

Precast concrete deck-to-girder mechanical connection for accelerated bridge construction

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- This paper presents an experimental investigation and design example of a new mechanical shear pocket connection for precast concrete deck-to-girder connections. Conventional connection methods are also reviewed for comparison.
- The new proposed mechanical connection is anticipated to be used in accelerated bridge construction.
- Experimental results show 25% higher interface shear resistance results for the new mechanical connection than for the conventional connection.

Full-depth precast concrete bridge deck systems have been increasingly used in new construction and as a replacement for deteriorating cast-in-place concrete decks because of their high quality, durability, and ease/speed of construction.¹ Full-depth precast concrete deck systems were originally used in the 1960s as noncomposite with the supporting girders, and their first use in composite construction was in 1973.² Composite full-depth precast concrete deck systems provide economical design because they satisfy strength and serviceability requirements using smaller and shallower girder sections than are used in noncomposite systems. Precast concrete deck panels are usually made composite with the supporting girders via shear connectors, such as shear studs, bent bars, or threaded rods, projecting from the girder top flange and embedded in either discrete shear pockets or continuous longitudinal troughs (channels) in the deck panels. A summary of different deck-to-girder connection details can be found in Tawadrous.³

This paper focuses on the deck-to-girder connection that uses discrete shear pockets and clustered shear connectors in concrete bridge girders. **Figure 1** shows two examples of these connections implemented in the construction of the Kearney East Bypass bridge and Belden-Laurel bridge in Nebraska. These connections were made using one or more threaded rods as shear connectors embedded in the precast, prestressed concrete girders at a maximum spacing of 4 ft (1.2 m). Rectangular or circular hollow structural sections (HSS) were used to form shear pockets in precast concrete deck panels at the same spacing over each girder

PCI Journal (ISSN 0887-9672) V. 65, No. 3, May–June 2020.

PCI Journal is published bimonthly by the Precast/Prestressed Concrete Institute, 8770 W. Bryn Mawr Ave., Suite 1150, Chicago, IL 60631.

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Kearney East Bypass Bridge



Belden-Laurel Bridge

Figure 1. Full-depth precast concrete deck panels with rectangular hollow structural section (HSS)-formed shear pockets used in Kearney East Bypass Bridge and circular HSS-formed shear pockets used in Belden-Laurel Bridge, both in Nebraska.

line. Grout or flowable concrete was then placed through grouting vents to fill the shear pockets and haunch area around the shear connectors to achieve composite section as shown in **Fig. 2**. The advantages of using HSS-formed shear-pocket connections in full-depth precast concrete deck systems are twofold. First, HSS is used as a stay-in-place form for the shear pocket, which eliminates the fabrication and stripping of the wood or foam blockouts. Second, HSS provides confinement to the concrete inside the shear pocket, which prevents the splitting or breakout of concrete due to

high stress concentration around the clustered connectors. This confinement also minimizes reinforcement congestion around the shear pockets and allows for increased spacing between pockets.

Several studies were conducted to investigate the structural performance and constructibility of HSS-formed shear-pocket connections.⁴⁻⁷ Although these studies have indicated excellent structural performance, they have shown that the performance is highly dependent on the quality and strength of the

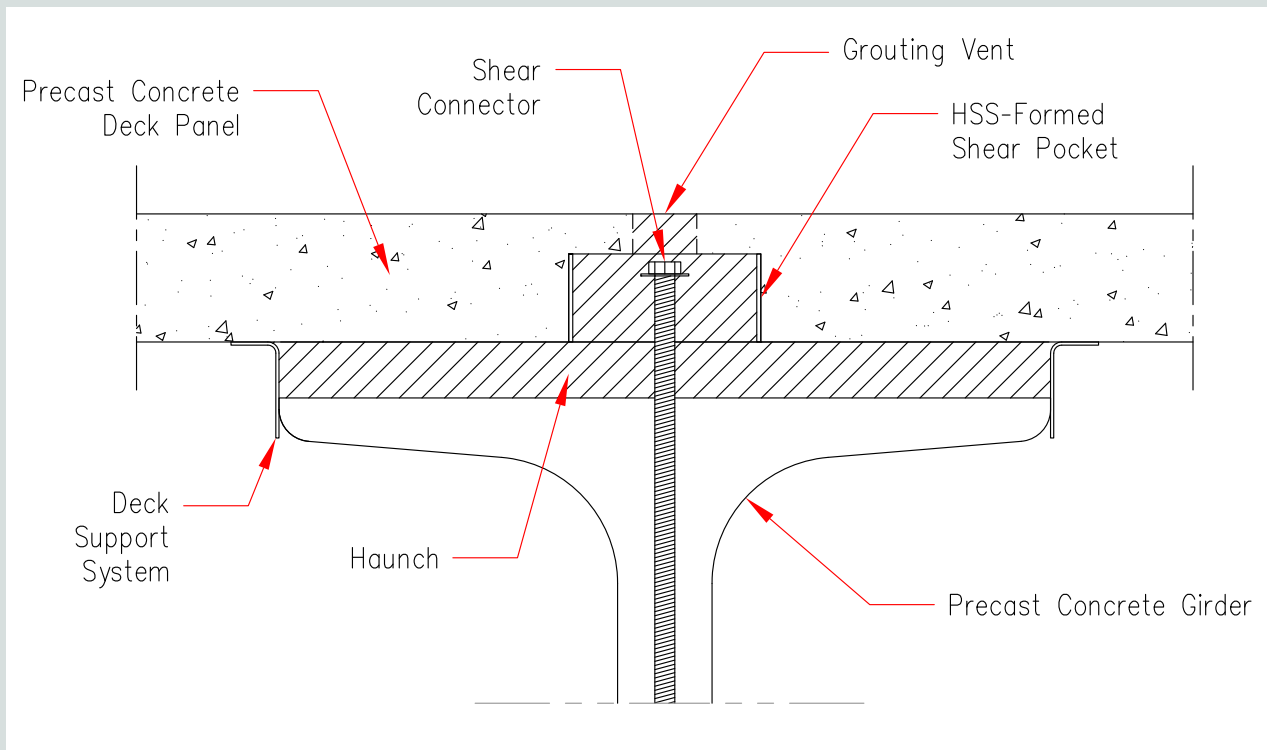


Figure 2. Typical hollow structural section (HSS)-formed shear-pocket connection detail.

grouting material and the embedment of shear connectors inside the shear pockets. Low grout strength, improper grout consolidation, or lack of connector embedment could result in premature concrete breakout. Alternatively, using small shear-pocket dimensions to avoid concrete breakout could result in constructibility problems, such as conflicts between pockets and connectors during erection due to inadequate production tolerances.

This paper presents a new mechanical connection (US Patent 10,508,434)⁸ that was developed by the authors to address the aforementioned limitations of conventional shear-pocket connections. The new connection provides a unique method to mechanically tie the precast concrete deck panel to the supporting concrete girder while accommodating production and erection tolerances. This method enhances the structural performance of the connection and eliminates its dependence on the strength or quality of the grouting material. The paper also presents the results of full-scale push-off tests conducted using the new connection as well as a conventional HSS-formed shear-pocket connection to compare their structural performance and constructibility.

