PRECAST, BIAXIALY PRESTRESSED
CONCRETE BRIDGE DECKS
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SYNOPSIS

This bulletin describes a project directed toward development of a precast, biaxially prestressed deck system adaptable to both deck replacement and new construction. The method of fastening the deck to the supporting girders was an integral part of the investigation.

The section on CONNECTION TESTS discusses five types of connections studied and the decision to use steel ties or studs protruding into deck blockouts as represented by Specimens BD-4 and BD-5 in the deck test. Shear friction procedures can be used to design these connections. The concrete infilled blockouts, however, did mar the wearing surface of the deck and created cracking problems.

The test specimen discussed in the DECK TEST section is depicted in Figure 4 on page 16. Precast prestressed planks pretensioned in their longitudinal direction (the transverse direction of the deck), and post-tensioned together in the direction of traffic (the longitudinal direction of the deck) were produced and constructed readily, indicating minimal disruption to traffic. The epoxy joints between planks combined with post-tensioning has adequate load carrying capability and insures continuity of the slab. The capacity of the test deck was 6.4 times larger than the design dual wheel load for AASHTO HS 20-44 including impact.

Although the CONCLUSIONS section discusses some unresolved problems, the overall result of this project indicates a strong potential market for precast, biaxially prestressed concrete bridge decks.
INTRODUCTION

This CTA project was developed in response to the need for replacement decks for highway bridges. The concept was to identify the best precast prestressed concrete system for replacement decks. If a successful system can be developed for replacement decks, then new construction will also be an appropriate application of that system.

Most existing highway bridge decks are cast-in-place concrete. Performance has not been entirely satisfactory to date, as evidenced by the need for replacement often after as little as seven years of service. The methods of construction, the materials used, and severe service conditions encountered lead to rapid deterioration. Most bridge decks are cast in exposed locations during the summer so that wind and high temperatures make finishing of stiff, high quality mixes very difficult. Long concrete hauls are frequently required. Forms may deflect, and reinforcement be displaced by workmen. In general, conditions are not favorable for achieving the high quality concrete needed for durability.

In service, bridge decks are subjected to a combination of live load stresses and abrasion from traffic, and in many parts of the country, to the action of deicing salts. Ordinary reinforced concrete cracks due to its basic structural action or temperature effects. Deicing salts penetrate into the cracks and eventually contact the reinforcing bars, leading to corrosion of the bars. Expansion of the products of corrosion generates internal stresses in the concrete leading to further cracking, spalling, and accelerated deterioration.

One logical answer to the durability problem is to provide the best concrete available -- plant precast, biaxially prestressed concrete. Such concrete is produced under favorable conditions with plant quality control and adequate curing. The biaxial prestress prevents cracking and the penetration of deicing salts. Precasting permits greater speed of erection and the advantage of being able to "walk your way" along the bridge, replacing the deck as you go. In addition, the disruption to traffic is minimal, which is a vital economic factor.
SCOPE

PROGRAM GOAL

The overall goal of the project was to develop a precast prestressed deck system that would be adaptable to both deck replacement and new construction for most steel and concrete multibeam bridges.

DESIGN CRITERIA

Two basic criteria were adopted for the precast system:

- The precast, biaxially prestressed concrete would be the wearing surface of the deck.
- The system would be designed for composite action with the supporting girders.

A literature survey was made, and it was found that several previous studies had used precast plank systems designed for composite action. Some had decks pretensioned in the long direction of the planks, and some had decks used as the finished wearing surface. However, no previous system used longitudinal post-tensioning to fasten the planks together to create a biaxial prestress condition in the deck.

SYSTEM INVESTIGATED

Based on the results of the literature review and a consideration of production and erection possibilities, it was decided that the system would consist of precast prestressed planks running the full width of the bridge and about 8 ft wide in the direction of traffic. The planks would be pretensioned in their longitudinal direction (the transverse direction of the deck), and post-tensioned together in the direction of traffic (the longitudinal direction of the deck). Epoxy would be used in the transverse joint between planks with no shear key provided. The method of fastening the deck to the supporting girders and the constructibility of the system were integral parts of the investigation.
EXPERIMENTAL PROGRAM

The experimental program consisted of two phases:

- Connections tests - A series of push-off tests was conducted to examine different methods for fastening the deck to the supporting beams.

- Test deck - A full-scale portion of a bridge was assembled and loaded as a proof test of the system. The connection between the precast deck and the steel supporting girders consisted of studs embedded in concrete infilled blockouts cast in the deck. Steel supporting girders were used because they represented the most likely first application of the precast, biaxially prestressed concrete bridge deck concept. The design for the connection between the deck and girders was based on the results of the connection tests.
CONNECTION TESTS

The literature study did not reveal a connection that could be characterized as the optimum for composite action between a steel or concrete girder and a biaxially prestressed precast concrete deck. Therefore, eleven push-off tests were performed on five different types of specimens to identify the limitations for connections made by either clamping or direct fastening of the deck to the girder and by embedment of connectors from the girder into concrete infilled deck blockouts. (1)

SPECIMENS

Details for the typical BD-5 specimens are shown in Figure 1. Those specimens simulated conditions for the connection between a new or replacement deck, Block A, and a composite steel girder, Block B. Properties of all eleven test specimens are given in Table 1. Details of the five connection types tested are illustrated in Table 2. All specimens were loaded as shown schematically in Figure 1.

