

# Composite behavior of precast concrete bridge deck-panel systems

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Full-depth precast concrete bridge deck panels can be used in place of a cast-in-place concrete deck to reduce bridge closure times during deck replacements and bridge construction. Panels are prefabricated at a precast concrete plant under controlled casting and curing conditions and then transported to the bridge site. At the jobsite, the panels are set in place on the girders and adjusted to the correct elevation with leveling bolts. These bolts extend through the panels and bear on the top flange of the girders. A panel-to-panel connection is then created in one of several ways, including narrow female-to-female keys filled with grout and post-tensioned, wider mildly reinforced joints filled with grout or concrete, or male-to-female joints cemented with epoxy and post-tensioned.

After the panels are connected, a composite connection to the girders is made. Composite action between deck and girders is provided by shear connectors that extend out of the girder and into shear pockets formed in the panels. The shear connectors are clustered at the shear pockets instead of being spaced uniformly along the length of the girder, as is typical with cast-in-place concrete decks. The shear connectors normally consist of either hooked reinforcing bars with prestressed concrete girders or shear studs with steel girders.

The discrete locations of the shear connectors for precast concrete deck-panel systems cause questions to be raised about the proper method of designing for horizontal shear transfer. Greater pocket spacing is desirable because it results in fewer blockout forms that have to be placed during fabrication and less grout that has to be placed during installation. Fewer blockouts reduce the number of areas

## Editor's quick points

- Precast concrete bridge deck panels can be used in place of a cast-in-place concrete deck to reduce bridge closure times for deck replacements.
- This paper presents experimental and analytical studies of different shear connector types and various pocket spacings to investigate the composite behavior of precast concrete bridge deck-panel systems.
- Based on the research, the paper recommends construction details and proposes design practices that account for the variables that make precast concrete bridge deck-panel systems unique.

that tend to cause durability problems and may result in shorter construction delays.<sup>1</sup> Current design provisions do not address the design of shear connectors for precast concrete bridge deck-panel systems.<sup>2,3</sup>

Some researchers have begun to investigate the horizontal shear capacity with full-depth precast concrete panels. Component tests were performed on steel girders with traditional shear-stud spacing requirements. The number of studs and configuration of shear studs were found to affect the load capacity.<sup>4</sup> Other researchers have investigated the maximum pocket spacing on steel girders. Component tests were performed with shear-stud pockets, and the results promoted extending the maximum spacing to 48 in. (1200 mm).<sup>5</sup> These component tests revealed the need to investigate full-scale systems with shear studs grouped together. In addition, there has been no research focused on precast concrete girders with hooked reinforcing bars in shear pockets.

## Objectives

The research program was developed to address the challenges and problems related to the design and behavior of shear connectors for precast concrete deck panels on prestressed concrete I-girders. Based on results, current design provisions and practices can be improved and modifications to code provisions can be made, if necessary.

The first objective was to study the composite action of the system. Typical hooked reinforcing bars and a new detail with shear studs were both considered in the test program. The new shear-stud detail is discussed in the section “Design of the Lab Mock-Up.” Both cyclic and overload tests were performed. The strains in the shear connectors and the vertical deflections of the system were used as the primary indicators for the level of composite action. The influence of shear-pocket spacing was also examined. Finite-element studies were also conducted to aid in making more-general conclusions about the composite action of the system.<sup>6</sup>

The second research objective was to examine the constructability of the system. A bridge consisting of precast concrete deck panels and precast, prestressed concrete girders was built. This bridge is called the lab mock-up. The construction process was well documented. Particular attention was paid to the types of shear connectors, pocket spacing, casting tolerances, and construction time.

## Research program

Experimental and analytical research programs were developed to accomplish the two objectives.<sup>6</sup> The experimental research program consisted of static and cyclic tests on a simply supported, full-scale bridge built in the Virginia Polytechnic Institute and State University Structures Laboratory. The research program included a detailed construc-

tability study, a creep and shrinkage study, and a durability study for the transverse joints. This paper focuses on the static and cyclic tests to evaluate the shear-connector system, as well as the component of the constructability study dealing with the shear connectors. The analytical program consisted of finite-element analyses using the commercial software DIANA.<sup>6</sup>

## Design of lab mock-up

The design was based on a 40-ft-long (12 m) simply supported bridge with five girder lines spaced at 8 ft (2.4 m) center to center. Other design parameters were as follows:

- American Association of State Highway and Transportation Officials’ *AASHTO LRFD Bridge Design Specifications* HL-93 Loading
- Deck and girder concrete 28-day strength: 6000 psi (41 MPa)
- Deck and girder release strength: 4000 psi (28 MPa)
- Prestressing strand: 1/2-in.-diameter (13 mm), 270 ksi (1860 MPa)

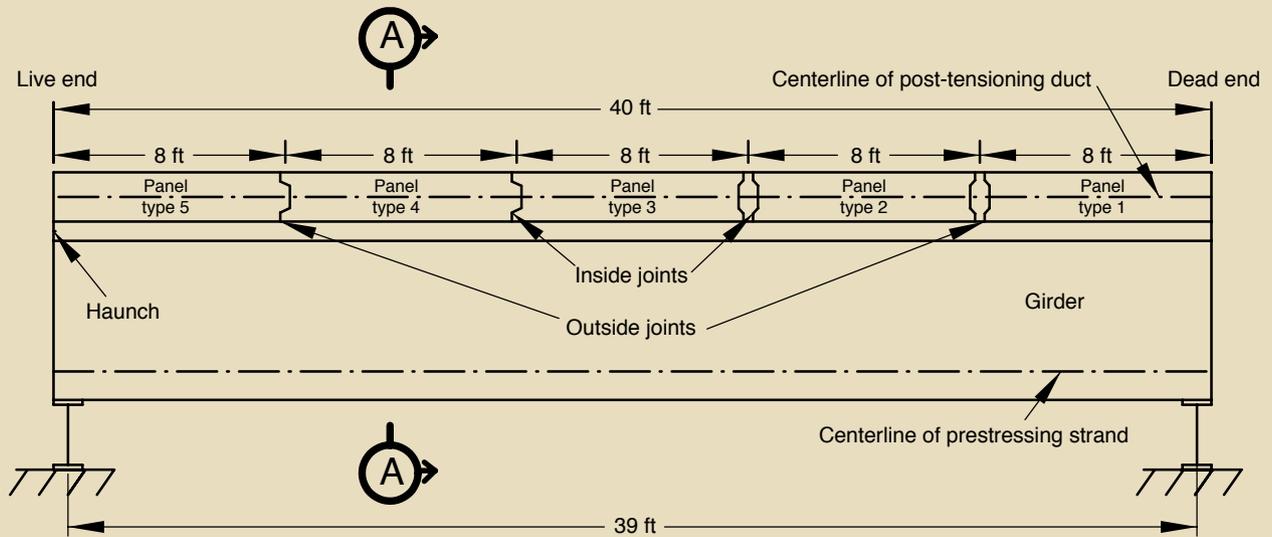
The lab mock-up consisted of two 40-ft-long (12 m) AASHTO type II girders spaced at 8 ft (2.4 m) center to center. The AASHTO type II girder is the most efficient girder for the 40 ft (12 m) simple span.

The deck was 8 in. (200 mm) thick, with 2 ft (0.6 m) overhangs. The haunch between the panels and girders was 2 in. (50 mm). **Figures 1** through **3** show the lab mock-up in elevation, plan, and section.

Twelve 1/2-in.-diameter (13 mm) strands were provided in the longitudinal post-tensioning ducts to provide a compressive stress across the transverse joints. The calculated initial level of deck panel compressive stress after all initial losses was 268 psi (1850 kPa). The calculated effective level of deck-panel compressive stress after all long-term losses was 200 psi (1380 kPa). This level was selected based on recommendations by Issa et al.<sup>7</sup>

Pocket spacing of 2 ft (0.6 m) was used for the first two panels at the live end of the bridge, where the strands were tensioned (Fig. 2).

