Using full-depth precast concrete panels to replace a bridge deck is an innovative approach. The major advantages of the precast concrete system are speed of construction, durability of the bridge deck, and full composite action between the bridge deck and the precast, prestressed concrete girders. Composite action can be achieved between the precast concrete panels and the precast concrete girders by using threaded steel bolts as shear connectors. Achieving efficiency in the placement of steel bolts in shear pockets, either spaced 2 ft (610 mm) on center or as required by the design, is of particular interest in this type of connection. Thirteen push-out specimens with shear pockets in the slabs were cast and tested under static loading to determine the ultimate strength and the mode of failure of the shear connection system. Parameters considered in this study were length of the steel bolts and number and configuration of steel bolts in the shear pocket. Experimental test results indicate that threaded steel bolts are capable of allowing the precast concrete panels and precast concrete girders to achieve full composite action. It was also determined that the application of the American Association of State Highway and Transportation Officials load and resistance factor design guidelines to determine the horizontal shear strength of the proposed type of shear connection conservatively estimates the design strength.
Because of structural deficiencies in a significant number of highway bridges in the United States, many are in need of replacement or rehabilitation. When considering the type and extent of a bridge rehabilitation program, transportation agencies are influenced mainly in terms of time and cost of construction. Recent rehabilitation programs of existing major highway bridges with high traffic volumes have increasingly used full-depth precast concrete deck slabs to shorten the time of construction and bridge closures.

The magnitude of the current bridge rehabilitation program can be appreciated from the records of the Federal Highway Agency (FHWA). Approximately half of the 600,000 highway bridges in the United States were built before 1940. Oehler, Seracino, and Yeo estimated that 30 bridges across the nation reach their design life every day. Moreover, the National Bridge Inventory (NBI) reported that 14% and 24% of the bridges classified as structurally deficient and functionally obsolete are exclusively made of concrete or prestressed concrete, respectively. In the past, full-depth precast concrete panels have been placed primarily on bridges with steel-girder supporting systems; however, the same panels can be used to replace deteriorated decks on precast, prestressed concrete girders.

In this study, the ability of a bridge’s superstructure system, consisting of precast, prestressed concrete I-girders (American Association of State Highway and Transportation Officials [AASHTO] type) and full-depth precast concrete slabs, to behave compositely is investigated. The desired composite behavior between the panels and girders can most practically be attained with flexible, mechanical shear connectors. The most common flexible, mechanical shear connectors currently used in practice are either welded studs (for steel girders) or embedded steel stirrups (protruding from concrete girders). For the proposed bridge superstructure system, threaded steel bolts are evaluated as a shear connector. For bridge deck replacement, these bolts would resist interfacial horizontal shear between the precast concrete panels and girders.

### Research Significance

Shear connectors are needed to achieve full composite action between the full-depth precast concrete slabs and the AASHTO-type precast concrete girders in a bridge superstructure. This study investigates the possibility of using threaded steel bolts as shear connectors between AASHTO-type concrete girders and full-depth precast concrete slabs in a bridge deck replacement system. The primary focus area was to evaluate the horizontal shear strength of the slab-girder system using different configurations of threaded steel bolts. In addition, the experimental test results were compared with the horizontal shear strength predicted by AASHTO and American Concrete Institute (ACI) equations.

### Objectives

The main objective of this study was to evaluate the composite behavior of a bridge section that consisted of a full-depth precast concrete slab connected to an AASHTO-type concrete I-girder using threaded steel bolts. The following test results were used to evaluate the proposed objective:

- The ultimate horizontal shear strength provided by the threaded steel bolts.
- The slip behavior at the interface of the full-depth precast concrete slab and the precast concrete beam.
- The effect of the number and configuration of bolts in a shear pocket on the ultimate horizontal shear strength.
- The mode of failure of the shear connection system.
- The horizontal shear strength predicted by the AASHTO and ACI equations.

### Literature Review

This study began because of the successful development of an analogous bridge superstructure system made with steel girders and full-depth precast, prestressed concrete slabs. A study of the development of the steel-precast system, which is backed by substantial research, was helpful to the authors when selecting procedures and parameters for this study. The literature on shear connectors refers mostly to applications in steel girder–precast concrete slab composite bridge systems. The literature covers substantial investigations of different types of mechanical shear connectors that have been studied since the mid-1950s. More recently, most of the studies of connections between concrete girders and panels have focused on studs as shear connectors due to their high demand in practice. Tadros et al. presented a debonded shear key system for prestressed concrete bridge girders with composite cast-in-place decks. The proposed system utilized the mechanical anchorage of concrete shear keys created on the top flange of a concrete girder combined with shear reinforcement crossing the interface. Their results showed that the proposed system performed well and had no detrimental effects on composite action or bridge stiffness.

Dedic compared the shear strength of studs and high-strength bolts. From the results of push-out tests, it was found that a similar behavior existed between the two connector types when loaded statically. The average ultimate strength was found to be slightly lower for two studs (58.1 kip [258 kN]) compared with that for two bolts (68.3 kip [304 kN]). The difference in average ultimate strength was attributed to the larger tensile strength of the bolts and the way the connectors were attached to the steel beam. The bolts were tightened with two nuts to the steel beam flanges, which was believed to have added stiffness to the shear connection. The studs, on the other hand, were welded to the beam. The mode of failure for both cases was the same: fracture of the bolt above the steel beam or at the weld location.