Each specimen contained two connections. The center-to-center spacing of those connections in the direction of loading was 9 in. (228.6 mm) for all specimens except BD-5-2 for which the spacing was 10 in. (254 mm). No. 4 Grade 60 reinforcing bars were used for all specimens. A36 steel was used for the 3/4 in. (19 mm) diameter bolts and plates of BD-1, 2, and 3. The nominal shear and tensile capacities of the 3/4 in. (19 mm) diameter wedge bolts used in BD-2 was 17 kips (75.6 kN). The nominal yield strength of the 1/2 and 3/4 in. (12.7 and 19 mm) diameter steel studs used in BD-3 and BD-5, respectively, was 50 ksi (345 MPa).

The specified strength of concrete and grout was 6,000 psi (41.4 MPa). Concrete strengths were obtained from tests of 4 in. by 8 in. (100 by 200 mm) cylinders. Grout strengths were obtained from tests on 2 in. (50 mm) cubes.
### Dimensions

<table>
<thead>
<tr>
<th>Specimen</th>
<th>a (in)</th>
<th>b (in)</th>
<th>c (in)</th>
</tr>
</thead>
<tbody>
<tr>
<td>BD-5-1</td>
<td>6</td>
<td>9</td>
<td>6</td>
</tr>
<tr>
<td>BD-5-2</td>
<td>5 1/2</td>
<td>10</td>
<td>12</td>
</tr>
</tbody>
</table>

**Figure 1:** Typical specimen, BD series.
<table>
<thead>
<tr>
<th>Block A</th>
<th>Block B</th>
<th>Grout</th>
<th>Block-out</th>
<th>$C_c$ (psi)</th>
<th>Roughness on Shear Plane</th>
<th>Area of Shear Plane (in²)</th>
<th>$F_u$ Ultimate Shear Force (lb)</th>
<th>$A_s$ Shear Rein. (in²)</th>
<th>$f_y$ Specified Yield Strength (psi)</th>
<th>$p_f$</th>
<th>$v_u$</th>
<th>$\mu$ Apparent Coefficient of Friction</th>
<th>Remarks</th>
</tr>
</thead>
<tbody>
<tr>
<td>BD-1-1</td>
<td>5729</td>
<td>5729</td>
<td>---</td>
<td>---</td>
<td>A smooth, B troweled</td>
<td>420</td>
<td>23.7</td>
<td>---</td>
<td>---</td>
<td>---</td>
<td>56.4</td>
<td>0.5</td>
<td>bolt axial force = 11 kips</td>
</tr>
<tr>
<td>BD-1-2</td>
<td>5729</td>
<td>5729</td>
<td>---</td>
<td>---</td>
<td>A sandblasted, B sandblasted</td>
<td>420</td>
<td>21,800</td>
<td>---</td>
<td>---</td>
<td>---</td>
<td>51.9</td>
<td>6.4</td>
<td>= 15 kips</td>
</tr>
<tr>
<td>BD-1-3</td>
<td>5729</td>
<td>5729</td>
<td>3519</td>
<td>---</td>
<td>A sandblasted, B sandblasted</td>
<td>380</td>
<td>---</td>
<td>---</td>
<td>---</td>
<td>---</td>
<td>?</td>
<td>?</td>
<td>= 15 kips</td>
</tr>
<tr>
<td>BD-1-4</td>
<td>5729</td>
<td>5729</td>
<td>unknown</td>
<td>---</td>
<td>A sandblasted, B sandblasted</td>
<td>380</td>
<td>38,700</td>
<td>---</td>
<td>---</td>
<td>---</td>
<td>95.2</td>
<td>---</td>
<td></td>
</tr>
<tr>
<td>BD-2</td>
<td>5729</td>
<td>5729</td>
<td>3008</td>
<td>---</td>
<td>A sandblasted, B sandblasted</td>
<td>380</td>
<td>50,600</td>
<td>---</td>
<td>133</td>
<td>---</td>
<td>---</td>
<td>---</td>
<td>shear capacity of wedge bolt = 17 kips</td>
</tr>
<tr>
<td>BD-3</td>
<td>4948</td>
<td>4948</td>
<td>2638</td>
<td>---</td>
<td>A smooth, B troweled</td>
<td>420</td>
<td>76,900</td>
<td>0.784</td>
<td>50,000</td>
<td>93.4</td>
<td>153</td>
<td>1.95</td>
<td></td>
</tr>
<tr>
<td>BD-4-1</td>
<td>unknown</td>
<td>unknown</td>
<td>2044</td>
<td>5953</td>
<td>A smooth, B 1/4&quot; roughness</td>
<td>380</td>
<td>78,800</td>
<td>0.80</td>
<td>60,000</td>
<td>126</td>
<td>202</td>
<td>1.6</td>
<td></td>
</tr>
<tr>
<td>BD-4-2</td>
<td>6643</td>
<td>6643</td>
<td>6827</td>
<td>5143</td>
<td>A smooth, B troweled</td>
<td>380</td>
<td>75,500</td>
<td>0.80</td>
<td>60,000</td>
<td>126</td>
<td>199</td>
<td>1.6</td>
<td></td>
</tr>
<tr>
<td>BD-4-3</td>
<td>7000</td>
<td>7000</td>
<td>6288</td>
<td>5820</td>
<td>A sandblasted, B 1/4&quot; roughness</td>
<td>380</td>
<td>99,500</td>
<td>0.80</td>
<td>60,000</td>
<td>126</td>
<td>262</td>
<td>2.1</td>
<td></td>
</tr>
<tr>
<td>BD-5-1</td>
<td>6877</td>
<td></td>
<td>5467</td>
<td>5506</td>
<td>A smooth</td>
<td>210</td>
<td>168,400</td>
<td>3.53</td>
<td>50,000</td>
<td>841</td>
<td>802</td>
<td>0.35</td>
<td></td>
</tr>
<tr>
<td>BD-5-2</td>
<td>6455</td>
<td></td>
<td>5559</td>
<td>5767</td>
<td>A smooth</td>
<td>278</td>
<td>160,000</td>
<td>3.53</td>
<td>50,000</td>
<td>635</td>
<td>647</td>
<td>1.0</td>
<td></td>
</tr>
</tbody>
</table>