Pocket spacing of 4 ft (1.2 m) was used for the first two panels at the dead end of the bridge, where the strands were anchored. Because the shear connectors were clustered in shear pockets instead of being dispersed more uniformly along the length of the bridge, the number of required connectors for each pocket was selected instead of a required connector spacing at a given location.



**Figure 1.** Elevation view of lab mock-up. Note: 1 ft = 0.305 m.

The middle panel along the girder line (panel 3 in Fig. 2) had 2.5 ft (0.75 m) pocket spacing. This panel served as a transition between the 2 ft (0.6 m) and 4 ft (1.2 m) ends for constructability.

The following design procedure was followed for each pocket:

1. The maximum vertical shear force at the location under consideration was calculated. The location was considered to be at the center of the pocket, and the shear associated with the worst-case live-load position was calculated.

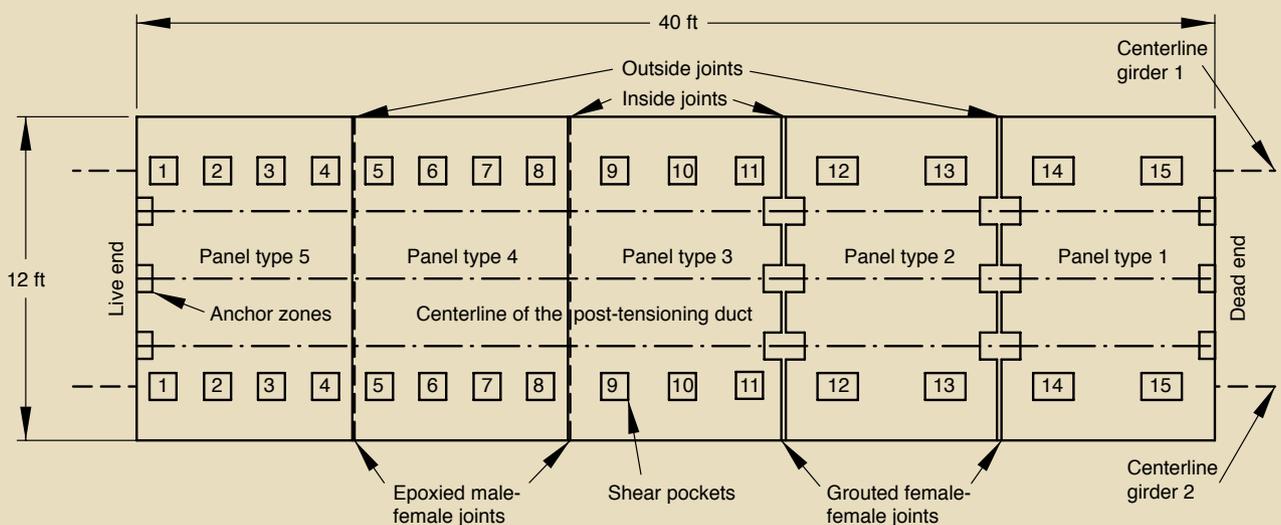
2. The horizontal shear force per inch was calculated using Eq. (1):

$$V_h = \frac{V_u}{d_v} \quad (1)$$

where

$V_h$  = horizontal shear force per inch

$V_u$  = factored vertical shear force



**Figure 2.** Plan view of lab mock-up. Note: 1 ft = 0.305 m.

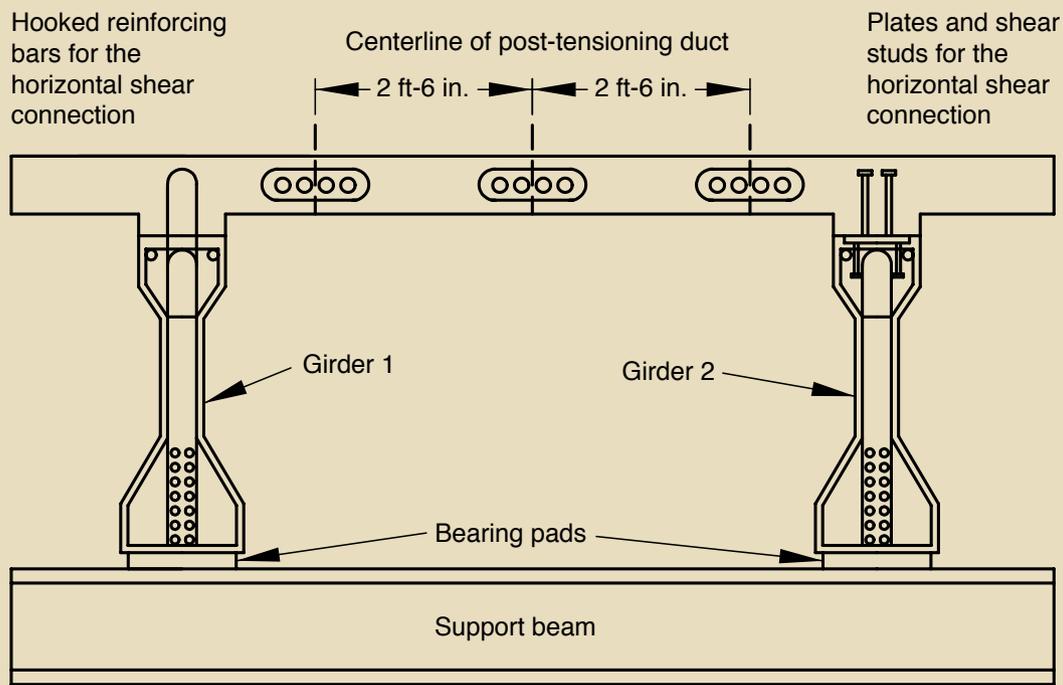


Figure 3. Section view of lab mock-up. Note: 1 ft = 0.305 m.

- $d_v$  = distance between the centroid of the steel in the tension side of the girder and the resultant center of the compressive force in the deck
- $b_v$  = width of the surface area engaged in shear transfer
- $c$  = cohesion factor
- $\mu$  = friction factor
- The tributary pocket spacing was calculated. The tributary pocket spacing was half the spacing on each side of the pocket under consideration.
  - The horizontal design shear force was calculated by multiplying the shear force per inch by the tributary pocket length.
  - AASHTO Eq. (5.8.4.31-3) was rearranged to solve for the required area of horizontal shear reinforcement. This resulted in Eq. (2), which was used to select the number of required shear connectors:

$$A_{s\_pocket} = \frac{\frac{V_h l_v}{\phi} - b_v l_v c - P_c}{f_y \mu} \quad (2)$$

where

$A_{s\_pocket}$  = area of steel in pocket

$l_v$  = tributary pocket spacing

$\phi$  = strength-reduction factor

= 0.9

= 75 psi (520 kPa) between two concrete surfaces cast at different times and not intentionally roughened (used for girder 1)

= 25 psi (170 kPa) between steel and concrete (used for girder 2)

= 0.6 $l$  between two concrete surfaces cast at different times and not intentionally roughened (used for girder 1)

= 0.7 $l$  between steel and concrete surfaces (used for girder 2)

$\lambda$  = 1.0 for normalweight concrete

$P_c$  = permanent net compressive force normal to the interface

$f_y$  = yield strength of the shear reinforcement

The vertical shear force  $V_u$  consisted of the shear due to the dead load of the girder, haunch, and deck as well as the shear due to live load. To provide a more uniform shear-