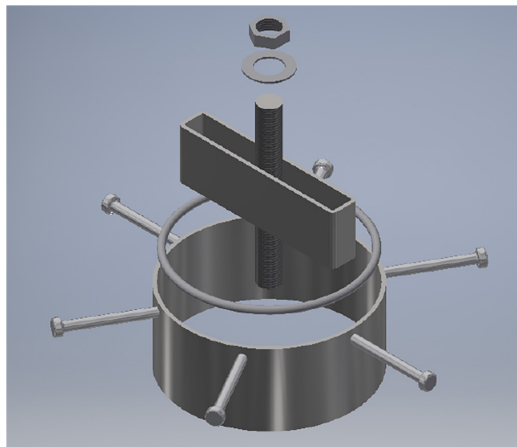
New connection conceptual design

To mechanically connect a precast concrete deck panel to a precast, prestressed concrete girder, the standard HSS-formed shear pockets and threaded-rod shear connectors are used. Circular HSS sections are often used in the case of a single

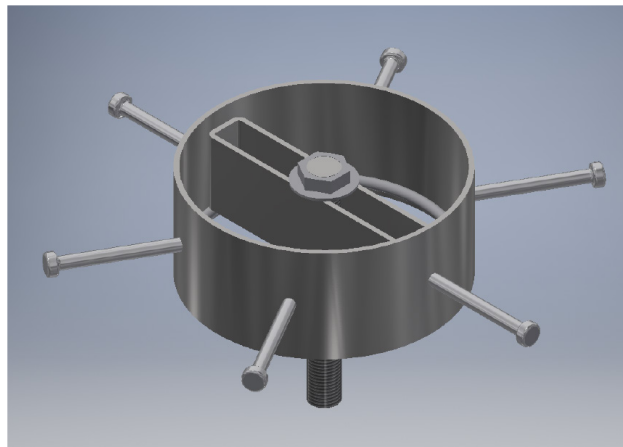
connector (one threaded rod per connection), and rectangular HSS sections are often used in the case of multiple connectors (two or more threaded rods per connection) (Fig. 3). In addition, a hoop steel bar or two straight steel bars are welded to the inside surface of the circular or rectangular HSS, respectively, to form a ledge at the bottom of the HSS. Then, another rectangular HSS section that fits inside the shear pocket is used around each shear connector to be supported by the welded bar or bars. A standard structural washer and nut are used on top of the inner HSS to clamp the precast concrete panel to the supporting girder. Nuts are made snug tight, and then flowable grout or concrete is placed from the top to fill the shear pocket and haunch around the shear connectors. In this design, the threaded rod is fully developed in the shear pocket by the mechanical anchoring system of the inner HSS and welded bar(s), which is independent of the grout strength and embedment depth. Shear studs are welded to the exterior surface of the HSS to transfer the clamping force to the deck panel. Figure 3 shows three-dimensional (3-D) assembled and exploded views of the mechanical connection for both circular and rectangular shear pockets. This connection works regardless of the location of the shear connector inside the shear pocket, which provides adequate tolerance for panel and girder production and erection.

Experimental investigation

Four specimens were tested to evaluate the performance of the new connection: two specimens with the mechanical-

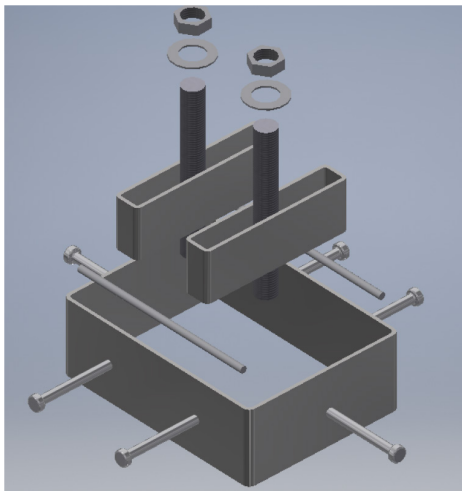


Exploded view

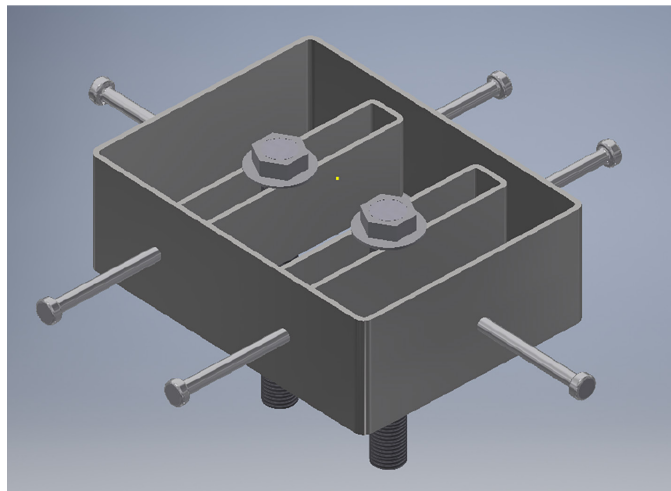


Assembled view

Circular HSS for single connector



Exploded view



Assembled view

Rectangular HSS for multiple connectors

Figure 3. Three-dimensional sketch of the exploded view and assembled view of circular hollow structural section for single connector and exploded view and assembled view of rectangular HSS for multiple connectors. Note: HSS = hollow structural section.

ly connected shear pocket (MCSP), labeled MCSP#1 and MCSP#2, and two specimens with the conventional shear pocket (CSP), labeled CSP#1 and CSP#2. The specimens use full-scale connections in 4 ft × 4 ft × 7.5 in. (1.2 m × 1.2 m × 109.5 mm) deck slabs on 6 ft × 2 ft 6 in. × 3 ft 4 in. (1.8 m × 0.76 m × 1.02 m) concrete blocks simulating bridge girders. A single 1.5 in. (38 mm) diameter and 3 ft (0.9 m) long threaded rod with structural washer and heavy hex half nut was used as a shear connector in all four specimens. The threaded rod has a cross section area of 1.41 in.² (909.68 mm²) and complies with ASTM A193 Grade B7⁹ with yield strength of 105 ksi (724 MPa) and ultimate strength of 125 ksi (862 MPa). A circular HSS 12.75 × 0.25 in. (323.85 × 6.35 mm) (that is, 12.75 in. outside diameter and 0.25 in. wall thickness) was used to form the shear pocket in all four specimens and

was 5.5 in. (139.7 mm) long. The circular HSS complies with ASTM A500 Grade B¹⁰ with yield strength of 42 ksi (290 MPa) and ultimate strength of 58 ksi (400 MPa). A total of five 0.75 in. (19 mm) diameter and 6 in. (152.4 mm) long shear studs were welded to the exterior surface of the circular HSS, as shown in **Fig. 4**, to anchor it to the concrete slab.