A similar type of shear connection was applied during the rehabilitation of the Well and River Bridge in Ontario, Canada. The project involved replacing the bridge deck while utilizing the original 50-year-old steel girders. The main purpose of the project was to achieve composite action between the deck and steel girders so that the bridge could support current local highway bridge design loads. The study included the effect of having different geometric configurations of studs in slab blockouts and that of having different heights of studs.
Table 1. Test Specimen Designation and Description

<table>
<thead>
<tr>
<th>Designation</th>
<th>Number of Specimens</th>
<th>Number of Bolts per Pocket</th>
<th>Size of Bolt, in.</th>
<th>Bolt Configuration</th>
</tr>
</thead>
<tbody>
<tr>
<td>FCS0</td>
<td>1</td>
<td>0</td>
<td>NA</td>
<td>NA</td>
</tr>
<tr>
<td>FC8S1</td>
<td>1</td>
<td>1</td>
<td>8</td>
<td>One centered</td>
</tr>
<tr>
<td>FC10S1</td>
<td>1</td>
<td>1</td>
<td>10</td>
<td>One centered</td>
</tr>
<tr>
<td>FC8S2L</td>
<td>1</td>
<td>2</td>
<td>8</td>
<td>Two perpendicular to traffic flow</td>
</tr>
<tr>
<td>FC10S2L</td>
<td>2</td>
<td>2</td>
<td>10</td>
<td>Two perpendicular to traffic flow</td>
</tr>
<tr>
<td>FC8S2P</td>
<td>1</td>
<td>2</td>
<td>8</td>
<td>Two parallel to traffic flow</td>
</tr>
<tr>
<td>FC10S2P</td>
<td>1</td>
<td>2</td>
<td>10</td>
<td>Two parallel to traffic flow</td>
</tr>
<tr>
<td>FC8S3L</td>
<td>1</td>
<td>3</td>
<td>8</td>
<td>Three perpendicular to traffic flow</td>
</tr>
<tr>
<td>FC10S3L</td>
<td>2</td>
<td>3</td>
<td>10</td>
<td>Three perpendicular to traffic flow</td>
</tr>
<tr>
<td>FC8S3V</td>
<td>1</td>
<td>3</td>
<td>8</td>
<td>Three in V-shaped line</td>
</tr>
<tr>
<td>FC10S3V</td>
<td>1</td>
<td>3</td>
<td>10</td>
<td>Three in V-shaped line</td>
</tr>
</tbody>
</table>

Note: NA = not applicable; 1 in. = 25.4 mm.

in a shear pocket. The conclusions from the research and the rehabilitation project confirmed that grouped shear connectors effectively produce composite action and that different heights of shear connectors in the same pocket will improve the connectors' performances. Additionally, it was observed that the shear transfer due to interface bond and friction was satisfactorily gained for the serviceability and fatigue limit states, but not for the ultimate limit state. However, the effect of different geometric configurations of studs did not seem to be significant, as the researcher does not mention any such related effect. The project simply opted for an orthogonal arrangement of eight studs in each shear pocket.

Shim et al. concluded that the cross-sectional area of a stud, height of a stud, and material properties of the surrounding concrete determine the strength, stiffness, and ultimate slip capacities of the shear connectors. The authors performed push-out tests under fatigue and ultimate loading to determine whether the studs exhibited integral behavior with the

Fig. 1. Top view of steel bolt configurations in a shear pocket. Note: 1 in. = 25.4 mm.
surrounding mortar. They reported that the studs failed with a large deformation along the shafts’ axes and, consequently, by shearing off at the weld level when statically loaded. They also reported splitting cracks in the haunch.

In recent research on the behavior of studs in shear pockets for precast concrete slabs, Issa et al. stated that the spacing of shear pockets depends on the configuration of the shear studs, which might be positioned transversely or longitudinally with respect to the bridge’s girders. Through extensive experimental work and finite element modeling, one of the recommendations from the authors was to use a spacing of 24 in. (610 mm) between shear pockets. The shear connection investigated consisted of 7/8-in.-diameter (22 mm) studs and a nonshrink cementitious grout in the shear pockets and haunches. Results from testing 28 push-out specimens indicated that the ultimate horizontal load resisted by the connection did not increase proportionately to the increase in the number of studs in a shear pocket. For instance, for the full-scale specimens with one shear pocket, the average ultimate strength was 69.2 kip (308 kN) for two studs, 92.9 kip (413 kN) for three studs (34% increase), and 121.4 kip (540 kN) for four studs (75.4% increase). Additionally, it was concluded that the configuration of studs in a shear pocket affected the slip behavior.

**EXPERIMENTAL PROGRAM**

The efficiency and adequacy of threaded steel bolts as shear connectors between full-depth precast concrete slabs and precast concrete girders were evaluated in a proposed bridge rehabilitation system. A total of 13 push-out specimens were cast and tested. One specimen was cast without bolts and its shear pocket was grouted with nonshrink cementitious grout to observe the contribution of grout materials to the specimen’s horizontal shear strength. The other 12 specimens were cast with one, two, or three bolts in each shear pocket and in various configurations. Table 1 shows the designations of the specimens, and Fig. 1 shows five configuration descriptions of the bolts. In the specimen designations, the first two letters, “FC”, stand for “full scale;” the number that follows indicates the height of bolt used; the letter “S” stands for “bolt-shear connector;” then the number of bolts per shear pocket follows; and finally, an arbitrary letter representing the geometric configuration is used.

An economically feasible steel bolt was the most attractive type for this study. The selected threaded steel bolts are specified as Grade 2, which is equivalent to the common bolt ASTM A307. The bolt manufacturer reported a yield strength of 36,000 psi (250 MPa) and a tensile strength of 58,000 psi (400 MPa). Parameters considered in this study are the number of bolts in a shear pocket, the configuration of bolts in a shear pocket, and the height of the bolts above the bottom of the slab. The diameter of the bolts, 1 in. (25 mm), was kept constant throughout this investigation.