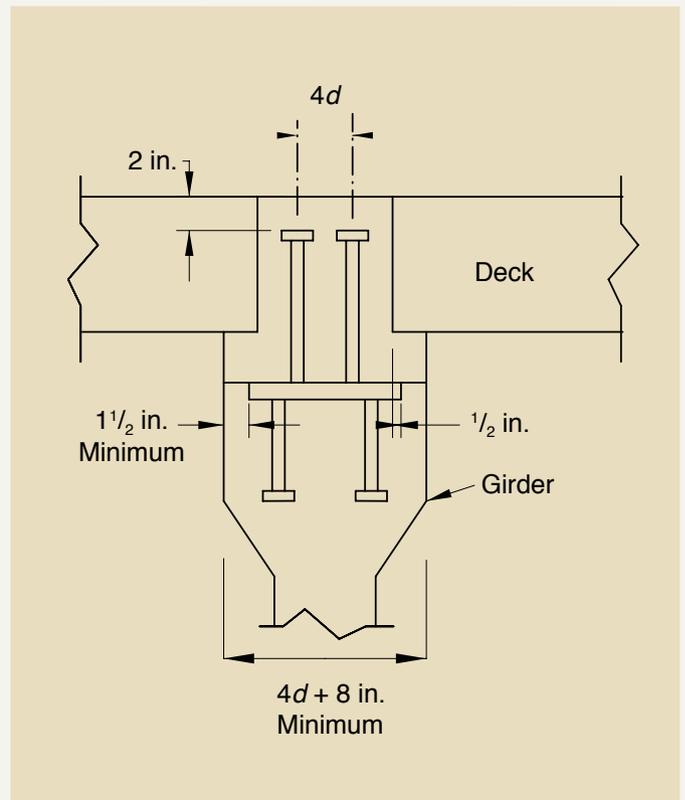


**Figure 4.** Placement of plates in girder 2.

connector design, the same number of shear connectors was provided in several pockets. This caused many of the pockets in regions with small shear forces to be oversized. However, the system was intentionally oversized for flexure and vertical shear so that the behavior of the different types of shear connectors and various pocket spacings could be studied.

The new shear-connector detail with the shear studs was fabricated by casting five steel plates in the top flange of the prestressed concrete girder. Five 1/4-in.-thick (6 mm) plates were placed in the top flange of girder 2 immediately after the concrete was placed in the formwork (**Fig. 4**). Five smaller plates were used rather than one large plate to make placing the plates easier.

The shear studs on the bottom of the plate were welded into place before casting the girders. Additional 7-in.-long (180 mm) shear studs were then welded directly to the top of the steel plate after the girder was erected and the deck panels were placed. **Figure 5** shows the dimensions required to satisfy AASHTO<sup>2,3</sup> cover and spacing requirements for the new connector detail. In Fig. 5,  $d$  is the stud diameter. The diameter of the shear studs used for this research program was 3/4 in. (20 mm), and the yield strength was 50 ksi (344 MPa).



**Figure 5.** Requirements for new shear-stud detail. Note:  $d$  = diameter of shear stud. 1 in. = 25.4 mm.

**Table 1.** Comparison of number of connectors required and number of connectors provided

Pocket number	Hooked reinforcing bars			Shear studs		
	Number of bars required	Number of bars provided	$\frac{V_{n\_prov}}{V_{n\_req}}$	Number of bars required	Number of bars provided	$\frac{V_{n\_prov}}{V_{n\_req}}$
1	8	8	1.03	7	6	0.93
2	8	8	1.11	6	6	1.00
3	7	8	1.20	6	6	1.09
4	6	8	1.31	5	6	1.19
5	5	8	1.44	5	6	1.31
6	5	8	1.61	4	6	1.46
7	4	8	1.81	4	6	1.65
8	4	8	1.90	4	6	1.70
9	3	8	2.14	3	6	1.89
10	2	8	2.76	3	6	2.44
11	3	8	1.87	4	6	1.60
12	6	10	1.42	7	8	1.29
13	10	10	1.06	9	8	0.96
14	12	16	1.25	11	12	1.14
15	15	16	1.06	13	12	0.97

Note:  $V_{n\_prov}$  = horizontal shear capacity provided;  $V_{n\_req}$  = horizontal shear capacity required.

Hooked reinforcing bars were used as shear connectors along girder 1. For this research program, separate bars were detailed and tied in with the stirrup and longitudinal bar reinforcing cage during fabrication of the girders. The length of the hooked reinforcing bars embedded in the girder was equal to the required development length for the bar size used. Grade 60 (410 MPa) no. 5 (16M) bars were used for the hooked reinforcing bar shear connectors.

**Table 1** shows the number of connectors required in each pocket using Eq. (1) and (2) and the number of connectors provided in each pocket for both girder 1 and girder 2. To simplify the design, it was desirable to reduce the number of combinations with different numbers of shear connectors in each pocket. This is typically done in practice. Table 1 shows the ratio of the horizontal shear capacity provided to the horizontal shear capacity required ( $V_{n\_prov}/V_{n\_req}$ ) based on Eq. (2).

## Instrumentation

During the casting operation, the panels and girders were instrumented with thermocouples and vibrating wire gauges (VWGs). The thermocouples and VWGs were located at the  $1/3$  points of the bridge span. **Figure 6** shows the loca-

tion of the VWGs and thermocouples through the depth of the cross section. The VWGs and thermocouples were used in another phase of this research program to examine creep and shrinkage behavior.

The panels and girders were instrumented with electrical-resistance (ER) strain gauges and displacement transducers (called wire pots in this paper). **Figure 6** shows the location of the ER strain gauges, which were located at the  $1/3$  points of the bridge span. ER strain gauges were also placed on selected shear connectors to measure the axial strain in the horizontal shear connectors during cyclic testing and static testing. The ER strain gauges were placed on the side of the shear studs perpendicular to the girder line within 2 in. (50 mm) of the top flange of the girder.

Wire pots were used to measure vertical displacements. The wire pots were connected to the bottom flange of the girder at each location. Selected vertical displacement measurement points coincided with the longitudinal position of the load points. **Figure 6** shows the location of the wire pots under the girders. During loading, the displacement measured by the wire pots under the applied loads included the displacement of the bearing pads used at the girder supports. To measure the displacement of the bear-

ing pads, wire pots were also placed 1 ft (0.3 m) toward the midspan of the bridge from the centerline of each bearing pad. These bearing pad displacements were used with the displacements measured by the wire pots located under the applied loads to determine the true displacement of the system.

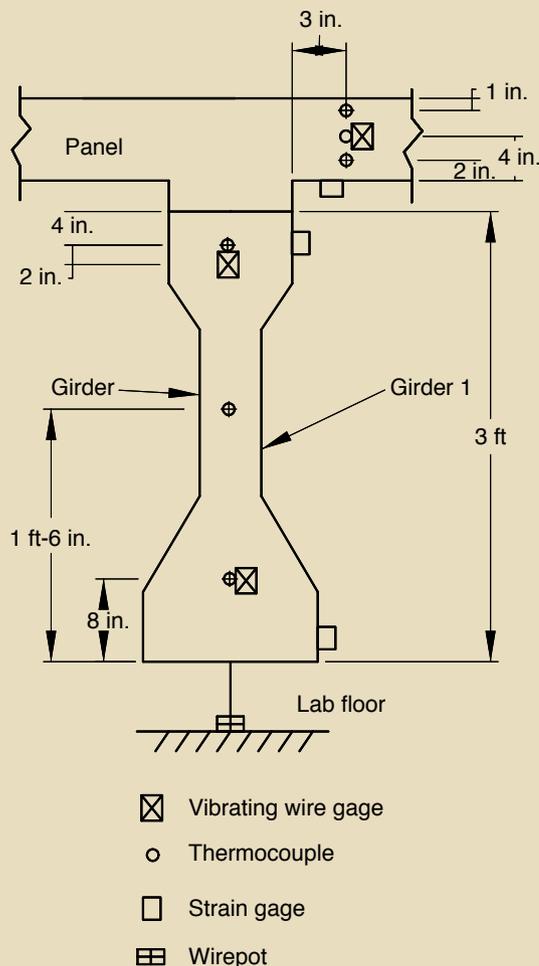
## Constructability study

Different stages of fabrication and construction of the lab mock-up were examined. The work hours, materials, and equipment required for each task were recorded. Any problems encountered and methods to resolve problems were also noted. Recommendations were made for the shear-connector type that worked best considering constructability.