The two specimens with mechanical connections had a no. 4 (13M) hoop bar welded to the inside surface of the circular HSS at ½ in. (12.7 mm) from the bottom edge to support the inner rectangular HSS 12 × 2 × 0.25 in. (304.8 × 50.8 × 6.35 mm) (that is, 12 in. deep, 2 in. wide, and 0.25 in. thick) that was 3¾ in. (95.25 mm) long and of the same grade as the circular HSS. Figure 4 shows the different components of the new shear-pocket connection.

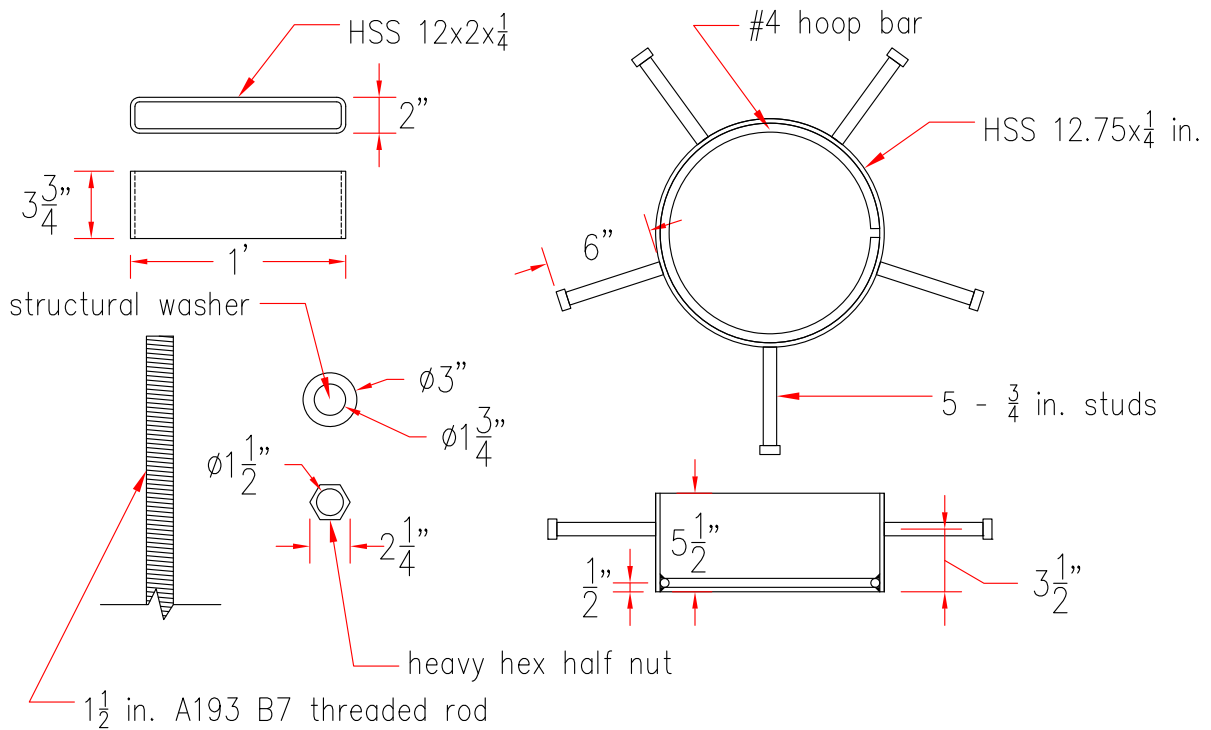


Figure 4. Components of the new shear-pocket connection. Note: HSS = hollow structural section. #4 = no. 4 = 13M; 1" = 1 in. = 25.4 mm. Ø = diameter.

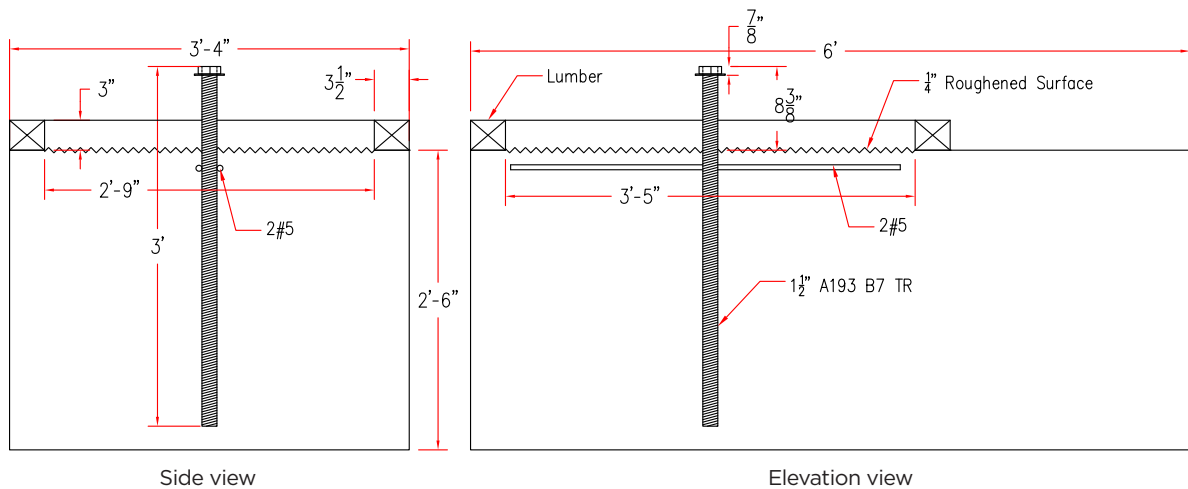


Figure 5. Side view (left) and elevation view of the concrete block used in all push-off specimens. Note: TR = threaded rod. #5 = no. 5 = 16M; 1" = 1 in. = 25.4 mm; 1' = 1 ft = 0.305 m.

Figure 5 shows the dimensions and reinforcement details of the concrete block used in all four tests to simulate the precast, prestressed concrete girder. The length of the block was 6 ft (1.8 m), and the width was 3 ft 4 in. (1.02 m) to match the top flange width of American Association of State Highway

and Transportation Officials (AASHTO) or PCI bulb-tee girders. Threaded-rod shear connectors were embedded in the blocks and projecting approximately 8 3/8 in. (212.725 mm) above the girder top flange. Two no. 5 (16M) bars were used in the top flange around the shear connector to support the

threaded rods during concrete placement and control block cracking around the rod during loading. Normalweight 6 ksi (41.4 MPa) self-consolidating concrete (SCC) was used to cast the four blocks. The top surface of the blocks was intentionally roughened to an amplitude of 0.25 in. (6.35 mm) using a metal rake following the common practice in producing precast, prestressed concrete girders. Two 2 × 4 in. (50.8 × 101.6 mm) lumber pieces were used all around the roughened area to form the haunch and support the concrete slab.

Figure 6 shows the dimensions and reinforcement details of the concrete slab used to simulate the precast concrete deck panels used for MCSP specimens. The slab is 4 ft × 4 ft × 7.5 in. (1.2 m × 1.2 m × 109.5 mm) and was made using the same concrete used for casting the concrete blocks. Typical top and bottom deck reinforcement was used in all slabs, and the HSS-formed shear pocket was placed at the center of each slab. The height of the shear pocket was 5.5 in. (139.7 mm), and the remaining 2 in. (50.8 mm) was formed using foam. Similar slabs were fabricated for CSP specimens; however, only a 4 in. (101.6 mm) diameter grouting vent was provided at the top of the HSS-formed shear pocket. Also, no hoop bars were welded to the inside of the HSS in case of conventional connection.