**TABLE 1:** Properties of specimens for shear connection tests.
<table>
<thead>
<tr>
<th>CONNECTION</th>
<th>ADVANTAGES</th>
<th>DISADVANTAGES</th>
</tr>
</thead>
<tbody>
<tr>
<td>CLAMPED</td>
<td>1. No disturbance of slab surface.</td>
<td>1. Poor appearance.</td>
</tr>
<tr>
<td></td>
<td>2. Accepts various misalignments.</td>
<td>2. Requires work under deck.</td>
</tr>
<tr>
<td></td>
<td>3. May loosen in service.</td>
<td>3. May loosen in service.</td>
</tr>
<tr>
<td></td>
<td>4 Specimens</td>
<td></td>
</tr>
<tr>
<td>BOLTED</td>
<td>1. No disturbance of slab surface.</td>
<td>1. Requires much skilled labor under deck.</td>
</tr>
<tr>
<td></td>
<td>2. Accepts various misalignments.</td>
<td>2. Poor appearance.</td>
</tr>
<tr>
<td></td>
<td>3. May loosen in service.</td>
<td>3. Low capacity of expansion bolts.</td>
</tr>
<tr>
<td></td>
<td>4. Fatigue could be a problem.</td>
<td></td>
</tr>
<tr>
<td></td>
<td>1 Specimen</td>
<td></td>
</tr>
<tr>
<td>WELDED</td>
<td>1. No disturbance of slab surface.</td>
<td>1. Requires much skilled labor under deck.</td>
</tr>
<tr>
<td></td>
<td>2. Accepts various misalignments.</td>
<td>2. Welding may be unsightly.</td>
</tr>
<tr>
<td></td>
<td>3. May loosen in service.</td>
<td>3. Suitable for new construction only.</td>
</tr>
<tr>
<td></td>
<td>1 Specimen</td>
<td></td>
</tr>
</tbody>
</table>
| CONCRETE TO      | 1. All work from above.                                                     | 1. Durability of fill material and disturbance of stresses around blockouts may be in question.
| CONCRETE BLOCKOUT| 2. Accepts various misalignments.                                           | 2. Filled blockout has appearance of patches.                                |
|                  | 3. Easy to complete.                                                        | 3. Suitable for new construction only.                                        |
|                  | 3 Specimens                                                                 |                                                                               |
| CONCRETE TO      | 1. All work from above.                                                     | 1. Durability of fill material and disturbance of stresses around blockouts may be in question.
| STEEL BLOCKOUT   | 2. Accepts various misalignments.                                           | 2. Filled blockout has appearance of patches.                                |
|                  | 3. Studs are easy to locate and install.                                     |                                                                               |
|                  | 2 Specimens                                                                 |                                                                               |

TABLE 2: Summary of comparisons (other than loading) among specimens for shear connection tests.
In the four specimens of series BD-1 the effect of varying the clamping force between the deck and girder was examined. Each specimen was made by modification of the one tested previously. The contact surfaces for the interfaces between the blocks were treated as shown in Table 1. The interfaces for BD-1-1 and BD-1-2 were left ungrouted while BD-1-3 and BD-1-4 had a 1/2 in. (12.7 mm) thick grout layer placed between the blocks. Each of the four bolts of the connection were torqued to clamping forces of 11, 14.7, 14.6, and 0 kips (48.9, 65.3, 64.9, and 0 kN) for BD-1-1, BD-1-2, BD-1-3, and BD-1-4, respectively.