**Live-load testing** The live-load testing program consisted of a series of static- and cyclic tests up to 2 million cycles at a frequency of 2 Hz. The loading for each of the two test setups (dead end and live end) consisted of four load points placed symmetrically about the longitudinal centerline of the bridge (Fig. 7). The two test setups were symmetric about the transverse centerline (midspan) of the bridge. Figure 8 shows a section view of the live-load test setup. The shear pockets are omitted for clarity. These equivalent loads created the same loading on each girder. This enabled a direct comparison of the performance of different shear connectors and their respective spacing. For the first 500,000 cycles, the load ranged from 2 kip/frame to 29.4 kip/frame (9 kN/frame to 131 kN/frame). This load range produced a maximum moment of 2250 kip-in. (254 kN-m) on each girder. This moment was equivalent to the maximum moment produced if the girders were loaded with the AASHTO LRFD fatigue load. During the cyclic tests, a compressive force of at least 1 kip (4.4 kN) was always present to prevent rotational movement of the spreader beam and to prevent damage to the bridge. For the next 1,500,000 cycles, the load ranged from 2 kip/frame to 44.7 kip/frame (9 kN/frame to 199 kN/frame). The load range of 42.7 kip/frame (190 kN/frame) corresponds to two AASHTO design wheel loads of 16 kip (71 kN) each multiplied by an impact factor of 1.33. This was greater than the AASHTO LRFD impact factor of 1.15 for fatigue.

Every 100,000 cycles to 300,000 cycles, the cyclic testing was stopped to conduct a static test on the system. The load was gradually increased to 44.7 kip/frame (199 kN/frame). The first static load test was performed before starting the cyclical loads. Intermediate static tests were done to determine whether there was any loss in stiffness in the lab mock-up due to loss of composite action, cracking, sliding at the joints, and so on, throughout the cyclic test program.

After the cyclic testing was completed on the dead- and live ends of the bridge, a static test was performed on the dead end and then on the live end. The purpose of



**Figure 6.** Location of instrumentation for panels and girders. Note: 1 in. = 24.5 mm; 1 ft = 0.305 m.

the test on the dead end was to determine whether the AASHTO LRFD required flexural strength of 15,500 kip-in. (1750 kN-m) and required vertical shear strength of 152 kip (676 kN) could be reached before a failure occurred at the horizontal interface between the girder and haunch. The load was gradually increased until failure.

An elastic analysis assuming full composite action between the deck panels and girders determined that an applied load of 187 kip/frame (832 kN/frame) would produce a moment equal to the required flexural strength of 15,500 kip-in. (1750 kN-m) and an applied load of 212 kip/frame (943 kN/frame) would produce a shear equal to the required vertical shear strength of 152 kip (676 kN).

## Results

### Constructability results

**Shear connectors** From a constructability standpoint, the steel plate with shear studs is easier to install than hooked reinforcing bars. It is quicker to place the steel

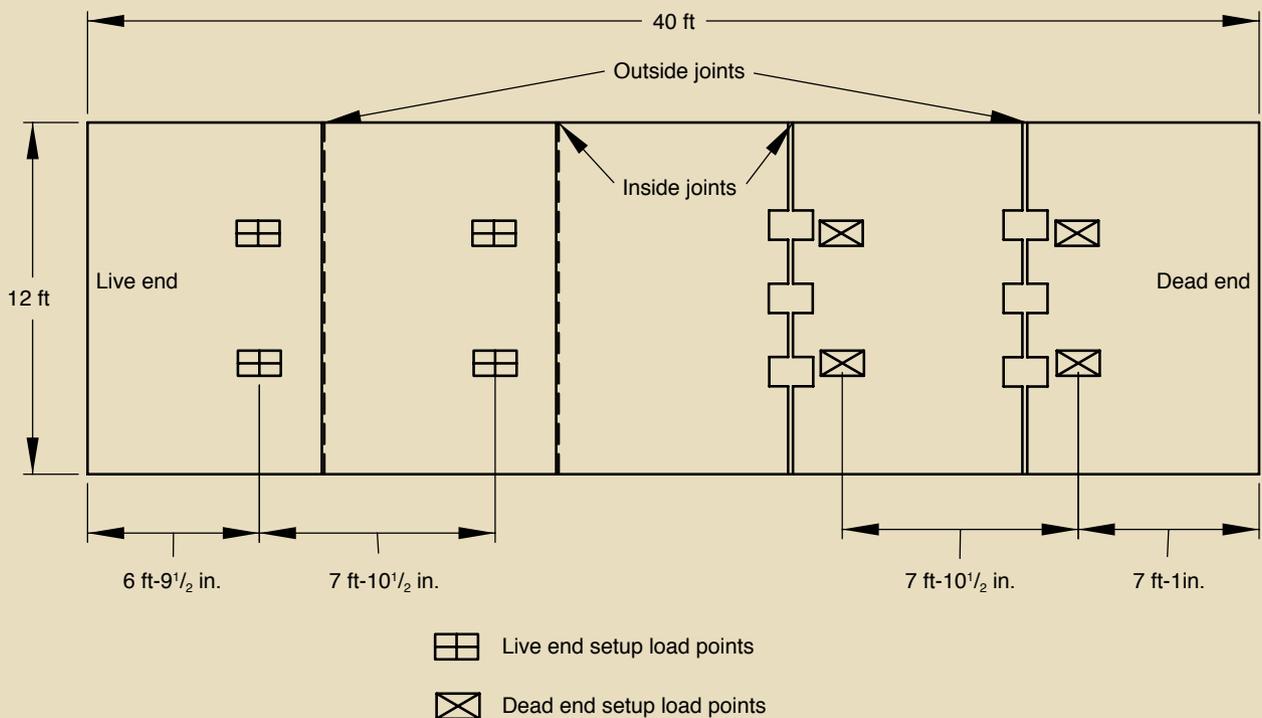


Figure 7. Plan view of live-load test setup. Note 1 in. = 25.4 mm; 1 ft = 0.305 m.

plate with the shear studs in the fresh concrete than to tie the hooked reinforcing bars with the reinforcing-steel cage. However, this may not be the case if the hooked reinforcing bars are detailed so that some of the stirrups along the length of the beam extend up into the shear pockets.

The steel plate detail allows the leveling bolts to bear directly on the girders because the majority of the top surface of the girder is steel. The frictional force that develops between the leveling bolt and the steel surface when the post-tensioning is applied is negligible. Girders with hooked reinforcing bars and a concrete top flange need to have a steel plate beneath the leveling bolts because large forces may be transferred otherwise.

The steel plate detail with the post-installed shear studs also allows the deck panels to be moved during erection without risk of conflicts with the shear-connector layout. Erecting panels with shear connectors already in place may damage the shear connectors if the panels accidentally collide with them. The steel plate detail is also better from a safety standpoint because the tripping hazard is eliminated.

**Pocket spacing and grouting process** After setting the panels, aligning all shear connectors, and forming the haunches, the haunches and shear pockets along each girder line were grouted. In this study, grout was placed into one pocket until the grout flowed through the haunch and was seen entering the adjacent pocket. This process was repeated until the entire girder line of pockets was full.

After the haunch and shear pockets were grouted and the formwork removed, gaps were present between the haunch and panel (Fig. 9). The gaps occurred near midspan for both girder 1 and girder 2. No other gaps were noticed.

Figure 2 shows that panel 3 is located at midspan. The interior pocket spacing for panel 3 was 2.5 ft (0.75 m). This process may have allowed air to become trapped between the grout placed in one shear pocket and the grout placed in the adjacent shear pocket. The presence of the gaps after grouting shows that it is desirable from a constructability standpoint to have a strict quality-control procedure for grouting.

To reduce the possibility of the formation of gaps in the haunch, it is recommended that each pocket be filled with grout until the next pocket starts to fill up with grout, which indicates that the haunch between is full. If grout is added to subsequent pockets too soon, air voids are more likely to form between the pockets.

Gaps did not occur between the haunch and pocket at the dead end of the bridge with 4 ft (1.2 m) pocket spacing. It is believed that 4 ft pocket spacing does not pose a problem when trying to fill the haunch with grout as long as a quality-control procedure is followed. The problems seen with trapped air were due to constructability problems and not the design spacing of the pockets. The gaps did not prove detrimental to the results.