Figure 7 shows the steps of erecting the push-off specimen with mechanical connection. The 3 in. (76.2 mm) thick

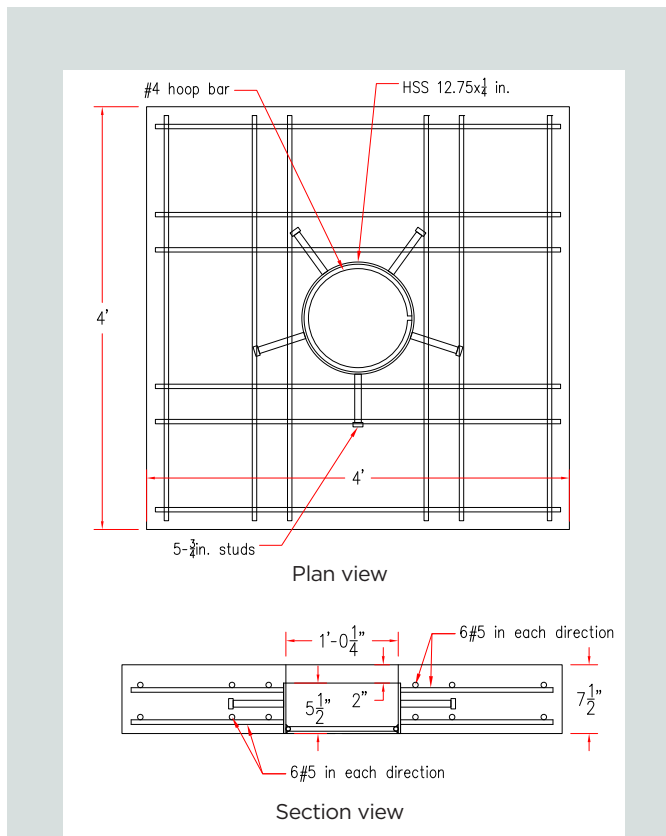


Figure 6. Plan view and section view of concrete slab used in new connection specimens. Note: HSS = hollow structural section. #4 = no. 4 = 13M; #5 = no. 5 = 16M; 1" = 1 in. = 25.4 mm; 1' = 1 ft = 0.305 m.

haunch formed on top of the concrete block using lumber was reinforced using four no. 4 (13M) bars in one direction and two no. 4 bars in the other direction. Then, the precast concrete slab was placed on top of the haunch forms so that the shear connector was inside the shear pocket. Only these two steps were the same for the specimens with CSP connections. The inner rectangular HSS was placed inside the shear pocket around the shear connector and supported on the welded hoop bar. Then the structural washer and heavy hex half nut were installed so that the nut was snug tight without applying excessive force to the concrete slab. Finally, a 6 ksi (41.4 MPa) SCC with 24 in. (609.6 mm) slump flow was placed through the open pocket to fill the haunch and shear pocket and ensure proper consolidation. It should be noted that the SCC was placed through a 4 in. (101.6 mm) diameter grouting vent in the case of CSP specimens. **Figures 8** and **9** show the views and dimensions of the assembled specimen with mechanical shear-pocket connection and CSP connection, respectively.

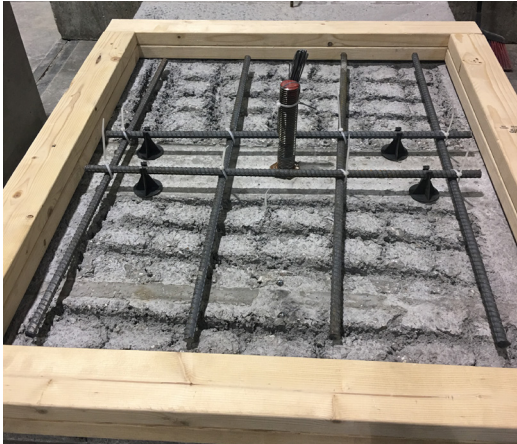
The push-off test setup and instrumentation are shown in **Fig. 10** and consist of the following main items:

- supporting frame made up of two horizontal threaded rods anchored to the reaction wall from one side and to a horizontal steel beam at the other side
- tie-down steel frame anchored to the floor to prevent rotation of the specimen
- hydraulic jack, load cell, and loading plates
- linear variable displacement transducer (LVDT) to measure relative movements between the concrete slab and concrete girder

A horizontal load was applied to the middle of the concrete slab using a 400 kip (1779.2 kN) hydraulic jack and spreader steel beam. The load was gradually increased at a rate of approximately 4 kip/sec (17.8 kN/sec) until failure occurred. The horizontal and vertical displacements of the concrete slab relative to the concrete block were measured using two horizontal LVDTs and one vertical LVDT. The load-displacement measurements of the four tested specimens were recorded, averaged, and plotted as shown in **Fig. 11** and **12**.

Figure 11 shows the load versus horizontal displacement plots, and Fig. 12 shows the load versus vertical displacement plots for all four specimens. In these figures, the two specimens with mechanically connected shear pockets are labeled MCSP#1 and MCSP#2, while the two specimens with CSP connections are labeled CSP#1 and CSP#2.

Based on the load-displacement plots shown in Fig. 11 and 12, three distinctive interface shear behaviors are presented. First is the linear portion of the plots with high slope and small relative displacement (less than 0.01 in. [0.254 mm]), which represents the cohesion component of the interface shear resistance of concrete. Second is the following linear



Forming and reinforcing the haunch



Placing the precast concrete slab on haunch forms



Placing the inner HSS around the shear connector



Installing the washer and nut (snug tight)



Placing SCC in the shear pocket and haunch

Figure 7. Erection sequence of the new shear-pocket connection. Note: HSS = hollow structural section; SCC = self-consolidating concrete.