Bolt lengths of 8 in. and 10 in. (200 mm and 250 mm) were used. The embedment lengths of the bolts into the precast concrete slab were 2 in. (50 mm) for the 8-in.-long (200 mm) bolt and 4 in. (100 mm) for the 10-in.-long (250 mm) bolt, measured from the bottom of the precast concrete slab. The depth of the haunch was 1 in. (25 mm). An embedded length of 5 in. (125 mm) of the bolts into the flanges of the precast concrete girder was kept constant in all specimens. Both bolts had a threaded length of 3 in. (75 mm) at one end, which was embedded in the flanges of the precast concrete girder segments. Table 1 and Fig. 2 show the assigned embedment lengths of a bolt’s shaft into the girder and the shear pocket. Note that such positioning of the bolt in a section meets the AASHTO standard specifications (Article 10.38.2.3).

To verify the experimental test results for each bolt con-

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**Table 2. Mixture Proportions for the Concrete for the Slab and Beam Specimens**

<table>
<thead>
<tr>
<th>Ingredients</th>
<th>Slab Concrete Proportions, per yd³ (m³)</th>
<th>Beam Concrete Proportions, per yd³ (m³)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Cement Type I, lb (kg)</td>
<td>683 (310)</td>
<td>660 (300)</td>
</tr>
<tr>
<td>Coarse aggregate, lb (kg)</td>
<td>1729 (785)</td>
<td>1798 (816)</td>
</tr>
<tr>
<td>Fine aggregate, lb (kg)</td>
<td>1088 (494)</td>
<td>1107 (503)</td>
</tr>
<tr>
<td>Silica fume, lb (kg)</td>
<td>68 (31)</td>
<td>–</td>
</tr>
<tr>
<td>Water, lb (kg)</td>
<td>271 (123)</td>
<td>290 (132)</td>
</tr>
<tr>
<td>Water-cement ratio</td>
<td>0.36</td>
<td>0.44</td>
</tr>
<tr>
<td>High-range water-reducing admixture</td>
<td>as required</td>
<td>as required</td>
</tr>
<tr>
<td>Air-entraining admixture</td>
<td>as required</td>
<td>as required</td>
</tr>
</tbody>
</table>
Table 3. Material Strength Properties of Concrete, Steel Bolts, and Epoxy

<table>
<thead>
<tr>
<th>Concrete Slab</th>
<th>Concrete Girder</th>
<th>Steel Bolts</th>
<th>Epoxy</th>
</tr>
</thead>
<tbody>
<tr>
<td>$f_c = 8900$ psi</td>
<td>$f_c = 8400$ psi</td>
<td>$f_y = 36,000$ psi</td>
<td>$f_y = 5100$ psi</td>
</tr>
<tr>
<td>$E_c = 5400$ ksi</td>
<td>$E_c = 5100$ ksi</td>
<td>$f_u = 58,000$ psi</td>
<td>$f_u = 5900$ psi</td>
</tr>
<tr>
<td>$\nu = 0.2$</td>
<td>$\nu = 0.2$</td>
<td>$E_c = 0.3$</td>
<td>$E_c = 320$ ksi</td>
</tr>
</tbody>
</table>

Note: $f_c =$ concrete compressive strength; $E_c =$ modulus of elasticity; $\nu =$ Poisson’s ratio; $f_y =$ steel yield strength; $f_u =$ steel ultimate strength; $f_c =$ epoxy tensile strength; $f_{\tau} =$ epoxy shear strength; $E_c =$ modulus of elasticity; 1 psi = 0.0069 MPa; 1 ksi = 6.895 MPa.

figuration, two similar specimens were fabricated for each bolt configuration, except that one specimen was made with 8-in.-long (200 mm) bolts and the other with 10-in.-long (250 mm) bolts.

Material Properties

High-performance concrete (HPC) with a design compressive strength $f_c$ of 8900 psi (61 MPa) was used to fabricate the precast concrete slabs. This type of concrete simulates the behavior of HPC commonly used for prestressed and post-tensioned concrete bridge elements, which has a minimum $f_c$ of 6000 psi (41 MPa). Table 2 shows the mixture proportions used in the fabrication of the slab blocks and the girder segments for the test specimens. Table 3 shows the pertinent material properties for each of the push-out specimen components used. The slab blocks were fabricated at the concrete laboratory at the University of Illinois at Chicago, while the precast concrete girders were produced by the Prestress Engineering Corp. in Blackstone, Ill.

Slab Specimens

The slab blocks were designed with a typical steel reinforcement arrangement. Each slab had the overall dimensions of 27 in. × 24 in. × 8 in. (690 mm × 610 mm × 200 mm) with a shear pocket in the shape of a centered, beveled hole. The dimensions of the shear pocket were 11 in. × 6 in. (280 mm × 150 mm) at the top and 10 in. × 5 in. (250 mm × 125 mm) at the bottom of the slab. The steel reinforcement at the top and bottom of the slab consisted of a grid of No. 5 (15M) bars with a spacing of 6 in. (150 mm) on center. The reinforcement was designed by AASHTO standard specifications, with the main reinforcement perpendicular to traffic flow. A concrete cover of 2 in. (50 mm) at the top and 1 in. (25 mm) at the bottom of the slab was used according to AASHTO Section 9.26.1.2. The slab blocks were cast in pairs, and each was thoroughly vibrated. Three 6 in. × 12 in. (150 mm × 305 mm) concrete cylinders were cast and tested according to ASTM C39 for each batch. The slab blocks and cylinders were cured with wet burlap for seven days and then set in the laboratory environment until testing.

Girder Segments

AASHTO Type II girder segments (stubs) were used in this study. The cross-sectional shape was modified for testing purposes. Figure 3 shows that the girder segments had two top flanges of identical shapes to attain symmetry in testing. The height of this symmetrical cross section also reduced from 36 in. (910 mm) to 30 in. (760 mm) in order to fit the specimen in the testing frame at the concrete laboratory. Two No. 5 (15M) reinforcing steel bars were placed in each flange, and No. 4 (13M) bars, spaced 6 in. (150 mm) on center, were used as web shear reinforcement.