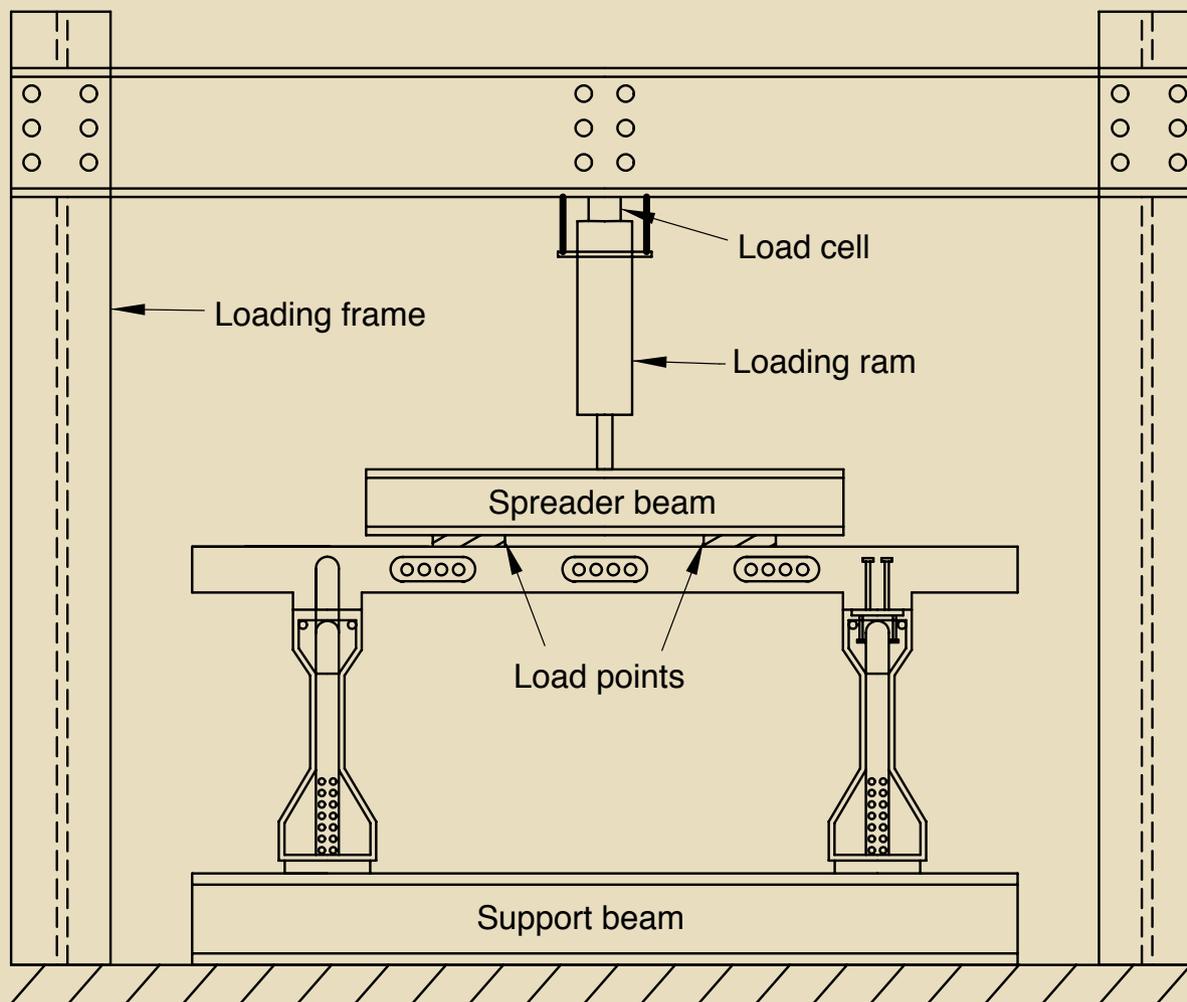


Figure 8. Section view of live-load test setup.

## Live-load testing results

Based on results from the intermittent static load tests, it appears that the cyclic loading had minimal effects, if any, on the degree of composite action in the lab mock-up. There was no cracking at the transverse joints, and no relative vertical movement between adjacent panels was measured by the wire pots. No cracking was observed in the girder or deck. There was a negligible difference (less than 0.01 in. [0.25 mm]) in the vertical deflections measured during the intermediate static tests. The axial strains in the shear connectors were less than 1% of the nominal yield strain. Based on these strains, the shear connectors were not engaged in resisting the horizontal shear stresses developed during cyclic loading for 2 ft (0.6 m) pocket spacing or 4 ft (1.2 m) pocket spacing. This indicates negligible loss in the bond between the grout and concrete.

During the dead-end tests, the first crack was a web shear crack at approximately 95 kip/frame (420 kN/frame). The first flexural crack occurred under the inside load point at

118 kip/frame (525 kN/frame). As load increased, additional flexure and shear cracks developed and propagated. The first cracks in the haunches were noted at 209 kip/frame (930 kN/frame) for girder 1 and 183 kip/frame (814 kN/frame) for girder 2.

Figure 10 shows the deflections of the dead end (4 ft [1.2 m] pocket spacing) of the lab mock-up during the static test at the outside loading point and the inside loading point. Figure 7 shows the inside and outside loading points. The tangential, vertical stiffness values were compared for the different static tests. The tangential, vertical stiffness  $K_{vert}$  at a load point was calculated as

$$K_{vert} = \Delta P / \Delta \delta$$

where

$\Delta P$  = applied load increment at the load point under consideration



**Figure 9.** Void in grout between haunch and panel. This void was near the midspan of the specimen, where shear was low, and was not detrimental to the performance of the shear connection.

$\Delta\delta$  = vertical deflection increment at the load point under consideration

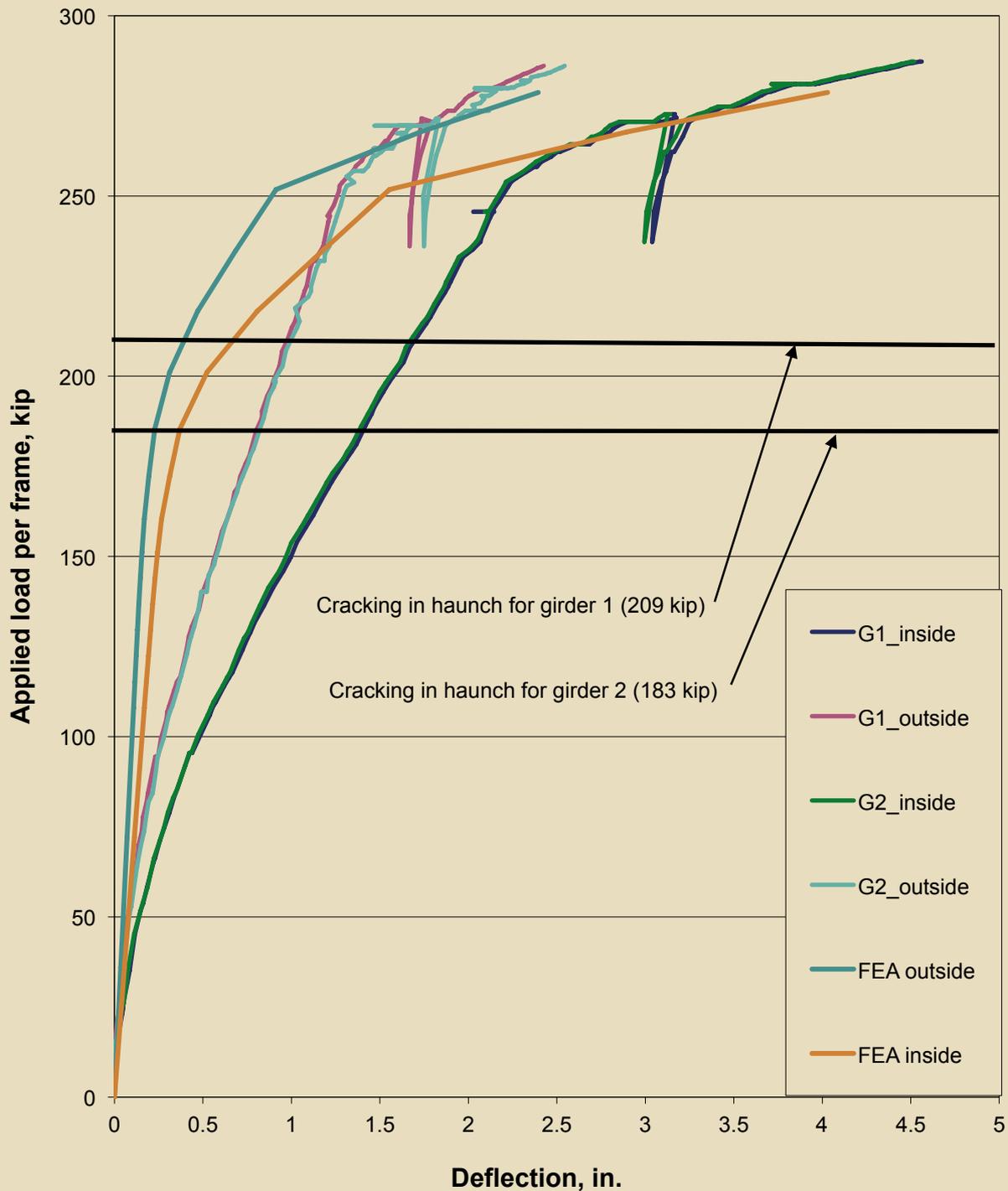
For applied loads greater than 270 kip/frame (1200 kN/frame), the vertical stiffnesses at the inside load point and outside load point were 10.0 kip/in. and 18.3 kip/in. (1750 kN/m and 3200 kN/m), respectively. This corresponds to 1.3% and 1.4% of the initial vertical stiffness at the inside load point and outside load point, respectively, measured during the first static load test.