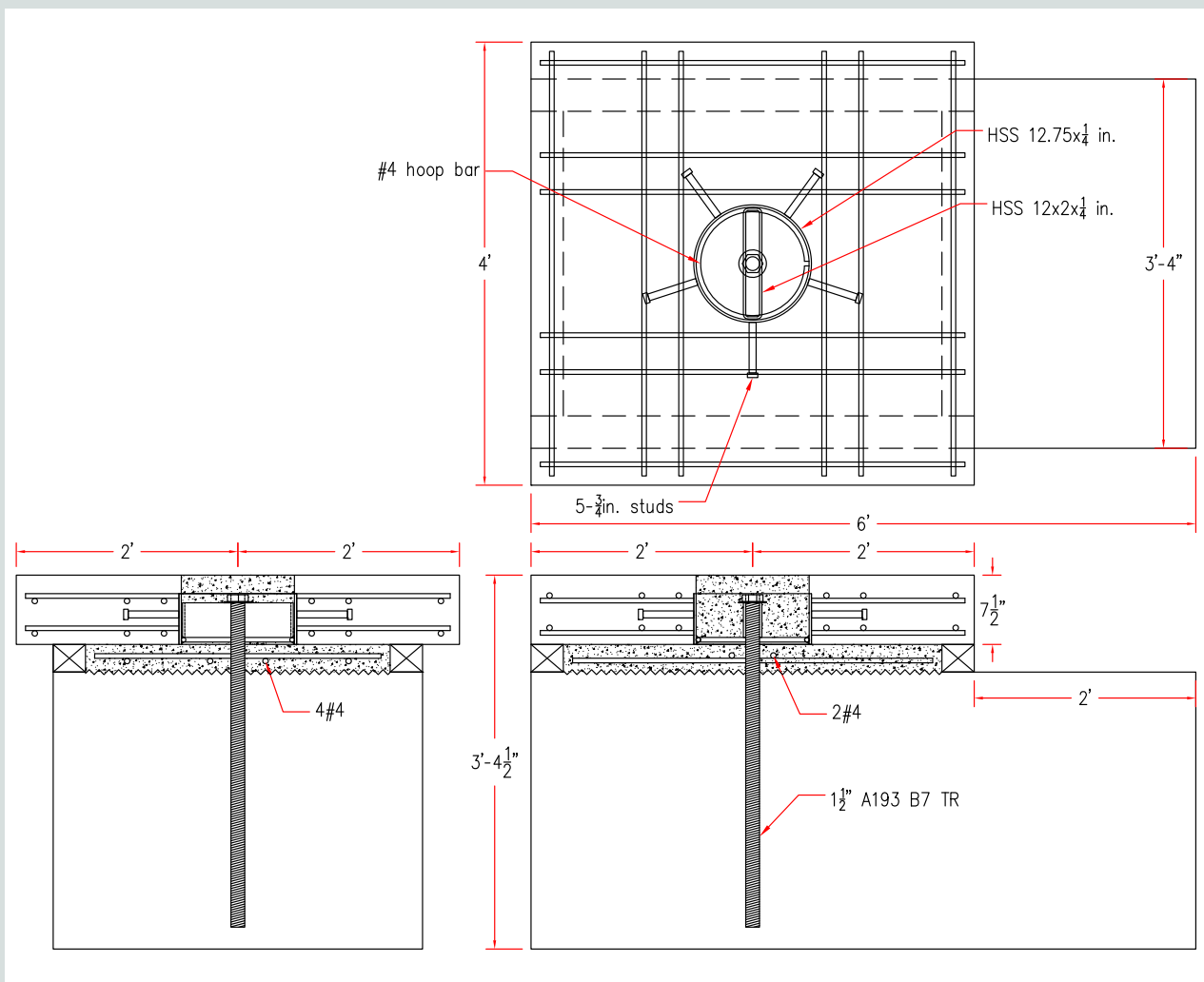


Figure 8. Views of the assembled push-off specimen with mechanical shear-pocket connection. Note: HSS = hollow structural section; TR = threaded rod; #4 = no. 4 = 13M; 1" = 1 in. = 25.4 mm; 1' = 1 ft = 0.305 m.

portion of the plots with milder slope and more significant relative displacement (up to 0.1 in. [2.54 mm]), which represents the friction component of the interface shear resistance due to the clamping force of the shear connector and aggregate interlock. Third is the remaining nonlinear portion of the plot with significant relative displacement (up to 1.7 in. [43.2 mm]), which represents the shear connector resistance to bending, known as the dowel effect.⁵ For the specimens with MCSP connections, this third portion was accompanied by a considerable increase in the load because the shear connectors were mechanically anchored (that is, fully developed) and their dowel effect contributed to the interface shear resistance until they were completely sheared off after experiencing significant deformations. Alternatively, the specimens with CSP connections experienced a decrease in their load-carrying capacity during the third portion because the shear connectors were not fully developed, resulting in a brittle concrete breakout failure at smaller displacements. CSP connections did not utilize the dowel effect of the high-strength threaded rods,

and their failure was dependent on the strength of the grout or concrete filling the shear pocket.

The two MCSP connections achieved an average ultimate capacity of 192.5 kip (856.24 kN) with corresponding average horizontal and vertical displacements of 1.67 and 0.42 in. (42.42 and 10.67 mm), respectively. The two CSP connections achieved an average ultimate capacity of 154.3 kip (686.3 kN) with corresponding average horizontal and vertical displacements of 0.1 and 0.09 in. (2.54 and 2.286 mm), respectively. This indicates that the MCSP connection achieved an average of 25% higher interface shear resistance and demonstrated higher ductility than did the CSP connection, which is attributed to engaging the dowel effect of the shear connectors that are mechanically anchored and reducing the dependence on the grout or concrete strength.

Figures 13 and 14 show the failure modes of the MCSP and CSP connections, respectively. The MCSP connections failed