Specimen BD-2 used 3/4 in. (19 mm) diameter wedge bolts to fasten the steel angles to Block B which represented an existing concrete girder. A 1/2 in. (12.7 mm) thick layer of grout was injected between the blocks and between the steel angles and Block B.

Specimen BD-3 represented new construction. Welding was used to directly fasten together the embedded anchor plates of Blocks A and B. A considerable amount of the grout injected along the interface leaked out.

The three specimens of the BD-4 series represented new construction. Ties protruded from Block B into grouted blockouts cast in Block A. A 1/2 in. (12.7 mm) thick grout layer was provided between the two blocks. The main variable between specimens was the roughness of the contact surfaces. The sides of the blockouts were coated with epoxy before grout was injected into the interface and the blockouts.

The two specimens of the BD-5 series were similar as apparent from Figure 1. Studs protruded from the steel beam into grouted blockouts in the concrete slab. The surface of the steel beam was sandblasted and a 1/2 in. (12.7 mm) thick layer of grout placed along the interface. Additional reinforcement and greater edge distances were provided for BD-5-2 than BD-5-1 in order to prevent the concrete shear cone failure that occurred around the studs nearest to the edge of Block A for BD-5-1.
All specimens simulated full-scale conditions for the connection between a deck designed for HS 20-44 loading, and girders placed approximately 7 ft on centers and spanning approximately 100 ft. Proportions for the connections were chosen after consideration of AASHTO-1977 requirements.\(^2\) Strength considerations dictated the size and spacing of the ties for the concrete-to-concrete connection of the BD-4 series. By contrast, fatigue considerations dictated the size and spacing of the studs for the steel-to-concrete connection of the BD-5 series. Thus, code requirements dictated a design shear capacity much greater for the BD-5 series than that for the BD-4 series.

Careful observations were made of the relative ease of construction, likely ability to compensate for misalignments on the job site, and appearance of each type of connection. The advantages and disadvantages of each connection are shown in Table 2. For each specimen, measurements were made of the relations between the shear force applied along the interface and the average of the slips between Blocks A and B for each side and each end of the interface.

**RESULTS**

Load-slip curves for all eleven specimens are shown in Figure 2.

Specimens BD-1-1 and BD-1-2, without grout along the interface, developed large slips at relatively low loads and had relatively low capacities. The greater torque applied to the bolts of BD-1-2 and the sandblasting of the interface surfaces caused no marked changes in the load-slip curve for BD-1-2 compared to that for BD-1-1. Strain gages attached to the bolts showed that the bolt forces did not increase significantly with loading. For BD-1-3, loading was discontinued at 76.8 kips (341.6 kN) with no failure having occurred. When the specimen was unloaded, the maximum slip of 0.0045 in. (0.1143 mm) was completely recovered. Specimen BD-1-4 was BD-1-3 with the clamping force in the bolts removed. The prior loading had no effect on the load-slip curve for BD-1-4 until failure occurred suddenly in the grout at 39.7 kips (176.5 kN).

Specimen BD-2, with wedge bolts, failed suddenly at a capacity of 68.8 kips (306 kN) and a slip of 0.004 in. (0.1016 mm). The capacity then decreased rapidly with increasing slips as indicated by the broken line in Figure 2. After removal of the load, it was found that one bolt had sheared.
Figure 2: Summary of load-slip curves.
The rate at which slips developed with increasing loads was greatest for BD-3 with the connection made by welding. That lack of stiffness was attributed to imperfect bonding of the interface as a result of leakage during the grouting operation. Failure was due to excessive slips at a load of 76.9 kips (342 kN). At failure there were several wide cracks surrounding the studs in Block B.

For the BD-4 series, simulating connections by ties to a concrete girder, specimens BD-4-1 and BD-4-3 had initial stiffness comparable to BD-1-3, BD-1-4, and BD-2. Specimen BD-4-1, with intentional roughness on the interface surface of Block B and a 2,000 psi (13.79 MPa) grout, developed a maximum capacity of 105 kips (467 kN) at a slip of about 0.02 in. (0.05 mm). A shearing failure occurred suddenly between Block A and the grout and the capacity then decreased with increasing slips to a plateau at about 77 kips (342.4 kN). Specimen BD-4-2, with no intentional roughness on the interface surfaces and a 6,000 psi (41.4 MPa) grout, exhibited a considerably lower initial stiffness than BD-4-1. However, the maximum capacity of 75.5 kips (335.8 kN) developed at a slip of 0.70 in. (17.7 mm) was similar to that for BD-4-1. For specimen BD-4-3, the roughness of the interface was similar to that of BD-4-1 and the strength of the grout similar to that of BD-4-2. The behavior was similar to that for BD-4-1. A sudden shearing failure occurred at a capacity of 153.1 kips (680.9 kN) and a slip of 0.05 in. (1.27 mm), and then the capacity decreased with increasing slips to a plateau at about 99 kips (440.3 kN).