At an applied load of 256 kip/frame (1140 kN/frame), there was a significant decrease in stiffness. The decrease in stiffness is most likely due to the strain in the prestressing strands in the girders exceeding the nominal yield strain. The load at which the prestressing strands exceed their nominal yield strain was calculated to be 269 kip/frame (1200 kN/frame). This was confirmed with the finite-element models, which indicated first yield at 268 kip/frame (1200 kN/frame). The calculated loads at first yield were within 5% of measured.

When 272 kip/frame (1210 kN/frame) load was reached,

the load was intentionally reduced to 237 kip/frame (1050 kN/frame). The load was then increased to 287 kip/frame (1280 kN/frame). Figure 10 also presents the results of the finite-element analysis performed using the software DIANA.<sup>6</sup> The presented analysis assumed full composite action between the girder and the deck. The measured displacements were somewhat higher at lower loads, but the ultimate strength and stiffness near ultimate were similar. This indicates that the system, at loads approaching ultimate, acted essentially as a fully composite system.

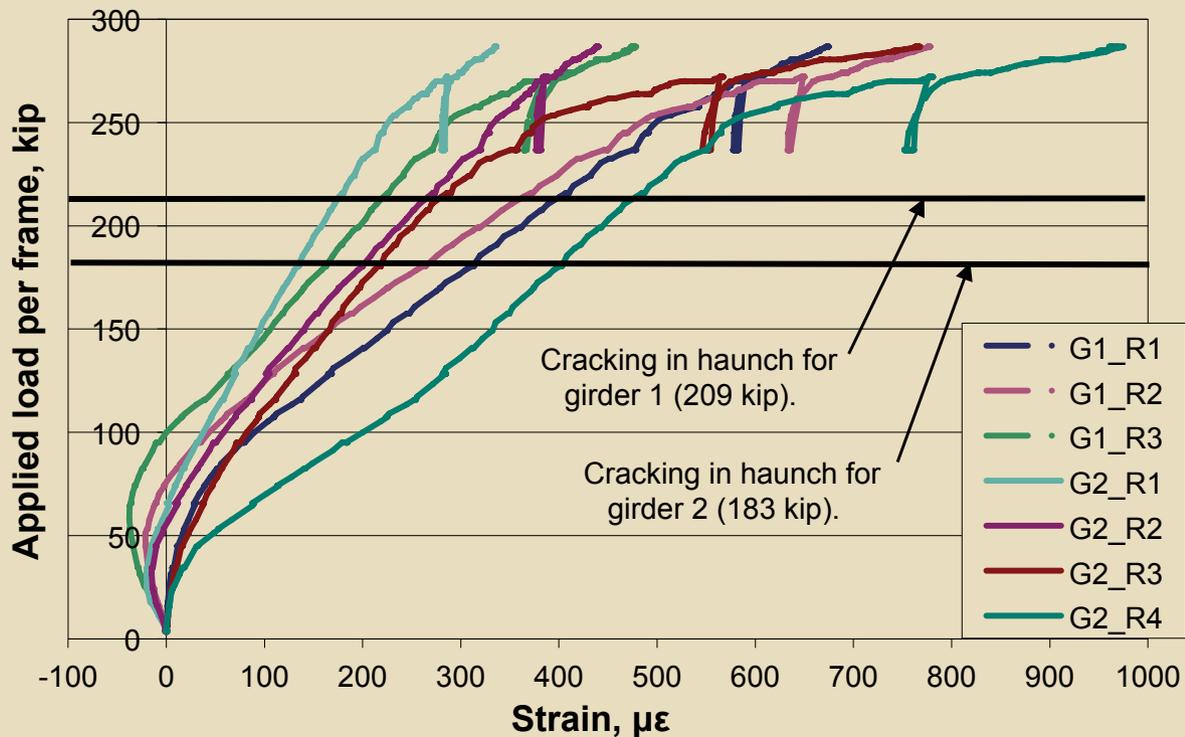
**Figure 11** shows the strains in the shear connectors for the static test at the dead end. The first part of the gauge identifier indicates the girder. For example, gauge G2\_R3 is located over girder 2. As stated previously, shear studs were used for girder 2 and hooked reinforcing bars were used for girder 1. The second part of the name of the gauge refers to gauge number. At an applied load of approximately 256 kip/frame (1140 kN/frame), the rate at which the strains increased with respect to the load increased. This is interpreted to indicate that the shear connectors were engaged in resisting the horizontal shear stresses as the cracking at the interface between the haunch and girder



**Figure 10.** Deflections during the static test at dead end. Note: Figure 8 shows locations of load points. Girder designated with outside = vertical deflection at outside loading point measured by a wire pot under that girder; girder designated with inside = vertical deflection at inside loading point measured by a wire pot under girder. 1 in. = 25.4 mm; 1 kip = 4.448 kN.

continued to propagate. The strains in the shear connectors in Fig. 11 were less than 50% of the predicted nominal yield strain for the entire range of applied loads. The strain behavior in the majority of the instrumented shear connectors was similar, with the exception of shear connector G2\_R4. Problems were encountered with this gauge on

preliminary live-load tests. This may be a contributing factor to the difference in the strain for this connector compared with other connectors. However, the strain in shear connector G2\_R4 was still small and shows the increase in the strain rate at an applied load of approximately 256 kip/frame (1140 kN/frame) like the rest of the shear connectors.



**Figure 11.** Strains measured in horizontal shear connectors during static test at the dead end. Note: G1 = girder 1 with hooked reinforcing steel; G2 = girder 2 with shear studs; R# = specific gauge number. 1 kip = 4.448 kN.

During the live end tests, the first crack was a web-shear crack at approximately 113 kip/frame (503 kN/frame). The first flexural crack occurred under the inside load point at 122 kip/frame (543 kN/frame). As load increased, additional flexure and shear cracks developed and propagated. The first cracks in the haunches were noted at 260 kip/frame (1106 kN/frame) for girder 1 and 196 kip/frame (872 kN/frame) for girder 2.