Fig. 3. Instrumentation setup and dimensions of test specimens. Note: 1 in. = 25.4 mm; LVDT = linear variable displacement transducer.
Specimen Assembly

The shear connection between the girder segment and the slab blocks was created by drilling 5 in. (125 mm) into the flanges of the girder segments. The embedment length of all bolts in all specimens was kept constant at 5 in. (125 mm). The specimen surfaces were properly cleaned for optimum bonding, but they were not intentionally roughened. The shear-pocket walls were sandblasted to expose the aggregate for better bonding with the grout. The drilled holes were cleaned with a wire brush, and then the dust was removed with an air hose. Assembly of the components in the test specimen was carried out to simulate the typical procedure used in the field. With the symmetrical girder standing on one of its flanges, the slab block was positioned on top of the other flange. The slab block rested on spacers to provide a minimum haunch of 1 in. (25 mm) between the bottom of the slab and the top surface of the flange. Haunches of this dimension are normally used for this type of full-depth precast concrete slab construction to allow for the required leveling across a section of a bridge. The formwork for the haunch was then positioned.

Epoxy grout was mixed and placed into each drilled hole to a certain height. Each bolt was inserted into the drilled hole with a twisting motion about its shaft to avoid any uneven distribution of epoxy, especially in the threaded region. Care was taken to position each bolt upright and not in contact with any of the sides of the hole as this would have diminished the contact effect with the epoxy. While looking from above the shear pocket, more epoxy was carefully added to the hole to a point where a ring of epoxy around the shaft of the bolt was clearly visible and just below the flange’s top surface. The epoxy was then left to dry with the bolt firmly positioned for one day. Fast-set epoxy grout, which can set within one or two hours, is recommended to avoid offsetting the rapid construction benefits of using the full-depth precast concrete panels. Nonshrink cementitious grout was used for filling the haunch and the shear pocket areas. The flowable grout was made with 0.85 gal. (3.2 L) of water for every 50 lb (23 kg) of grout material. The mixed grout was placed into the shear pocket and then thoroughly vibrated to make sure it was evenly distributed within the haunch and shear pocket areas. Also, for each grout placement, 4 in. x 8 in. (100 mm x 200 mm) control grout cylinders and 2 in. (50 mm) cubes were cast. The grout was cured with wet burlap for one day. After three days, the specimen was turned upside down to assemble the other side of the slab-girder connection and to grout the shear pocket and slab haunch. The same procedure was used in all test specimens. The slabs were carefully aligned to create a symmetrical testing setup.

PUSH-OUT TEST SETUP

The push-out test was selected because it historically has been employed to obtain the ultimate horizontal shear strength as well as the slip capacity of shear connectors experimentally. Many researchers have found this test setup to be the most reliable method in obtaining the load-slip behavior of connectors. In fact, this type of test has been favored over more realistic, but also more costly, methods, such as testing of full-scale superstructure bridge systems. Results from these tests indicate that the push-out tests yield more conservative values than those obtained from the other methods. The test method is a simple and safe alternative for quantitatively determining the strength of connectors. In the analogous case of steel girder–concrete slab systems, push-out tests also produced conservative design values for fatigue endurance (cyclic) testing of shear connectors.

The relative slip between the precast concrete slab blocks and the precast concrete girder segment was measured using four linear variable displacement transducers (LVDTs). The LVDTs were attached to the sides of the girder’s flanges at the centerline of the shear pockets and were in contact with the bottom side of the slabs. For recording purposes, one side of each test specimen was called Side A, while the other side was called Side B (Fig. 3). The measurements from the two LVDTs on each side of the test specimen (A or B) were averaged and recorded to obtain the slip for that side. Figures 3 and 4 show the connection of LVDTs to the specimen components. The loading apparatus consisted of two synchronized hydraulic cylinders. Data were collected and recorded with a data acquisition system. Figure 5 shows the test setup.

Fig. 4. Connection of linear variable displacement transducers (LVDTs) to the concrete components.

Fig. 5. Test setup. Note: LVDT = linear variable displacement transducer.
DISCUSSION OF TEST RESULTS

Observations during testing were recorded to understand the deformations and modes of failure of the push-out test specimens. Various comparisons and correlations of test results were made to determine the effectiveness of threaded steel bolts as shear connectors. Each test began with small incremental load steps to ensure even distribution of applied load throughout the concrete girder segment. Testing then progressed with larger uniform load steps until the ultimate load of the specimen was reached. For each load increment, the applied load and the corresponding slip between the precast concrete girder and the precast concrete slab blocks were recorded. For the majority of the test specimens, the load remained nearly constant for some time after the ultimate load was reached.

Crack Development

All the specimens exhibited a similar trend in behavior during testing. It was observed that when the applied load approached 30 kip to 37 kip (133 kN to 164 kN), a fine crack developed very close to the connector level, about 1 in. (25 mm) deep in the haunch. This single crack was common to each side where an LVDT was placed. As the load increased, crack widths increased, running along both edges of the haunch. More specifically, these cracks ran from the top of the test specimen, forming a gap between the haunch and the flange of the girder, and continued propagating vertically down along the slab-haunch interface. During this testing stage, investigators could hear noises as a result of the interface bond failure.

Crack size increased as the load increased, followed by an instantaneous loud noise. The loud noise was attributed to the...
complete separation of the haunch from the specimen components (debonding). For most of the tested specimens, the load at this stage is considered the ultimate load. Thereafter, bolts at the interface carried the applied load. At this stage, the bolts showed that their flexural bending stiffness and yielding characteristics impeded the shear connection from abruptly slipping at larger loading rates. Figure 6 shows that the haunch grout failed in shear followed by yielding of the bolts. Additionally, cracks were observed in the webs of the precast concrete girder segments during testing and were found to affect the outcome of the tests significantly. In two tests, crack formation started in the web at the top of the specimen and extended to the flanges. For the majority of the tests, however, the cracks in the web were not a concern, as addressed later in the discussion of test results.