The BD-5 specimens, simulating connections by studs to a steel beam, showed initial stiffness comparable to those of the BD-4 specimens. Specimen BD-5-1 failed in a brittle manner at 168.4 kips (749 kN) when the concrete surrounding the studs nearest the end of Block A spalled. For BD-5-2, with a greater distance from the studs to the end of Block A, the behavior up to about 70% of the ultimate capacity of 193.3 kips (859.7 kN) was similar to that for BD-5-1. BD-5-2 failed by shearing of all eight studs at a slip of 0.4 in. (10.16 mm) and a capacity of 180 kips (800.6 kN).
DISCUSSION

Transfer of shear across the interfaces was accomplished by a combination of four mechanisms: cohesion, mechanical resistance caused by roughness, dowel action, and frictional resistance provided by clamping. The test results were consistent with experience from shear friction tests. The strength of the connections at high slips increased with increase in the pfy value for the reinforcement securely anchored on both sides of the shear plane. A strength at low slips, exceeding that at high slips, and considerably less dependent on the pfy value, was developed when there was cohesion and a roughened shear plane. As illustrated by the results for BD-4-1 and BD-4-3, for connections with similar roughnesses, the maximum capacity, the slip at that capacity, and the capacity at large slips, increased as the strength of the grout increased. As illustrated by the results for BD-4-2 and BD-4-3, the maximum strength, and the stiffness of the connection at service loads, increased with increase in the roughness of the interface. A complete grout layer was needed to develop the full potential stiffness of a connection. However, the strength of that layer had little effect on the stiffness.

Use of the shear-friction procedures described in the PCI Design Manual provided a reasonably conservative estimate of the measured strengths. By those procedures the shear strength, $V_f$, is evaluated from the expression:

$$V_f = \phi \mu A_s f_y$$

where $\mu = 1.0$ for $pf_y \leq 600$ psi

and $\mu = \frac{300}{pf_y} + 0.5$ for $pf_y > 600$ psi

$\phi = \frac{300}{pf_y}$ the capacity reduction factor taken as 0.85.

$A_s$ = the total area of the ties crossing the shear plane.

$p$ = the reinforcement ratio for those ties.

$f_y$ = the yield stress of ties.

$pf_y$ should be $\leq (0.6 f'_c - 530)$ psi.
Consideration should also be given to the strength being limited by a full or partial shear cone failure in the concrete surrounding the connectors. To reduce the possibility of shear cone failures for a concrete-steel composite system that uses welded studs, the length to diameter ratio for the studs should be not less than four, with adequate reinforcement and edge distance provided for the concrete surrounding those studs.

In practice, repeated traffic loads and temperature gradients can cause cracking along even a roughened and grouted shear plane. However, the likelihood of continuous interface cracking decreases as the roughness increases. Hence for an adequately roughened and bonded plane, it is probably sufficient to require that the $f_y$ value for the reinforcement crossing the shear plane insure a capacity at high slips exceeding the capacity required according to strength design concepts. In contrast, if the interface surfaces are not roughened, or are roughened but not properly bonded, then continuous interface cracking from early in the loading history is likely and the $p$ value for the reinforcement crossing the shear plane should probably be selected based on repeated load criteria.

The experience gained during the construction of the test specimens and the behavior of the specimens demonstrated that connections such as BD-4 and BD-5, made by extending steel ties or studs into concrete infilled blockouts in the deck, were preferable to the connections made by clamping, welding, or bolting. Adequate ultimate capacities, working load stiffnesses, and fatigue characteristics can be readily obtained with blockout connections. However, those connections also have marked disadvantages. Blockouts mar the appearance and disturb the smoothness of the precast wearing surface. The concrete within the blockouts is non-prestressed and therefore the potential durability of the deck diminished.
DECK TEST

If concrete infilled blockout connections are used, the most likely first application of a precast, biaxially prestressed concrete deck will be for a composite steel and concrete bridge. To confirm the feasibility of such a system, to identify the problems inherent in the construction, and to determine the structural response of the system, a full-scale portion of a prototype bridge was assembled and subjected to a variety of loading tests. In particular, the load-carrying capacity of the transverse joints between planks and the structural participation of the cast-in-place concrete in the blockouts were evaluated.

TEST SPECIMEN

An overall view of the test deck is shown in Figure 3 and details of the reinforcement in Figure 4. The transverse prestressing consisted of two layers of five 1/2 in. (12.7 mm) diameter 270 k (1,200 kN) strands per panel at a spacing of 20 in. (508 mm). The longitudinal prestressing consisted of 5/8 in. (15.9 mm) diameter Dywidag bars placed centrally within the deck at the spacings shown in Figure 4. For moments evaluated by AASHTO procedures, the nominal tensile stresses created in such a deck by a 16,000 lb wheel load would be about \(6\sqrt{f_{c}}\) and \(3\sqrt{f_{c}}\) for the transverse and longitudinal directions, respectively. In practice more appropriate stress levels would be \(3\sqrt{f_{c}}\) and \(6\sqrt{f_{c}}\). Higher stress levels were used in the test deck to permit better evaluation of the effects of cracking and to simplify construction difficulties. Deformed bars were added at the locations shown in Figure 4 to provide handling reinforcement for the panels and post-cracking protection at critical sections of the deck and the corners of the blockouts. The bars in the longitudinal direction were placed in a single layer at mid-depth of the panel. Those in the transverse direction were placed in two layers with 2 in. (50 mm) of concrete cover. The latter reinforcement was omitted from around the blockouts in one of the panels in order to study the consequent effects.

