Laboratory and Field Evaluation of the 24th Street Bridge

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ABSTRACT

The Iowa Department of Transportation (Iowa DOT) replaced the 24th Street Bridge in Council Bluffs, IA, which crosses over Interstate 80/29. The design includes prestressed, precast deck panels that are longitudinally post-tensioned on composite steel girders. The 24th Street precast deck panels represent a step forward from previous applications of similar systems in Iowa and strive to further improve the accelerated bridge construction design concept. The testing and evaluation for this structure consists of two primary components: a laboratory component and a field component. The laboratory testing program was intended to help answer specific design and construction questions of the Iowa DOT for the final design. The main focus of the field-testing program was to evaluate the structure both during and after construction. Results from laboratory testing and field testing are presented herein.

Key words: accelerated bridge construction—bridge—precast concrete
1. INTRODUCTION

The Iowa Department of Transportation (Iowa DOT) replaced the 24th Street Bridge in Council Bluffs, IA, which crosses over Interstate 80/29. The design includes prestressed, precast deck panels that are longitudinally post-tensioned on composite steel girders. The 24th Street precast deck panels represent a step forward from previous applications of similar systems in Iowa and strive to further improve the accelerated bridge construction design concept. The successful implementation of the project has far reaching implications for the State of Iowa, as it will allow for continuation and development of work initiated through previous IBRC projects. The project directly addresses the IBRC goal of demonstrating (and documenting) the effectiveness of innovative construction techniques for the construction of new bridge structures.

The testing and evaluation for this structure consists of two primary components: a laboratory component and a field component. The laboratory testing program was intended to help answer specific design and construction questions of the Iowa DOT for the final design. The main focus of the field testing program was to evaluate the structure both during and after construction. Results from laboratory testing and field testing are presented herein.

1.1 Bridge Description

The superstructure of the 24th Street Bridge is comprised of prestressed, precast deck panels supported on steel girders. There are 12 lines of girders spaced at 9 ft 0 in. on center, with a maximum girder length between field splices of 121.75 ft. Refer to Figure 1 for a structural steel layout of the superstructure and a girder elevation view (the label system of the girder layout is used throughout the document).

The steel girders were designed to act compositely with the deck, requiring the deck be connected to the girders through the use of shear connectors. The composite action was obtained with the use of shear stud pockets in the precast panels. Shear studs were welded on the girder within the “pockets” of the panels; the “pockets” were then filled with grout. The shear stud pockets are spaced at 2 ft along the length of the panels. The deck panels are 10 ft long x 52 ft 4 in. wide x 8 in. thick. Each panel has 28 1 in. by 3 in. embedded ducts to house the longitudinal post-tensioning strands.
Figure 1.1. 24th Street Bridge layout and elevation view
2. LABORATORY TESTING

2.1 Background

The bridge design calls for shear studs to be welded to the top flange of the superstructure girders within a preformed deck panel “pocket” to provide composite action between the precast panels and the girders. A constructability concern of the pocket configuration needed to be addressed; thus, a mock-up was created to test the constructability of the pocket size and grout flowability.

In order to longitudinally post-tension the deck panels, ducts are installed in the deck panels, which must then be connected in the field prior to stringing the post-tensioning strands. The design requires the ducts in adjacent precast panels be joined by a duct coupler and be sealed with a waterproof covering in order to protect the ducts from infiltration of grout when the transverse joints are cast. To test the integrity of this coupler and waterproof covering, a mock-up of the system was made and the performance verified.

The precast panels require the transverse edges of the panels be “roughened” for shear resistance prior to grouting the transverse joints. Four different alternatives were tested to determine the most effective surface treatment for shear resistance: Control, Diamond Plate Form, Chemical Etching, and Sandblasting. Recommendations for surface treatment are made based on the test results.