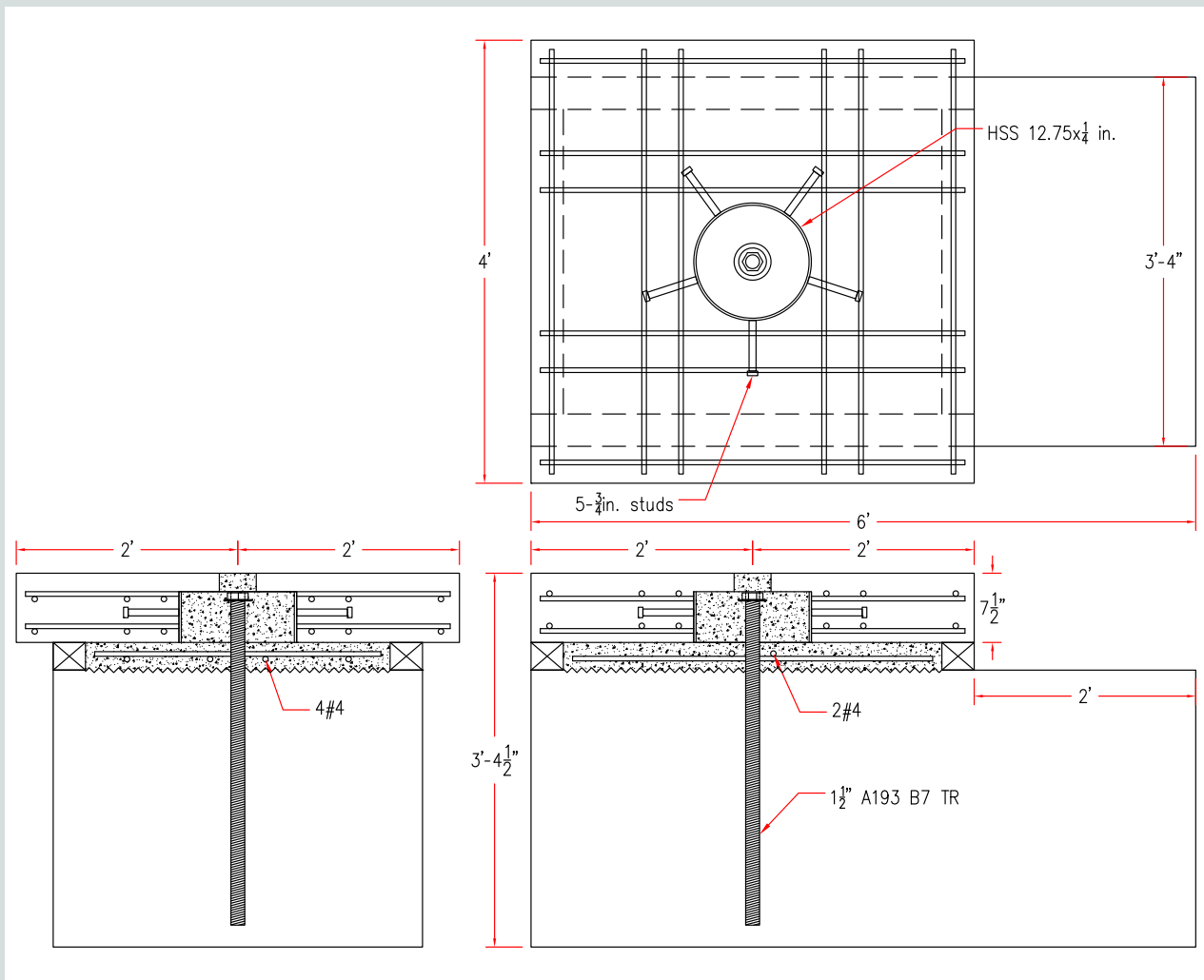


Figure 9. Views of the assembled push-off specimen with conventional shear-pocket connection. Note: HSS = hollow structural section; TR = threaded rod; #4 = no. 4 = 13M; 1" = 1 in. = 25.4 mm; 1' = 1 ft = 0.305 m.

by shearing off the threaded-rod shear connectors without any damage to the concrete inside the shear pocket or haunch. The CSP connections failed in a brittle mode by concrete breakout forming a cone around the shear connector. It should be noted that the SCC used for grouting the haunch and shear-pocket areas in all specimens had an average compressive strength of 6.5 ksi (44.8 MPa) at the time of testing.

Design example

A design example was performed to predict the expected shear resistance of the two connection types. The new shear-pocket connection was designed for the bridge design example 9.1a, presented in chapter 9 of the *PCI Bridge Design Manual*.¹¹ The superstructure of this bridge consists of six BT-72 beams spaced at 9 ft (2.7 m) on center (**Fig. 15**). The deck was designed to act fully composite with the precast concrete beams to resist all superimposed dead loads, live loads, and impact. Design live load was HL-93. The bridge

was 120 ft (36.6 m) long, single span, with a total width of 51 ft (15.4 m).

Analysis of this bridge showed that the factored horizontal shear force V_h at the critical section was 2.86 kip/in. (0.323 kN/m) due to all loads acting on the composite section. The shear pocket connection was designed at 48 in. (1219 mm) spacing along the girder lines. Therefore, the minimum nominal interface shear resistance V_{ni} was estimated as follows using a resistance reduction factor ϕ of 0.9:

$$V_{ni} = \frac{V_h}{\phi} = \frac{2.86 \times 48}{0.9} = 152.5 \text{ kip (678.3 kN)}$$

To design the shear connector, a minimum specified 28-day concrete compressive strength of 6 ksi (41.4 MPa) was specified for precast, prestressed concrete deck panels and shear pocket and haunch. The shear pocket is formed using ASTM A500¹⁰ Grade B HSS 12.75 × 0.25 in. (324 × 6.35 mm), which resulted in an interface shear area of

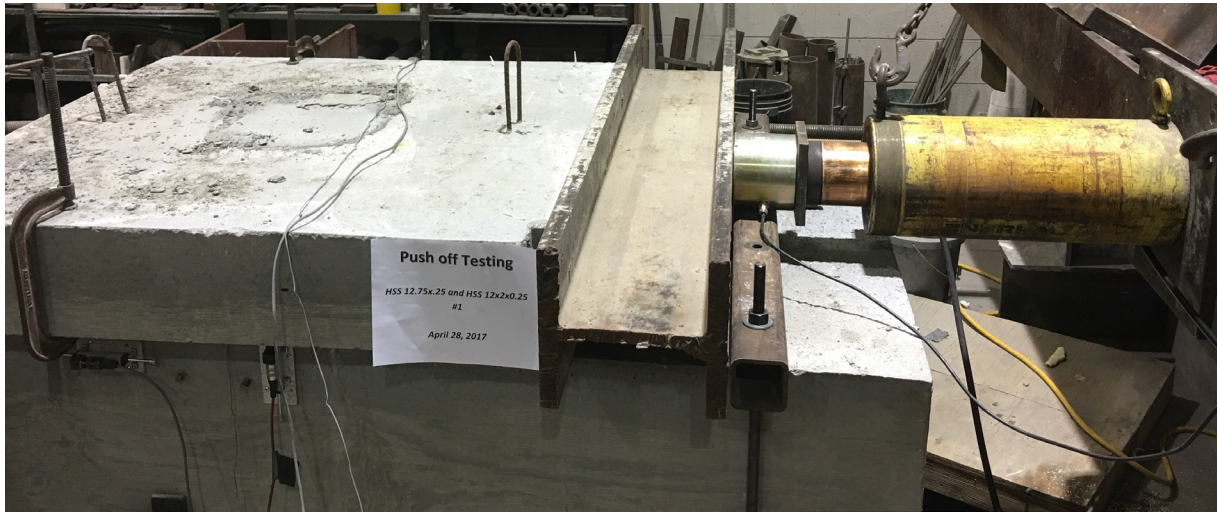


Figure 10. Push-off test setup. Note: LVDT = linear variable displacement transducer.

117.86 in.² (76,039 mm²). A single 1.5 in. (38.1 mm) diameter shear connector of ASTM A1939 Grade B7 threaded rod was used, which has a cross section area of 1.41 in.² (910 mm²). These are the same properties that the connection tested in the experimental investigation section of this study had.

According to *AASHTO LRFD Bridge Design Specifications* article 5.8.4,¹² interface shear resistance was calculated at the soffit of the deck panel for monolithic concrete as follows:

$$\begin{aligned} V_{ni} &= cA_{cv} + \mu(A_{vf}f_y + P_c) \\ &= 0.4 \times 117.86 + 1.4(1.41 \times 60 + 0.0) \\ &= 165.6 \text{ kip (736.6 kN)} \end{aligned}$$

where

- c = cohesion factor
- A_{cv} = area of concrete considered to be engaged in interface shear transfer
- μ = friction factor
- A_{vf} = area of interface shear connectors crossing the shear plane within the area A_{cv}
- f_y = yield stress of interface shear connectors but not more than 60 ksi
- P_c = permanent net compressive force normal to the shear plane

The nominal shear resistance V_{ni} used in the design shall not be greater than the lesser of either of the following:

$$\begin{aligned} V_{ni} &\leq K_1 f'_c A_{cv} = 0.25 \times 6.5 \times 117.86 = 191.5 \text{ kip (851.8 kN)} \\ V_{ni} &\leq K_2 A_{cv} = 1.5 \times 117.86 = 176.8 \text{ kip (786.4 kN)} \end{aligned}$$

where

- K_1 = fraction of concrete strength available to resist interface shear
- K_2 = factor limiting interface shear resistance
- f'_c = specified concrete compressive strength

Thus, V_{ni} is 165.6 kip (736.6 kN), which is the predicted capacity of the CSP connection and is greater than the required nominal strength of 152.5 kip (678.3 kN).