Force-Slip Behavior

Slip was measured and recorded for each load increment and plotted versus the corresponding load. Figures 7 through 12 show the load-slip curves of all shear pocket configurations for Sides A and B. For all specimens, the flanges of the precast concrete girder segment were not intentionally roughened. Inspection of Fig. 7 for the specimen without bolts shows a curve with virtually no slip. This test simply shows the strength of the frictional bond between the haunch and the concrete surfaces. This test (of the specimen without bolts) allowed the observation of the bond contribution of the haunch grout material to the strength of the connection system.

Each load-slip curve in Fig. 8 through 12 shows two specimens with the same bolt configuration in each shear pocket but with different bolt lengths. These figures show that the effect of the bolt length embedded into the precast concrete slab on the ultimate strength is minimal, except for the specimens with one bolt per shear pocket.

Specimens with One Centered Bolt

Figure 8 shows the load-slip curves for the specimens with or one bolt 8 in. or 10 in. (200 mm or 250 mm) long centered in the shear pocket. The push-out test specimen with the 8-in.-long (200 mm) bolt exhibited symmetric behavior in that the slip values from Sides A and B were nearly identical. Inspection of these curves revealed that once the ultimate load is reached in these specimens, the bond interface failed and the load trans-
ferred to the shear connectors. For the 10-in.-long (250 mm) bolt specimen, one side reached the ultimate load sooner than the other. This might have occurred due to a slightly unsymmetrical distribution of the load through the specimen. This is also valid for the rest of the specimens tested, and observations from the figures show that this effect has no influence on the slip capacity and ultimate strength of the specimens. The ultimate shear strengths of the specimens tested with one centered bolt, for bolt lengths of 8 in. (200 mm) and 10 in. (250 mm), were 37.2 kip and 61.4 kip (165 kN and 273 kN), respectively. Comparing these strengths, a difference of 24 kip (107 kN), in favor of the 10 in. (250 mm) bolt specimen, is evident. This difference seems rather large for the 10 in. (250 mm) bolts with 4 in. (100 mm) embedment into the precast concrete slab compared with the 8 in. (200 mm) bolts with 2 in. (50 mm) embedment into the precast concrete slab. Consequently, due to the limited number of tests, specimens with a single bolt in a shear pocket were not considered further in this study.

Specimens with Two Bolts Perpendicular to Traffic

Figure 9 shows the load-slip curves for the specimens with two bolts in the shear pocket and oriented perpendicular to traffic flow for both lengths 8 in. and 10 in. (200 mm and 250 mm). The ultimate shear strength of the specimen with the 8-in.-long (200 mm) bolts was 69.2 kip (308 kN), while for the 10-in.-long (250 mm) bolts it was 73.2 kip (325.6 kN). It is clear that the 10-in.-long (250 mm) bolt specimen had a slightly larger strength than the 8-in.-long (200 mm) bolt specimen. The difference between the specimens containing the 8 in. (200 mm) and the 10 in. (250 mm) bolts was about 4 kip (18 KN). In both specimens, the load at ultimate was sustained until the bond at the interface failed.

Specimens with Two Bolts Parallel to Traffic

Figure 10 shows the load-slip curve for the specimen with two bolts oriented parallel to the traffic flow. The ultimate shear strength of the specimen with the 8-in.-long (200 mm) bolts was 65.7 kip (292 kN), while the strength was 68.6 kip (305 kN) for the 10-in.-long (250 mm) bolts. The 10-in.-long (250 mm) bolt specimen showed slightly higher shear strength than the 8-in.-long (200 mm) bolt specimen by as much as 3 kip (13 kN). The load in both cases was sustained until bond failure.

Specimens with Three Bolts Perpendicular to Traffic

Figure 11 shows the load-slip curves for the specimens with three bolts perpendicular to the traffic flow for 8-in.-long and 10-in.-long (200 mm and 250 mm) bolts. The ultimate shear strength of the specimen with the 8-in.-long (200 mm) bolts was 94.2 kip (419 kN), while for the 10-in.-long (250 mm) bolts it was 104.5 kip (465 kN). The difference in shear strength of 10.3 kip (46 kN) in favor of the 10-in.-long (250 mm) bolts is obvious. The 10-in.-long (250 mm) bolt specimen showed a larger slip capacity—that is, a larger slip at ultimate load and larger horizontal shear strength—than that of the 8 in. (200 mm) bolt specimen. The specimens with the three-bolts-per-pocket configurations presented some difficulties during testing. Due to the stress concentration at the top of the precast concrete girder segments, cracks initiated in the girder's web and then extended to the flanges. The cracks initiated in the precast concrete girder and kept propagating down in the web with some inclination. In some instances during testing, LVDTs were detached suddenly from the specimen as a result of excessive cracking.

Specimens with Three Bolts in a V-Shaped Line Configuration

For the case of three bolts in a V-shaped line configuration, the problems of excessive cracking in the concrete girder mentioned previously (premature cracks in the web) led to the conclusion that the ultimate load for the 10 in. (250 mm) bolt specimen was not reached. Figure 12 shows the distinct appearance of the curves for Sides A and B, which demonstrate the irregular and non-uniform slip measurements.

FINITE ELEMENT ANALYSIS

Nonlinear finite element analysis (FEA) using ANSYS® software, version 9, was used to simulate the behavior of the test specimens, including those with and without steel bolts. ANSYS element SOLID65 (3-D Reinforced Concrete Solid) was selected to model the concrete for the slabs and the gird-

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**Fig. 13.** Types and properties of the element used in the finite element analysis (FEA).