2.2. Laboratory Test Results

2.2.1 Shear Stud Pocket Investigation—Bend Test and Grout Flowability

To investigate the ability to perform the necessary bend test in the specified stud pocket, a mock-up of two successive stud pockets was created out of plywood with a piece of plate steel simulating the beam top flange. The mock-up was also used to investigate the ability of grout to flow through the stud pockets into the haunch between the precast panels and the steel girder top flanges. The haunch was taken as the minimum allowable haunch. Six studs were welded to the plate steel as specified in the plans.

No difficulty was found in bending the four studs in the corners of the stud pocket. Bending of the center two studs was more arduous but still possible. The grout was then placed into the two stud pockets in the mock-up and agitated with an electric vibrator. Figure 2.1 shows the mock-up after removal of the forms, clearly indicating that grout can sufficiently flow through the stud pockets and into the haunch area in the specified dimensions.
2.2.2 Evaluation of Duct Splicing Performance

The 1 in. x 3 in. duct splice connection detail was evaluated to determine if grout or moisture would seep into the duct at the connection. Two mock-up duct splices were constructed and placed in grout. One duct splice was constructed of Polyken waterproof duct tape, as shown in Fig. 3.2a. The second splice mock-up was constructed with butyl rubber strips wrapped around the joint interfaces of the duct and coupler; the longitudinal joint of the coupler was sealed with a strip of Polyken duct tape, as shown in Fig. 3.2b. Both methods of grout proofing the splices were found acceptable.

2.2.3 Evaluation of the Influence of Surface Treatment on Transverse Joint Shear Transfer

The following four surface “roughening” alternatives were tested and evaluated: Control (i.e., no roughening), Diamond Plate forms, Chemical Etching, and Sandblasting. For each alternative, three specimens were tested, each specimen consisting of a 6 in. x 6 in. x 6 in. grout cube sandwiched between
two concrete cubes of similar dimensions. Push-out tests were performed on each of the specimen. Figure 2.3 illustrates a typical deflection vs. load curve for the sandblasted specimen.

![Figure 2.3. Deflection vs. load plot for Sandblasted specimen #3](image)

A progression in higher shear bond strength is evident as one moves from the Control specimens to Diamond Plate specimens to Chemically Etched specimens to Sandblasted specimens. Sandblasting appears to be the most effective surface treatment for resistance to pure shear.

### 2.3 Laboratory Testing Conclusion

Three laboratory tests were conducted to evaluate design and construction issues for the 24th Street Bridge in Council Bluffs, IA. The tests consisted of evaluating the shear stud pockets, (including the stud bend test and grout flowability), duct splicing performance, and the influence of surface treatment on transverse joint shear transfer. The following conclusions were made from the laboratory testing:

1. No difficulty in installing the shear studs in the precast panel pockets was foreseen by the contractor or encountered by the research team.
2. Conducting the bend test on the studs in the precast panel pockets was feasible for all six studs.
3. Grout with the proposed slump can sufficiently flow through the stud pockets into the haunch areas. It is anticipated that air will remain trapped in these areas. The impact of the voids is not known.
4. The waterproof duct tape and butyl rubber methods of grout proofing the duct splices were both acceptable.
5. Sandblasting the surface of the concrete/grout joint was the most effective surface treatment for resistance to shear.
3. FIELD TESTING

3.1 Background

The Iowa State University Bridge Engineering Center in conjunction with the Iowa DOT developed the monitoring and evaluation plan for the bridge. The plan entailed investigating prestressed and post-tensioning strand corrosion, panel joint pressure during post-tensioning, strains during deck panel handling, and static live-load performance of the completed bridge.

3.2 Field Test Results

3.2.1 Corrosion Monitoring

The six prestressed strands were instrumented with Vetek V2000 corrosion monitoring systems during panel fabrication, and six sacrificial post-tensioning strands were instrumented during bridge construction. The corrosion was checked after the panels were placed on the girders and approximately six months later. No corrosion is evident on any of the strands at this time.