Blockouts of the same size as those for specimens BD-4 and BD-5 as shown in Figure 1 were provided with two different center-to-center spacings, 20 in. (508 mm) and 10 in. (254 mm).

The strengths specified for the concrete and grout were 6,000 psi (41.4 MPa) at the time of test.
Figure 3: Schematic view of test deck.
Figure 4: Plan view of arrangement of blockouts and reinforcement.
Two slightly different joint details were used between planks. As shown in Figure 5, one joint was made with provision for a short grade transition zone between planks to help accommodate misalignments and one joint was made flush. The coupling arrangement for the longitudinal tendons is shown in Figure 6. The elongations in the partially stressed tendons used to take up the joints were maintained by shims. Those shims replaced the nuts that would otherwise have been needed. That action reduced the size of the housing needed to accommodate the coupling. In a long bridge, the coupling length required to allow for elongations may be substantial.

**PRODUCTION OF PANELS**

After placing and pretensioning the strands, deformed bars, ducts for post-tensioning, and the other items necessary to make the panels, the concrete was cast, vibrated externally, and cured in the forms under a polyethylene sheet for 48 hours. Figure 7 shows those panels at the time the concrete was cast. The concrete consisted of 11% by weight Type III cement, 32% sand, 51% gravel, and 6% water. The maximum diameter of the gravel was 1/2 in. (12.7 mm). The panels and nine 4 in. diameter by 8 in. long (100 mm by 200 mm) test cylinders were cast and cured in the same manner. At 48 hours, the polyethylene sheet was removed and the strands released. The compressive strength was 2,460 psi (17.0 MPa). Cracks developed from the edge to one-third of the length of the panel along two of the strands for the upper layer of strands in Panel 2. The panels were removed from the form and stored in the laboratory for approximately 1 month until the time of assembly, shown in Figure 8.

**CONSTRUCTION**

Setting Support Girders: The steel beams available for this project were W10 x 66, too small to provide composite action with the deck under load. Therefore the steel beams were prevented from deflecting vertically by providing continuous support on concrete beams grouted to the laboratory floor. The top surfaces of the steel beams were sandblasted. Measurements showed the maximum difference in height between neighboring steel beams prior to placement of the deck to be 7/16 in. (11 mm).
Figure 6: Detail for coupling Dywidag bar.
Figure 7: Casting concrete bridge decks.

Figure 8: Placing concrete bridge decks.
Sealant: Rounded 1 in. (25 mm) diameter polyethylene foam strips were placed along both edges of the top flanges of the steel beams. Those strips were large enough to compensate for the maximum difference in height between beams and to provide a space for grouting between the deck and beam. Attachment of the foam was by package sealing tape at 3 to 4 ft (0.9 to 1.2 m) intervals.

Placement of Planks and Nominal Compression of Epoxy Joints: Panel 1 was set to the correct position using steel shims placed on top of the beams. Those shims insured at least a 1/4 in. (6 mm) gap between the deck and the beams and vertical alignment of the joint. Due to space limitation in the laboratory, the six 5/8 in. (15.9 mm) Dywidag bars were threaded through Panel 1 protruding the width of Panel 2. Panel 2 was threaded over these protruding bars, and landed on the shims supporting Panel 1. In the field the Dywidag bars could have been threaded through 2 or 3 panels after they were positioned. Panel 2 was positioned with the joint between Panels 1 and 2 open about 6 in. (152 mm). Adhesive Engineering Epoxy 1001 LPL, a product of relatively low viscosity, was applied to the open joint. Panel 2 was then moved close to Panel 1, and the joint closed by stressing the two short bars located 5 ft 6 in. from the edge of the panel. The compression on the joint was about 50 psi (0.34 MPa). Panel 3 was set in place in much the same way as Panel 2. The epoxy applied to the joint between Panels 2 and 3 was Adhesive Engineering Type 1180.

The epoxy used in the joint between Panels 1 and 2 tended to run out, requiring frequent refilling and wasting of expensive material. This could be controlled by sealing the bottom with foam taped to the concrete during assembly. The epoxy used in the joint between Panels 2 and 3 was more viscous. As a result, a portion of the joint around one of the post-tensioning ducts contained voids and grout leaked from the duct.

Final Post-Tensioning: Final post-tensioning was done 24 hours after completion of the epoxy joints. Tendons were jacked in pairs: first the two center bars, then the outside bars, and finally the two bars that had been used for temporary stressing. The final compression on the joint was about 150 psi (1.03 MPa). The operations involved in threading, coupling, and prestressing the Dywidag bars were simple and efficient.