**Fig. 14.** Finite element analysis (FEA) simulation and mesh.
er. The concrete failure criterion was chosen based on the William-Warnke model. An idealized Hognestad concrete stress-strain curve was used to model the strength properties of concrete. Table 3 presents the compressive strength, elastic modulus, and Poisson’s ratio of concrete. The steel reinforcement was modeled using the ANSYS LINK8-3D element. It was assumed that the steel was an elastic-plastic material, identical in tension and compression, with a yield strength $f_y$ of 60 ksi (410 MPa), an elastic modulus of 29,000 ksi (200 GPa), and a Poisson’s ratio of 0.3. ANSYS element SOLID 45 was used for modeling the loading steel plate, the supporting steel plates, and the steel bolts. An elastic modulus of 29,000 ksi (200 GPa) and a Poisson’s ratio of 0.3 were also used as the properties of the steel plates and bolts. The steel bolts were modeled as an elastic-plastic material, identical in tension and compression, with a yield strength $f_y$ of 36 ksi (250 MPa). The steel plates were assumed to be made of linear-elastic material. The epoxy was modeled as a layered solid element, ANSYS element SOLID46, and was assumed to be a linear-elastic material. Table 3 presents the material

**Table 4. Experimental Test Results and Predicted Horizontal Shear Strength**

<table>
<thead>
<tr>
<th>No. of Bolts</th>
<th>Length of Bolts, in. (mm)</th>
<th>Specimen Designation</th>
<th>Horizontal Shear Strength, kip (kN)</th>
<th>Horizontal Shear Strength (ANSYS), kip (kN)</th>
<th>Average Horizontal Shear Strength, kip (kN)*</th>
<th>Average Horizontal Shear Strength, kip (kN)**</th>
<th>AASHTO LRFD, kip (kN)‡</th>
<th>Grout Compressive Strength, ksi (MPa)</th>
<th>Mode of Failure</th>
</tr>
</thead>
<tbody>
<tr>
<td>none</td>
<td>—</td>
<td>FC8S0</td>
<td>22.8 (101)</td>
<td>23.1 (103)</td>
<td>22.8 (101)</td>
<td>22.8 (101)</td>
<td>22.5 (100)</td>
<td>7.7 (53)</td>
<td>S</td>
</tr>
<tr>
<td>1</td>
<td>8 (200)</td>
<td>FC10S1</td>
<td>37.2 (165)</td>
<td>35.5 (165)</td>
<td>37.2 (165)</td>
<td>49.3 (219)</td>
<td>36.0 (160)</td>
<td>6.8 (47)</td>
<td>Y</td>
</tr>
<tr>
<td></td>
<td>10 (250)</td>
<td>FC10S2L</td>
<td>61.4 (273)</td>
<td>59.4 (264)</td>
<td>61.4 (273)</td>
<td>67.5 (300)</td>
<td>50.0 (222)</td>
<td>7.4 (51)</td>
<td>Y</td>
</tr>
<tr>
<td>2</td>
<td>8 (200)</td>
<td>FC8S2L</td>
<td>69.2 (308)</td>
<td>66.3 (292)</td>
<td>69.2 (308)</td>
<td>70.9 (315)</td>
<td>7.4 (51)</td>
<td>8.1 (56)</td>
<td>Y</td>
</tr>
<tr>
<td></td>
<td>10 (250)</td>
<td>FC8S2P</td>
<td>65.7 (292)</td>
<td>63.6 (283)</td>
<td>65.7 (292)</td>
<td>65.7 (292)</td>
<td>7.0 (48)</td>
<td>Y</td>
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</tr>
<tr>
<td></td>
<td></td>
<td>FC10S2L</td>
<td>73.2 (326)</td>
<td>71.9 (320)</td>
<td>73.2 (326)</td>
<td>73.2 (326)</td>
<td>7.4 (51)</td>
<td>Y</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>FC10S2P</td>
<td>68.6 (305)</td>
<td>66.5 (296)</td>
<td>68.6 (305)</td>
<td>68.6 (305)</td>
<td>7.4 (51)</td>
<td>Y</td>
<td></td>
</tr>
<tr>
<td>3</td>
<td>8 (200)</td>
<td>FC8S3L</td>
<td>94.2 (419)</td>
<td>92.6 (292)</td>
<td>94.2 (419)</td>
<td>96.2 (428)</td>
<td>5.9 (41)</td>
<td>Y</td>
<td></td>
</tr>
<tr>
<td></td>
<td>10 (250)</td>
<td>FC8S3V</td>
<td>98.1 (436)</td>
<td>96.4 (412)</td>
<td>98.1 (436)</td>
<td>98.1 (436)</td>
<td>6.1 (42)</td>
<td>Y</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>FC10S3L</td>
<td>104.5 (465)</td>
<td>102.6 (456)</td>
<td>104.5 (465)</td>
<td>104.5 (465)</td>
<td>7.1 (49)</td>
<td>Y</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>FC10S3V</td>
<td>84.2&quot; (375)</td>
<td>106.4 (473)</td>
<td>84.2&quot; (375)</td>
<td>106.4 (473)</td>
<td>6.9 (48)</td>
<td>Y</td>
<td></td>
</tr>
</tbody>
</table>

* Average ultimate strength based on configuration of bolts.
† Average ultimate strength based on number of bolts.
‡ Predicted horizontal shear strength based on AASHTO LRFD code.
** Premature failure (ultimate horizontal shear strength was not reached).

Note: AASHTO = American Association of State Highway and Transportation Officials; LRFD = load and resistance factor design; S = shear failure of the haunch grout; Y = flexural yielding of bolts, with haunch grout already having an S failure; 1 kip = 4.448 kN, 1 ksi = 6.895 MPa.
properties of all the specimen components. Figure 13 shows the element types used in the FEA.

By taking advantage of the symmetry of the specimen and the loading, only half of the specimen was analyzed by FEA, which reduced the computing time and computer disk space requirements significantly. At the plane of symmetry, the displacements in the directions perpendicular to the plane of symmetry were restrained. A convergence study was carried out to determine the appropriate mesh density. Figure 14 shows the typical FEA mesh. Perfect bonding was assumed between the steel reinforcement and the concrete, as well as between the epoxy and the steel bolts. The applied load was divided into a series of small load increments (0.1 kip [0.45 kN]) up to failure. Newton-Raphson equilibrium iterations were used to check the convergence at the end of each load increment. Failure of each model was identified when the solution for a load increment of 0.001 kip (0.0045 kN) did not converge.