3.2.2 Handling Performance of Precast Deck Panels

To better understand how the precast panels are impacted during handling and placement, two panels were instrumented with strain gauges. These panels were monitored from the time the panels were picked up in the staging yard until placement on the bridge. Figure 3.1a shows the field layout of the strain gauges on the panel. Figures 3.1b and 3.1c show the panels being picked up from the staging yard and moved onto the bridge girders. The strain data obtained from the handling of the panels has not been evaluated for performance.
3.2.4 Diagnostic Live-Load Test

A diagnostic live-load test was conducted on the completed bridge to compare structural performance with expected design performance. Strain gauges and deflection transducers were installed on critical superstructure members, and semi-controlled vehicle loads crossed the bridge. Strain gauges were placed at the pier, mid-span, and abutment of the north span. Deflection transducers were placed at the mid-span of the north span only. Accelerometers were also placed at mid-span and quarter-span of the bridge north span. A fully loaded three axle dump truck was driven over the bridge. The transverse position of the truck was varied with six different load cases.

3.2.4.1 Deflection Transducers

Deflection transducers were installed on the bottom flange of the seven east girders. Representative time-history deflections for Load Case 2 at the north span mid-span with respect to front axle position are shown in Figure 3.2. The load truck was located in the center of the east driving lane for Load Case 2. In general all load cases produced the same deflection shape for the bridge with positive deflection when the truck was on the south span and negative deflection when the truck was on the north span. For all the load cases a deflection range of -0.32 to +0.21 in. was seen when the truck was on the south span and a range of -0.4 to +0.23 in. when the truck was on the north span for the seven girders. The maximum north span deflection of -0.41 in. corresponds to a span to deflection ratio of L/5120.
3.4.2.2 Strain Gauges

A total of 36 strain gauges were placed on the bridge, with 20 located at mid-span and 8 located at both the pier and north abutment. The cross-sectional strain gauge configurations are shown in Figure 3.3. Figure 3.4 represents the time-history strain for all the girders at the mid-span bottom flange location. Figure 3.5 shows the strain at girder J having a strain gauge configuration of #3. The top flange strain and the strain on the bottom of the slab are very similar, indicating composite action between the girder and slab is taking place. The strains also indicate the natural axis is near the interface of the slab and girder. The full range of strain measured for all gauges can be seen in Table 3.1. The largest strain range was at the mid-span and bottom flange location with a range of 104 $\mu$e.

![Figure 3.3. Cross-sectional strain gauge configuration](image)

Figure 3.2. Representative time history deflections for Load Case 2
Figure 3.4. Representative time-history strain for mid-span bottom flange load location, Load Case 2

Figure 3.5. Girder J cross-sectional strain, Load Case 2

Table 3.1. Range of strain at various locations

<table>
<thead>
<tr>
<th>Location</th>
<th>Bottom Flange</th>
<th>Top Flange</th>
<th>Bottom of Slab</th>
</tr>
</thead>
<tbody>
<tr>
<td>Abutment</td>
<td>-4 to +14</td>
<td>-5 to +6</td>
<td>NS</td>
</tr>
<tr>
<td>Pier</td>
<td>-16 to +3</td>
<td>-1 to +5</td>
<td>NS</td>
</tr>
<tr>
<td>Mid-span</td>
<td>-22 to +66</td>
<td>-5 to +5</td>
<td>-2 to +6</td>
</tr>
</tbody>
</table>

NS—No strain gauge at location
3.3 Field Test Conclusion

The field testing of the 24th Street Bridge in Council Bluffs, IA, consisted of two components: evaluation of the handling performance of the precast deck panels and evaluation of the performance of the deck under semi-controlled loading conditions (including deflection and strain measurements). Based on the information obtained thus far from the field testing, the following conclusions were determined:

1. The prestressing and post-tensioning strands monitored for corrosion have indicated no active corrosion taking place.
2. The north span deflection of the bridge ranged from -0.41 in. to +0.21 in., which corresponds to a span to deflection ratio of L/5120.
3. The strain over various location on the girders and slab ranged from -22 to +66 µε, with the largest strain being located at the on the bottom flange of girder K.