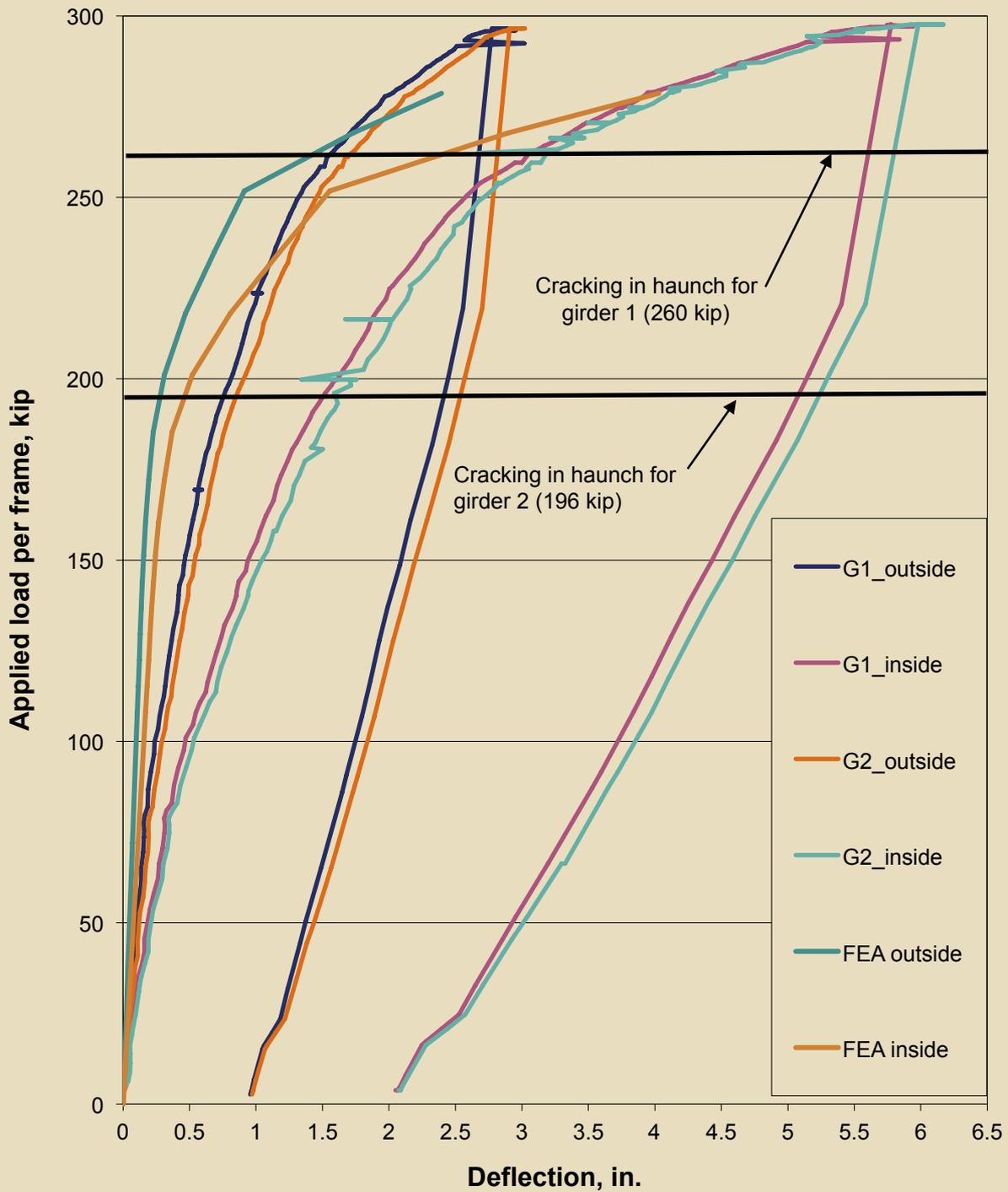
**Figure 12** shows the deflections of the live end (2 ft [0.6 m] pocket spacing) of the lab mock-up during the final static test at the outside loading point and the inside loading point. The vertical stiffness of the lab mock-up at the loading points was less at the onset of the tests during the first static test than during the intermediate static test in the cyclic testing program. The applied load versus deflection plot did not have a distinct change in slope after cracking like the dead end as seen in Fig. 10. A more gradual transition in the applied load versus deflection plot for the live end may have been due to the cracks that existed from the previous test on the dead end.

Visible cracking occurred at the interface between the haunch and girder at applied loads of 260 kip/frame and 196 kip/frame (1160 kN/frame and 872 kN/frame) for girder 1 and girder 2, respectively. The rate of increase of the strains in the shear connectors with respect to the applied load was greater than that before cracking occurred in

the haunch. **Figure 13** shows this for a selected number of shear connectors on the live end of the bridge. The increase in strain rate with respect to the load is interpreted to indicate that the shear connectors were engaged in resisting the horizontal shear stresses after cracking at the interface between the haunch and girder. However, the strains in the shear connectors are less than 50% of the nominal yield strain for the entire range of applied loads.

Both the live end and dead end of the lab mock-up failed in flexure by crushing of the concrete on the top surface of the bridge deck adjacent to the inside loading points. The maximum moment reached during the static tests on the dead and live ends of the lab mock-up were 23,700 kip-in. and 24,500 kip-in. (2680 kN-m and 2770 kN-m), respectively. The difference in the maximum moments for the two tests is 3%. The pocket spacing had little influence on the flexural strength of the lab mock-up. In addition, this provided confirmation that any initial cracking from testing the dead load before the live load did not affect the ultimate behavior of the specimen.

The AASHTO LRFD-required flexural strength, including self weight and the AASHTO HS-20 design vehicle, of the lab mock-up was 15,500 kip-in. (1750 kN-m). The required strength is defined here as the factored design moment divided by the strength-reduction factor. Both the live end and dead end measured strength exceeded the



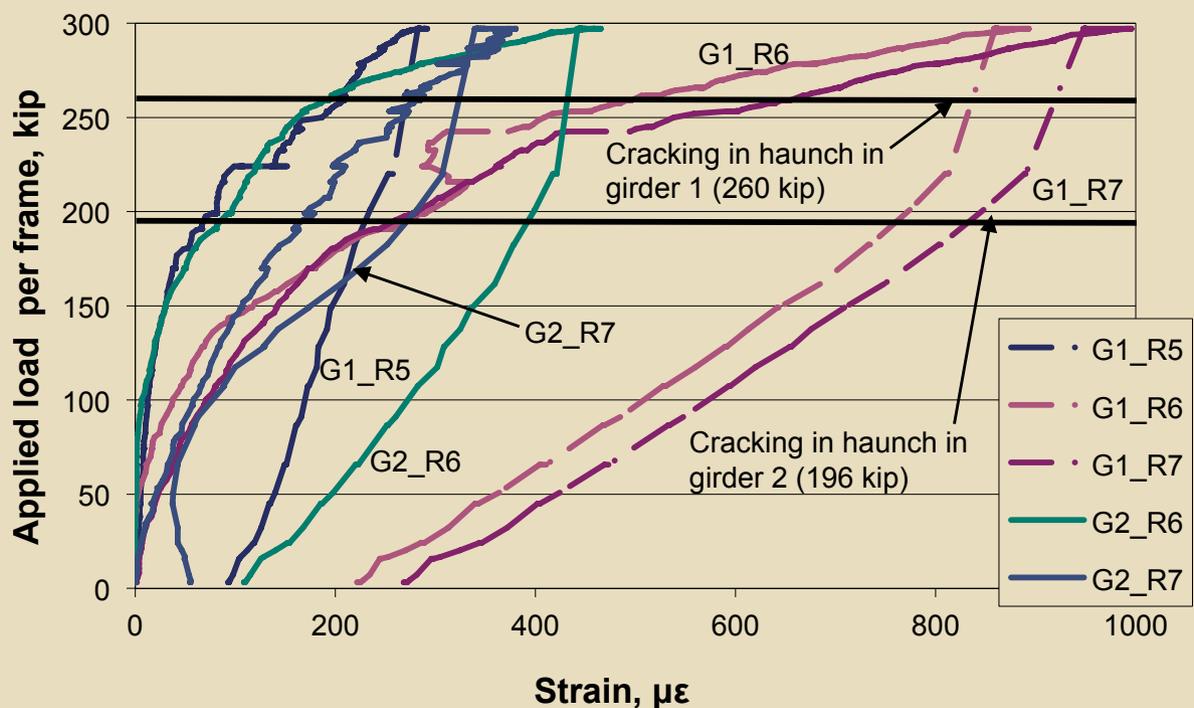
**Figure 12.** Deflections during static test at live end. Note: Figure 8 shows locations of load points. 1 in. = 25.4 mm; 1 kip = 4.448 kN.