According to Hatami,⁵ the dowel effect of the shear connector contributes to the interface shear resistance of the connection as follows:

$$V_{dowel} = \frac{k n f_d d_b^3}{3.5 l_a} \leq \frac{A_{vf} f_d}{\sqrt{3}}$$

where

- V_{dowel} = dowel effect contribution to interface shear resistance
- k = factor of 2.0 for headed shear connectors and 1.0 for connectors without head
- n = number of shear connectors

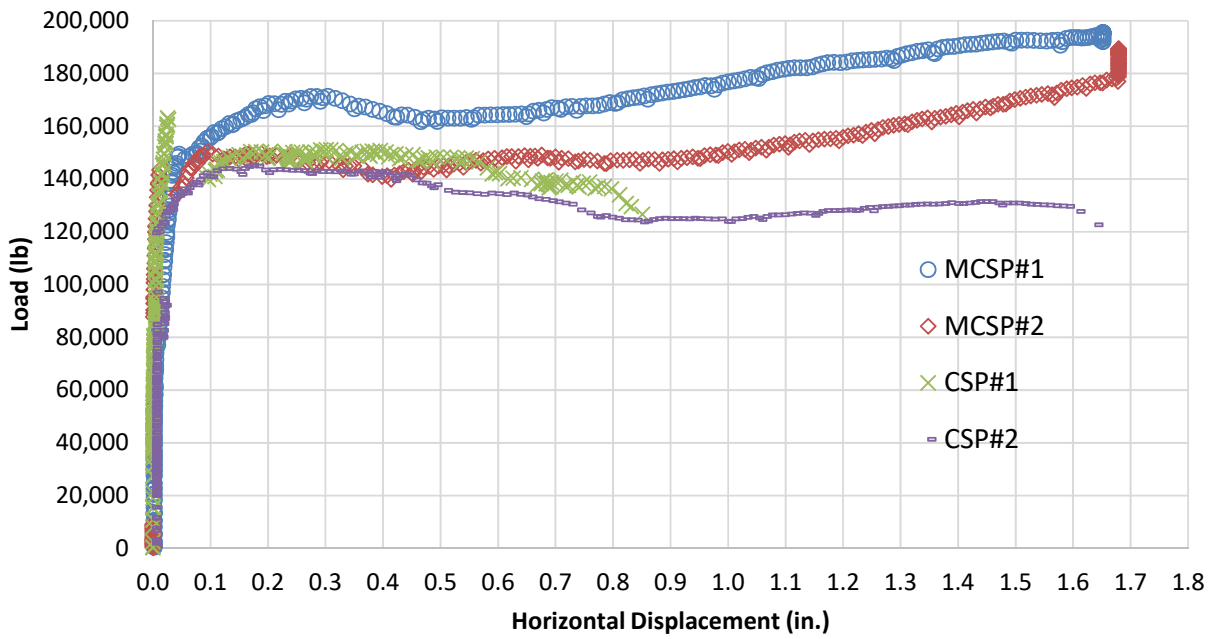


Figure 11. Load-horizontal displacement relationships of all specimens. Note: CSP = conventional shear pocket; MCSP = mechanically connected shear pocket. 1 in. = 25.4 mm; 1 lb = 4.448 N.

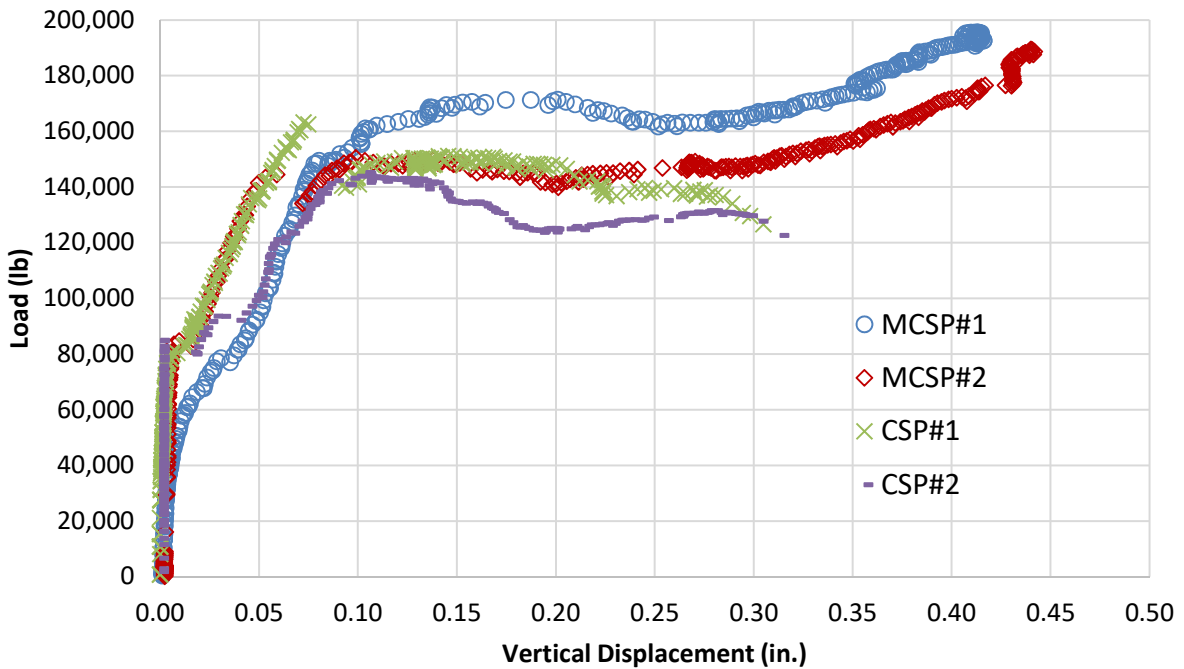


Figure 12. Load-vertical displacement relationships of all specimens. Note: CSP = conventional shear pocket; MCSP = mechanically connected shear pocket. 1 in. = 25.4 mm; 1 lb = 4.448 N.



Figure 13. Failure mode of mechanically connected shear pocket connections.



Figure 14. Failure mode of conventional shear-pocket connections.

$$f_d = \text{yield strength of the shear connectors beyond 60 ksi}$$

$$d_b = \text{diameter of a single shear connector}$$

$$l_a = \text{embedment length of the shear connector}$$

$$V_{dowel} = \frac{2 \times 1 \times (105 - 60) \times 1.5^3}{3.5 \times 4.5}$$

$$= 19.3 \text{ kip (85.8 kN)} < 36.6 \text{ kip (162.8 kN)}$$

Thus, $V_{ni} + V_{dowel} = 165.6 + 19.3 = 184.9 \text{ kip (822.4 kN)}$, which represents the predicted capacity of the mechanically connected shear pocket.