Table 4 shows the ultimate horizontal shear forces predicted by the nonlinear FEA. Figure 15 shows a typical deformed shape and bending of the steel bolt at ultimate. Inspection of Table 4 reveals that there is a good agreement between the nonlinear FEA and the test results in terms of ultimate horizontal shear strength. Inspection of Fig. 6 and Fig. 15 reveal that there is a good agreement between the nonlinear FEA and the test results in terms of failure modes. The failure mode started by shearing and/or debonding of the grout in the haunch followed by bending of the steel bolts. Figure 16 shows a typical shear stress contours at failure.

Figures 17 and 18 show the load-slip curves obtained from the experimental test results and the FEA for the specimen without bolts, with one bolt centered, with two bolts perpendicular to traffic, and with three bolts perpendicular to traffic. Inspection of the load-slip curves, reveals that a good agreement was observed between the experimental test results and the nonlinear FEA.
The test results were compared to investigate the effect of the number, configuration, and length of bolts on their horizontal shear strength. Furthermore, the test results were also compared with the ACI 318-02 code, the AASHTO standard specifications, and the AASHTO LRFD specifications. The test results showed that the ultimate shear strengths of the 10-in.-long (250 mm) bolt specimens were slightly higher than those of the 8-in.-long (200 mm) bolt specimens. Inspection of Table 4 reveals that the longer bolts provided an increase of approximately 6% and 12% in horizontal shear strength of the specimens containing them for the cases of two bolts and three bolts per pocket, respectively. The only exception is the unexpectedly large horizontal shear stress obtained from the specimen having one 10-in.-long (250 mm) bolt per shear pocket (specimen FC 10S1). The other issue related to the effect of the bolt length is the slip resistance. The 10-in.-long (250 mm) bolt specimens consistently exhibited larger slip resistance values, especially for the three bolt configurations. In other words, the longer bolts experienced relatively more yielding at ultimate load, which increased the specimen capacity to slip more while sustaining the applied load.

The variation in ultimate shear strength seems to be almost insignificant when considering different bolt configurations and orientation for the same number of bolts. It is concluded that, due to the small variation in strength shown by the two-bolt and the three-bolt specimen configurations, the shear strength of the connection is not dependent on the configuration and placement of the bolts within a shear pocket as long as the number of bolts is the same. This could provide practical benefit for the construction of such systems. Lack of accuracy in drilling the holes can be tolerated so that speed in drilling can be achieved. Figure 19 shows that the ultimate shear capacity increases proportionally with the number of bolts per pocket. Additionally, Fig. 17 and 18 show the comparison of slip behavior for the different configurations of bolts.

Comparison of the test results with the applicable codes and specifications was made to predict the horizontal shear between the concrete girders and the precast concrete slabs. The area of the bolts used was 0.785 in.² (506 mm²) with a yield strength and an ultimate strength of 36,000 psi (248 MPa) and 58,000 psi (400 MPa), respectively. The contact surface area has a width of 12 in. (305 mm) and a length of 24 in. (610 mm). The shear connector blockouts were spaced at 2 ft (610 mm) on center where possible.

**Test Result Comparison with ACI 318-02**

ACI 318-02 Section 17.5.2 recommends that when contact surfaces are clean and intentionally roughened, the horizontal shear stress \( \nu_{sh} \) shall not be taken greater than 80 psi (0.56 MPa) (no shear reinforcement is required).

For the tested specimen without shear reinforcement or ties and with a surface that was not intentionally roughened, the shear strength from experimental test results was 76 psi (0.52 MPa), which is about 95% of the 80 psi (0.56 MPa) recommended by ACI 318. It was expected that if the contact surface of the flange was intentionally roughened, the experimental results would yield 80 psi (0.56 MPa) or higher.

When 80 psi \( \leq \nu_{sh} \leq 290 \) psi (0.56 MPa \( \leq \nu_{sh} \leq 2 \) MPa), with intentionally roughened surface, ACT 318-02 recommends using the minimum reinforcement calculated from the following equation:

\[
A_f = \frac{50b_fs}{f_y} \quad \text{(in.}^2 \text{ or mm}^2) \tag{1}
\]

The minimum reinforcement required according to Eq. (1) for shear connectors with \( f_y = 36,000 \) psi (248 MPa), \( b_s = 12 \) in. (305 mm), and spacing \( s = 24 \) in. (610 mm) along the concrete girder yields a horizontal shear reinforcement area \( A_f = 0.40 \text{ in.}^2 \) (258 mm²) per shear pocket. For the specimens tested with shear reinforcement of one bolt \( (A_f = 0.785 \text{ in.}^2 \text{ or mm}^2) \) and two bolts \( (A_f = 1.57 \text{ in.}^2 \text{ or mm}^2) \) and with a surface not intentionally roughened, the predicted shear strengths from the tests were 171 psi and 240 psi (1.2 MPa and 1.65 MPa), respectively, which is within the limit of the code (80 psi \( \leq \nu_{sh} \leq 290 \) psi). For the case of two 1-in.-diameter (25 mm) bolts with \( f_y = 60,000 \) psi (420 MPa) provided per shear pocket, the resulting shear strength is \( 1.57 \times 36 = 56.5 \text{ kip}, \) which is about 18% higher than shear strength provided by four No. 4 bars with \( f_y = 60 \text{ ksi} (414 \text{ MPa}) \).

