tr>
<td>REMOVING BEARINGS STA 125-3,36</td>
<td>CY</td>
<td>26</td>
<td>24</td>
<td>50</td>
</tr>
<tr>
<td>ALUMINUM CUSHION CONNECTORS, 2770, EACH</td>
<td>CY</td>
<td>26</td>
<td>24</td>
<td>50</td>
</tr>
<tr>
<td>BEARING ASSEMBLIES FIXED STA 125-3,36</td>
<td>CY</td>
<td>26</td>
<td>24</td>
<td>50</td>
</tr>
<tr>
<td>BEARING ASSEMBLIES, FLEXIBLE, EACH</td>
<td>CY</td>
<td>26</td>
<td>24</td>
<td>50</td>
</tr>
<tr>
<td>PLASTIC RUBBER AND OTHER MATERIALS STA 125-3,36</td>
<td>CY</td>
<td>26</td>
<td>24</td>
<td>50</td>
</tr>
<tr>
<td>WASHED SQUEEZE MORTAR, STAKES STA 125-3,36</td>
<td>CY</td>
<td>26</td>
<td>24</td>
<td>50</td>
</tr>
<tr>
<td>FRAMING MATERIALS, STEEL, EACH</td>
<td>CY</td>
<td>26</td>
<td>24</td>
<td>50</td>
</tr>
<tr>
<td>WASHED SQUEEZE MORTAR, STAKES STA 125-3,36</td>
<td>CY</td>
<td>26</td>
<td>24</td>
<td>50</td>
</tr>
<tr>
<td>CONCRETE SURFACE REPAIR</td>
<td>SF</td>
<td>8.5</td>
<td>8.5</td>
<td>17</td>
</tr>
<tr>
<td>PRECAST, PRESSURIZED POST-TENSIONED, CONCRETE DECK PANELS</td>
<td>SF</td>
<td>52.5</td>
<td>52.5</td>
<td>105</td>
</tr>
<tr>
<td>CONCRETE DECK</td>
<td>SF</td>
<td>52.5</td>
<td>52.5</td>
<td>105</td>
</tr>
</tbody>
</table>

**STATE PROJECT NUMBER:** 1001-00-73

**SPECIAL PROVISION:**
- NEW STEEL ON/Y
- SPECIAL PROVISION

**QUANTITIES**

**SHEET 2**
CROSS SECTION THRU ROADWAY LOOKING EAST

SECTION A - TRANSVERSE P.T. DUCT DETAIL

NOTE:
1. FOR SLAB PLAN, SEE SHEET 13.
2. FOR DECK PANEL ELEVATIONS, SEE SHEET 19.
3. FOR PARAPET DETAILS, SEE SHEET 22.
4. GROUT SHEAR STUD POCKETS, KEYWAY JOINTS AND HAULWEL PAYMENT TO BE INCLUDED WITH ITEM PRECAST PRESTRESSED/POST TENSIONED CONCRETE DECK PANEL.
POST-TENSIONING NOTES:

1. An alternate post-tensioning system using a strand of different size or type of material is not allowed.
2. An alternate post-tensioning system using a different transverse post-tensioning configuration is not allowed.
3. An alternate post-tensioning system using a different longitudinal post-tensioning configuration is allowed per the notes on Sheet 1B and the Contract Documents.
4. The contractor is responsible for the design and inclusion of all post-tensioning apparatus and equipment.
5. Submit a post-tensioning stress sequence per the Special Provisions. See Sheet 1 for a suggested longitudinal post-tensioning sequence for each construction phase. See Sheet 13 and Section C-C on Sheet 18 for a suggested transverse post-tensioning stress sequence.
6. Deck panels must be permitted to slide on girders during stressing.
7. For additional post-tensioning notes, see Sheet 14.

SECTION K-K
Transverse PT coupling at Long. Jt.

NOTES:
* All ducts shall be 75/8" O.D. with 65/8" O.D.
Corrugated, 75/8" O.D. L/D or equivalent.
** All sleeves & end sleeves shall be 75/8" O.D. L/D or equivalent.
*** All couplers shall be 75/8" O.D. L/D or equivalent.

SECTION C-C PANEL 2

SECTION B-B PANEL 1

SECTION M-M
Not to scale.