Table 1 summarizes push-off test results of the four specimens and includes the mode of failure and predicted nominal interface shear resistance. Interface shear resistance provisions of section 5.8.4 of the AASHTO LRFD specifications¹² were used to calculate the predicted value for CSP connections as presented in the design example. However, for MCSP connections, an additional term that accounts for the dowel effect of the connector was added based on the work of Hatami,⁵ which is presented in detail in the design example. The ratio of measured-to-predicted nominal interface shear resistance for the CSP specimens (0.93) indicates that the interface shear capacity of CSP connections is slightly less than predicted by the AASHTO LRFD specifications, which could be attributed to inadequate development of the shear connector in the shear-pocket concrete, inadequate strength of the shear-pocket concrete and grout, or both. However, the ratio of measured-to-predicted nominal interface shear resistance for the MCSP connections (1.04) indicates that the interface shear capacity of MCSP connections can be accurately predicted by the AASHTO LRFD specifications after accounting for the dowel effect, which is the advantage of the MCSP connections as they fully develop the shear connectors.

Conclusion

This paper presented the design and testing of a new shear-pocket connection for full-depth precast concrete deck construction. The new connection mechanically ties the

precast concrete deck panels to the supporting concrete girder while accommodating production and erection tolerances. Push-off testing was conducted to compare the performance of the new connection with that of CSP connections. Based on the test results, the following conclusions could be made:

- Using the MCSP increased the interface shear resistance of the connection by 25% compared with using a CSP.
- The constituent components of the new connection are readily available and can be easily and economically fabricated and assembled while accommodating current production and erection tolerances.
- The new mechanical connection fully utilizes the dowel effect of the shear connectors as they are fully anchored in the shear pocket. This results in a more ductile behavior than that of CSP connections controlled by the brittle behavior of concrete breakout, which is highly dependent on the strength and quality of concrete and grout filling material.
- The AASHTO LRFD specifications¹² provisions of interface shear resistance can be used to accurately predict the capacity of the new connection after accounting for the dowel effect of the shear connectors.

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Table 1. Summary of push-off test results

Specimen ID	$V_{measured}^a$ kip	Average $V_{measured}^a$ kip	$V_{predicted}^b$ kip	$\frac{V_{measured}}{V_{predicted}}$	Mode of failure
MCSP#1	195.5	192.5	184.9	1.04	Connector was sheared off; no concrete failure in shear pocket
MCSP#2	189.4				
CSP#1	163.2	154.3	165.6	0.93	Connector was bent; concrete breakout in the shear pocket
CSP#2	145.3				

Note: CSP = conventional shear pocket; MCSP = mechanically connected shear pocket; $V_{measured}$ = measured interface shear resistance;

$V_{predicted}$ = predicted nominal interface shear resistance. 1 kip = 4.448 kN.

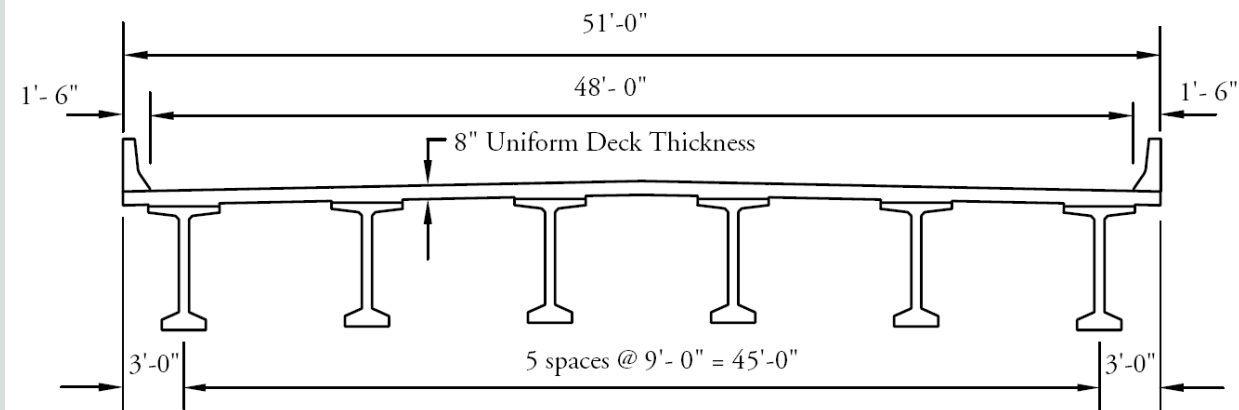


Figure 15. Bridge cross section. Source: Reproduced by permission from PCI (2011). Note: 1" = 1 in. = 25.4 mm; 1' = 1 ft = 0.305 m.

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Notation

- | | |
|----------|--|
| A_{cv} | = area of concrete considered to be engaged in interface shear transfer |
| A_{vf} | = area of interface shear connectors crossing the shear plane within the area A_{cv} |
| c | = cohesion factor |
| d_b | = diameter of a single shear connector |
| f'_c | = specified concrete compressive strength |
| f_d | = yield strength of the shear connectors beyond 60 ksi |
| f_y | = yield stress of interface shear connectors but not more than 60 ksi |
| k | = factor 2.0 for headed shear connectors and 1.0 for connectors without head |
| K_1 | = fraction of concrete strength available to resist interface shear |
| K_2 | = factor limiting interface shear resistance |
| l_a | = embedment length of the shear connector |
| n | = number of shear connectors |

P_c = permanent net compressive force normal to the shear plane

V_{dowel} = dowel effect contribution to interface shear resistance

V_h = factored horizontal shear force

$V_{measured}$ = measured interface shear resistance

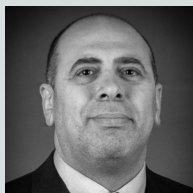
V_{ni} = nominal interface shear resistance

$V_{predicted}$ = predicted nominal interface shear resistance

μ = friction factor

ϕ = shear resistance reduction factor

About the authors



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Abstract

Full-depth precast concrete deck panels are commonly used for accelerated bridge construction and for their production quality and durability. Full-depth precast concrete deck panels are often made composite with the supporting girders by designing deck-to-girder connections to resist horizontal shear between the two components. Grouted open channels or shear pockets in deck panels around the shear connectors of the girders is the current practice. The structural performance of these connections is highly dependent on grout quality and strength, and their constructibility is restricted by production and erection tolerances. This paper presents a new mechanical deck-to-girder connection that minimizes these drawbacks as it connects the precast concrete deck panels to concrete girders mechanically. The performance of the new connection is less dependent on grout quality and strength while allowing relatively high production/construction tolerances. An experimental investigation was conducted to evaluate the constructibility and structural performance of the mechanical connection and compare them with those of conventional connections. Test results indicated that the interface shear resistance of the mechanical connection was 25% higher than that of conventional connections, in addition to its enhanced constructibility.

Keywords

Bridge girder, composite section, full-depth deck panel, mechanical connection, shear-pocket connection.

Review policy

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

Reader comments

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