Installation of Studs: To simulate the BD-5 test, four studs were placed in one blockout using a standard stud welding gun operating through the blockout. It was found that ease of construction would have required slightly larger blockouts, so three studs were used in all of the other blockouts in the deck. Stud installation was done concurrently with curing of the joints without any evidence of disturbing the epoxy.
Grouting of Post-Tensioning Ducts: The post-tensioning ducts were grouted immediately prior to grouting under the planks, to utilize the same pump for both operations. The grout consisted of 69% by weight Type I portland cement, 30% water, and 1% expanding grout agent. It was found that all but two ducts were blocked at one or more locations. On demolition, the blockages were found to be due to intrusion of epoxy from the joints. A positive means for keeping epoxy out of the ducts is needed, such as foam O-ring seals at the duct joint. It is common practice to swab ducts out when epoxy is used.

Grouting Under the Planks: The 1/4 in. (6.35 mm) space between the top of the girder flange and the bottom of the plank was grouted using a bent 1/4 in. (6.35 mm) diameter tube attached to the base of a standard grout pump. Grout was injected at each blockout in turn until it flowed out from under the plank at the next blockout. The operation proceeded quickly and smoothly.

Infilling of Blockouts: Six blockouts in Panel 1 were filled with an epoxy mortar consisting of one part 1001 LPL epoxy and 1.2 parts silica sand. This procedure resulted in a concave surface detrimental to the wearing surface. The sides of the remaining blockouts were coated with 1001 LPL epoxy to bond the wet concrete, and then filled with a concrete consisting of 16% by weight Type III portland cement, 37% sand, 51% 3/8 in. (9.5 mm) diameter pea gravel, and 6% water. The operation did not proceed smoothly. The resulting surface was porous, rough, and discolored. In spite of the use of the epoxy bonding agent on the inside of the blockouts, a barely visible shrinkage crack occurred around the cast-in-place concrete in every blockout. An expansion agent should have been added to the mix to avoid the shrinkage problem.

LOADING

Setup: A simulated axle loading consisting of two concentrated loads spaced 7 ft (2.13 m) on centers acting through 8 in. by 12 in. (200 mm by 304.8 mm) pads was applied in two different locations. Location 1, shown in Figure 9, was at the center of Panel 2. Location 2, shown in Figure 10, was immediately adjacent to one of the joints. During the first cycle of loading, the maximum load per point was 15 kips (66.7 kN), well below the cracking load. In the second cycle, the load was taken at each location successively to well above cracking. Finally, the loading was moved to Location 1 and increased until failure occurred in the slab.

Instrumentation: Thirty mm wire strain gages were mounted on the top and bottom of the planks as shown in Figure 11. During the test to destruction, deflections were measured by precise leveling.
Figure 9
Loading at Location 1

Figure 10
Loading at Location 2
Odd numbers from 33 through 69, even numbers from 58 through 64, and 70 indicate gages on the top surface of the deck.

The remaining numbers indicate gages on the bottom surface of the deck.

Figure 11: Strain gage layout.
RESULTS

Pre-Cracking Behavior:

- Participation of Concrete in Infilled Blockouts – Strains on both the top and bottom surfaces of the concrete deck along the edge of the top flange of the center girder for 15 kip (66.7 kN) loads applied at Location 1 are plotted in Figure 12. The degree of structural participation of the concrete in the blockouts was about 60% of that of the precast concrete for the tension side, and almost zero on the compression side. Abrupt changes in strain occurred near the boundary of the blockouts.

At time of testing, the concrete in the blockouts had a compressive strength of 6,880 psi (47.4 MPa) and a split tensile strength of 660 psi (4.55 MPa). The precast concrete of the deck had a compressive strength of 4,900 psi (33.8 MPa) and a split tensile strength of 600 psi (4.14 MPa).

- Ratio of Strains – It is apparent from Figure 12 that the tensile strains over the center girder for the loading at Location 1 were always larger than the corresponding compressive strains except for gage 49. By contrast, when the loading was applied at Location 2, the tensile strains over the center girder at Location 2 were almost the same as the compressive.

The maximum ratio of the longitudinal tensile strain to the transverse tensile strain for the center of the deck's span was 49% for loading at Location 1 and 68% for loading at Location 2. The ratio predicted for an elastic, homogeneous slab is 83%.
Figure 12: Strains over center girder for 15 K pre-cracking loads at Location 1.
Cracking - First cracking in the longitudinal direction for the positive moment region, as indicated by non-linearity in the strain versus load curves plotted in Figure 15, was defined to have occurred at a load of 15 kips (66.7 kN) per point for Location 1 and 24 kips (106.7 kN) at Location 2. The predicted cracking load was 18 kips (80.0 kN).

Tensile strain versus load relationships for the negative moment region are shown in Figure 16. First cracking was defined to have occurred at the load of 19 kips (84.5 kN) per point for Location 1 and 38 kips (169.0 kN) for Location 2. The predicted cracking load was 34.5 kips (153.4 kN).