AASHTO required strength.

The maximum shear reached during the static tests was 206 kip on the dead end and 213 kip (917 kN and 948 kN) on the live end of the lab mock-up. The required vertical shear strength of the lab mock-up is 152 kip (676 kN). The lab mock-up with either 2-ft (0.6 m) or 4-ft (1.2 m) pocket spacing was capable of exceeding the required verti-

cal shear strength. The flexural design and vertical shear design were modified such that the lab mock-up would not fail before the horizontal shear forces exceeded the horizontal shear design capacities in the regions with high shear forces.

**Table 2** shows the ratio of the resulting maximum applied horizontal shear force at each shear pocket from the static



**Figure 13.** Strains measured in horizontal shear connectors during static test at live end. Note: G1 = girder 1 with hooked reinforcing steel; G2 = girder 2 with shear studs; R# = specific gauge number. 1 kip = 4.448 kN.

**Table 2.** Ratio of applied horizontal shear to horizontal shear capacity for final static tests

Pocket number	$V_{applied}/V_{prov}$ dead end	$V_{applied}/V_{prov}$ live end
1	0.47	1.24
2	0.47	1.24
3	0.47	1.24
4	0.47	1.24
5	0.47	0.37
6	0.47	0.37
7	0.47	0.37
8	0.52	0.41
9	0.56	0.58
10	0.56	0.58
11	0.64	0.67
12	0.49	0.67
13	0.52	0.70
14	1.19	0.49
15	1.19	0.49

Note:  $V_{applied}$  = maximum applied horizontal shear force at each shear pocket;  $V_{prov}$  = nominal horizontal shear capacity at each shear pocket from the AASHTO LRFD Bridge Design Specifications shear friction equation.

tests to the nominal horizontal shear capacity at each shear pocket from the AASHTO LRFD shear friction equation. Pocket 1 was closest to the live end of the bridge with the 2 ft (0.6 m) pocket spacing, and pocket 15 was closest to the dead end of the bridge with the 4 ft (1.2 m) pocket spacing. The regions from the support to the outside loading point had the highest shear based on the shear diagram for the loading conditions. The region with the highest shear incorporates all of the pockets within the exterior panel. The horizontal shear force developed at each pocket during the static tests is 19% higher and 24% higher than the nominal horizontal shear capacity at the dead end and live end, respectively. This indicates that for this design condition the number of shear connectors can be reduced and the lab mock-up can still reach the required flexural strength, the required vertical shear strength, and the required horizontal shear strength.

Both the 2 ft (0.6 m) pocket spacing and 4 ft (1.2 m) pocket spacing are capable of providing the required strength. In addition, the strains of less than 50% of the nominal yield strain at maximum loads in all of the connectors indicate that both the hooked reinforcing bars and the shear studs perform well as shear connectors.

During the final static test at the dead end, cracking occurred at the interface between the haunch and girder 1 at an applied load of 209 kip/frame (930 kN/frame) and at the interface between the haunch and girder 2 at an applied load of 183 kip/frame (814 kN/frame). These applied loads

correspond to 111% of the required flexural strength and 99% of the required vertical shear strength for girder 1 and 97% of the required flexural strength and 86% of the required vertical shear strength for girder 2. During the final static test at the live end, cracking occurred at the interface between the haunch and girder 1 at an applied load of 260 kip/frame (1160 kN/frame), and cracking occurred at the interface between the haunch and girder 2 at an applied load of 196 kip/frame (872 kN/frame). These applied loads correspond to 138% of the required flexural strength and 123% of the required vertical shear strength for girder 1 and 104% of the required flexural strength and 92% of the required vertical shear strength for girder 2.

## Conclusion

The exceptional performance of the lab mock-up under flexural loads, vertical shear loads, and horizontal shear loads compared with the AASHTO LRFD design along with the low strains in the shear connectors show that using either hooked reinforcing bars or shear studs as shear connectors in both 2 ft (0.6 m) pocket spacing and 4 ft (1.2 m) pocket spacing is acceptable. The hooked reinforcing bars and the new shear-stud detail had axial strains less than 50% of the nominal yield strain for the fatigue and strength tests. With the combination of shear studs and 4 ft (1.2 m) pocket spacing, cracking occurred at the interface of the haunch and girder at 86% of the required vertical shear strength. However, cracking was expected to occur at the interface to engage the shear connectors in resisting the horizontal shear forces, and this detail provided adequate strength.

## Recommendations

- For the new shear-stud detail, shear studs must be welded to the bottom surface of the plate before its embedment in the top of the girder. The spacing  $s$  of these shear studs can be determined from the following equation:

$$s = \frac{\mu(2A_{stud}f_y + P_c)}{\frac{V_u}{d_v\phi} - cb_v} \quad (3)$$

where

$A_{stud}$  = cross-sectional area of one shear connector

The spacing of the shear studs is determined assuming two shear studs will be placed in each row.

- Under this research program, 2 ft (0.6 m) pocket spacing and 4 ft (1.2 m) pocket spacing performed well. The results were based on a girder depth of 36 in. (0.9 m). In general, the pocket spacing  $s_{pocket}$  should be

determined using the following equation:

$$s_{pocket} \leq d_v \cot(\theta) \quad (4)$$

where

$\theta$  = angle at which the shear cracks form (according to modified compression field theory)

The calculation for obtaining  $\theta$  is outlined in AASHTO LRFD.<sup>2,3</sup> The angle  $\theta$  can be conservatively taken as 45 deg. The pocket spacing determined from Eq. (4) should not be greater than 4 ft (1.2 m).

- The new shear-stud detail allows for easier and quicker placement of the deck panels during deck construction and eliminates the tripping hazard. Although the new shear-stud detail was shown to be the superior shear-connector type of the two types tested, hooked reinforcing bars provide a suitable alternative.

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$\lambda$  = 1.0 for normalweight concrete  
 $\mu$  = friction factor  
 $\phi$  = strength-reduction factor

## Notation

$A_{s\_pocket}$  = area of steel in pocket

$A_{stud}$  = cross-sectional area of one shear connector

$b_v$  = width of the surface area engaged in shear transfer

$c$  = cohesion factor

$d$  = stud diameter

$d_v$  = distance between the centroid of the steel in the tension side of the girder and the resultant center of the compressive force in the deck

$f_y$  = yield strength of the shear reinforcement

$K_{vert}$  = tangential, vertical stiffness =  $\Delta P / \Delta \delta$

$l_v$  = tributary pocket spacing

$P_c$  = permanent net compressive force normal to the interface

$s$  = spacing

$s_{pocket}$  = pocket spacing

$V_{applied}$  = maximum applied horizontal shear force at each shear pocket

$V_h$  = horizontal shear force per inch

$V_{n\_prov}$  = horizontal shear capacity provided

$V_{n\_req}$  = horizontal shear capacity required

$V_{prov}$  = nominal horizontal shear capacity at each shear pocket from the *AASHTO LRFD Bridge Design Specifications* shear friction equation

$V_u$  = factored vertical shear force

$\Delta P$  = applied load increment at the load point under consideration

$\Delta \delta$  = vertical deflection increment at the load point under consideration

$\theta$  = angle at which the shear cracks form (according to modified compression field theory)



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## Synopsis

Experimental and analytical studies were conducted to investigate the composite behavior of precast concrete bridge deck-panel systems. A full-scale, single-span

bridge was constructed. Fatigue tests and ultimate load tests were conducted with different shear-connector types and various pocket spacing. Different construction details were compared for strength, fatigue performance, and ease of construction. Results of the research program were used to recommend the best construction details to use and propose design practices that account for the variables that make precast concrete bridge deck-panel systems unique.

## Keywords

Composite construction, deck panel, shear detail, shear pocket.

## Review policy

This paper was reviewed in accordance with the Precast/Prestressed Concrete Institute's peer-review process.

## Reader comments

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