At Location 1, hairline cracks opened first in the boundaries of the blockouts and then developed from the corners of those blockouts toward neighboring blockouts parallel to the longitudinal line of the girder.

Post-Cracking Behavior: Figure 19 indicates that after cracking, the tensile strains in the precast concrete over the center girder increased rapidly while the tensile strains in the cast-in-place concrete decreased to about 50\% of the values before cracking.

TEST TO DESTRUCTION

The test to destruction of the deck with loads applied at Location 1 was terminated by a punching failure, shown in Figure 20, at a load of 132 kips (587.1 kN) per point. The load deflection relationship at the center of the deck is shown in Figure 21. That relationship indicates that general yielding had not occurred prior to collapse.
Figure 13: Tensile strains at center of deck span before cracking.

Figure 14: Tensile strains over center girder before cracking.
Figure 15: Tensile strains at center of deck span up to cracking.
Figure 16: Tensile strains over center girder up to cracking.
Figure 17: Tensile strains at Location 1 after cracking.

Figure 18: Tensile strains at Location 2 after cracking.
Figure 19: Strains over center girder for 15 K post-cracking loads at Location 1.
Figure 20: Failure surface after punching on test to destruction of deck.
Figure 21: Deflection in the center of deck on test to destruction.
DISCUSSION

Influence of Concrete Infilled Deck Blockouts: The lower structural participation of the concrete in the deck blockouts could have been due to a lower stiffness for the epoxy coating on the interior surfaces of the blockouts and to an absence of reinforcement crossing the blockouts. That reduced structural participation resulted in lower compressive strains and higher tensile strains for loadings at Location 1 than at Location 2. The nature of the strains indicates that significant axial tensile stresses (membrane stresses) developed in the deck. Abrupt changes in strain occurred around the blockouts nearest the load.

The reduced participation of the concrete in the blockouts resulted in considerably lower cracking loads for loadings at Location 1 than at Location 2. That behavior is undesirable from the standpoint of the likely resistance of the deck to attack by deicing salts. Blockouts less crowded than those in the test deck would reduce the membrane effect, and a reduction in the sharpness of the corners of the blockouts would be helpful in reducing abrupt stress changes at the blockouts. However, little increase in the center-to-center spacing of the blockouts is possible if fatigue considerations control the design of the shear connection.

The capacity of the test deck was 6.4 times larger than the design dual wheel load for AASHTO HS20-44, including impact.

Transverse Joints: As apparent from Figures 13 through 18, there was little difference in the behavior before and after cracking at Location 2. It is apparent that an epoxy joint combined with post-tensioning has adequate load carrying capability and insures considerable continuity of the slab.
CONCLUSIONS

The following conclusions are drawn from the literature studies, design studies, and test results reported in this bulletin and in Reference 1:

1) For the types of shear connection likely for precast bridge decks, a shear friction procedure predicts behavior and strengths in reasonable agreement with those measured. The strength of such connections at large slips increases with increase in the \( p_{fy} \) value for the reinforcement that crosses the shear plane and is securely anchored on both sides of that plane. A strength at low slips, exceeding that at high slips and less directly dependent on the \( p_{fy} \) value for the reinforcement, is developed when there is cohesion as well as a roughened shear plane. The stiffness of the connections at service loads increases markedly as the roughness of the interface increases.

2) Connections, such as BD-4 and BD-5, made by extending steel ties or studs into blockouts in the deck, are easier to construct and exhibit a better behavior than connections made by clamping, welding, or bolting. However, there are marked disadvantages associated with such connections. The concrete infilled blockouts mar the appearance and disturb the smoothness of the precast wearing surface. This concrete is non-prestressed. The blockouts cause stress concentrations, and early cracking of the concrete around the blockouts is likely. That cracking further reduces the effectiveness of the infilled concrete. The blockouts also diminish the longitudinal prestress in the concrete over the girder. Considerable tensile stresses are likely at the same location due to the steel girders restraining the long-term deformations of the concrete. Those long-term effects will also lower the cracking load. Cracking increases the possibility of deicing salt attack.

3) The development of easier installation methods for concrete-steel composite systems using high strength bolts should be investigated. That connection is likely to possess more favorable structural characteristics than those of the BD-4 and BD-5 specimens, especially for fatigue loadings.\(^{(5,6)}\)
4) For concrete-concrete composite systems, further investigations are needed to determine realistic criteria for the proportioning of shear connections for fatigue loadings. Further work is also needed to develop connections that effectively connect a new deck to an existing girder.

5) A segmental precast, biaxially prestressed deck system with epoxy dry joints and longitudinal post-tensioning by Dywidag bars is easy to construct. Care in applying epoxy to the joints and grouting the ducts will assure a quality product.

6) Prestress in the longitudinal direction proved to be beneficial for the load carrying capability of the joint between panels. Calculations also show that such prestress can markedly reduce the likelihood of deck cracking due to external loads, temperature difference, or long-term shrinkage effects.
REFERENCES


6. Salmons, John R., Private communication, University of Missouri -- Columbia.