When 290 psi \( \leq \nu_{sh} \leq 500 \) psi (2 MPa \( \leq \nu_{sh} \leq 3.45 \) MPa),

\[
\nu_{sh} = 260 + 0.6 \left( \frac{A_f f_y}{b_s s} \right) \quad \text{(psi or MPa)} \tag{2}
\]

For the specimens tested with three bolts and the surface not intentionally roughened, the predicted shear stress from test results was 331 psi (2.3 MPa), which is within the limits of the ACI 318 equation (Eq. [2]). Applying Eq. (2) for three bolts \( A_f = 2.355 \text{ in.}^2 \text{ or mm}^2 \) yields a horizontal shear stress of 436 psi (3.0 MPa), which is about 32% higher than the predicted strength from the test results.

**Test Result Comparison with AASHTO Standard Specifications**

AASHTO standard specifications provide the following in Article 9.20.4.3 for the horizontal shear strength of the concrete girders and the precast concrete slabs. The area of the bolts used was 0.785 in.² (506 mm²) with a yield strength and an ultimate strength of 36,000 psi (248 MPa) and 58,000 psi (400 MPa), respectively. The contact surface area has a width of 12 in. (305 mm) and a length of 24 in. (610 mm). The shear connector blockouts were spaced at 2 ft (610 mm) on center where possible.

When \( \nu_{sh} \leq 80 \) psi \( (\nu_{sh} \leq 0.56 \text{ MPa}) \), with intentionally roughened surface, no shear reinforcement is required.

Once more, for the tested specimen without shear reinforcement or ties and with a surface that was not intentionally roughened, the shear stress from the experimental test result was 76 psi (0.52 MPa), which is about 95% of the 80 psi (0.56 MPa), according to AASHTO standard specifications. It was expected that if the contact surface of the flange was intentionally roughened, the experimental results would yield 80 psi (0.56 MPa) or higher.

When 80 psi \( \leq \nu_{sh} \leq 350 \) psi (0.56 MPa \( \leq \nu_{sh} \leq 2.41 \) MPa) with intentionally roughened surface, use the minimum reinforcement calculated from the following equation:

\[
A_f = \frac{50b_fs}{f_y} \quad \text{(in.}^2 \text{ or mm}^2) \tag{3}
\]
The minimum reinforcement required according to Eq. (3) for shear connectors with \( f_y = 36,000 \text{ psi} \) (248 MPa), \( b = 12 \text{ in.} \) (305 mm), and \( s = 24 \text{ in.} \) (610 mm) along the concrete girder yields a horizontal shear reinforcement area \( A_y = 0.40 \text{ in.}^2 \) (258 mm\(^2\)) per shear pocket. For the specimens tested with shear reinforcement of one bolt \( (A_y = 0.785 \text{ in.}^2 \) [506 mm\(^2\)]), two bolts \( (A_y = 1.57 \text{ in.}^2 \) [1013 mm\(^2\)]), and three bolts \( (A_y = 2.355 \text{ in.}^2 \) [1519 mm\(^2\)]) and the surface not intentionally roughened, the predicted shear stresses from the tests were 171 psi, 240 psi, and 331 psi \( (1.2 \text{ MPa}, 1.65 \text{ MPa}, \text{ and } 2.3 \text{ MPa}) \), respectively. The clamping stress for the specimens tested with shear reinforcement of one bolt, two bolts, and three bolts were 98 psi, 196 psi, and 294 psi \( (0.67 \text{ MPa}, 1.35 \text{ MPa}, \text{ and } 2.0 \text{ MPa}) \), respectively.

Of all the code equations, the AASHTO LRFD equation yields the most conservative values for a clamping stress.\(^1\) AASHTO LRFD uses a linear relationship based on shear-friction theory. From the test results, the AASHTO LRFD can be used to design steel bolts as shear connectors for the full-depth precast concrete panels installed on precast, prestressed concrete girders.

**CONCLUSIONS**

A total of 13 push-out tests were conducted to observe the behavior and horizontal shear strength capacity of threaded steel bolts as shear connectors between concrete I-girders and precast concrete slabs under monotonic static loading. The following conclusions can be drawn from the collected data and observations:

- Steel bolts can provide the necessary ultimate horizontal shear strength at the interface of precast concrete girders and full-depth precast concrete slab bridge construction to achieve full composite action.
- There is a nearly proportional increase in the ultimate horizontal shear strength capacity as the number of steel bolts in a shear pocket increases.
- Different geometric orientations and configurations for a specific number of bolts have no significant effect on the horizontal shear strength. Different lengths of bolts in a shear pocket have a minimal influence on the horizontal shear strength.
- Failure for all the specimens was accompanied by a loud noise as the bond at the interface between the precast concrete girder and the haunch failed and was followed by flexural yielding of the steel bolts, which avoided the abrupt failure of the connection.
- The proposed shear connection system can be designed conservatively with the use of the AASHTO LRFD Section 5.8.4.1 Equation 5.8.4.1-1 for horizontal shear strength.
- The steel bolts can be used as shear connectors in full-depth precast concrete slabs installed on precast, prestressed concrete girders to achieve full composite action.
- Good agreement was observed between the FEA and the experimental test results in terms of the ultimate shear strength capacity and failure mode for all specimen types.

**ACKNOWLEDGMENTS**

The experimental work was carried out in the structural laboratory at the Department of Civil and Materials Engineering, University of Illinois at Chicago. The support of Andy Keenan from Prestress Engineering Corp. for the fabrication of the precast concrete segments is appreciated. Special thanks are also due to Master Builders for its assistance.
REFERENCES


CONVERSION FACTORS

1 in. = 25.4 mm
1 ft = 0.3048 m
1 yd³ = 0.765 m³
1 lb = 4.448 N
1 kip = 4.448 kN
1 psi = 0.0069 MPa
1 ksi = 6.895 MPa

APPENDIX: NOTATION

\( A_{cv} \) = area of concrete engaged in shear transfer
\( A_{sf} \) = area of shear reinforcement crossing the shear plane
\( c \) = cohesion stress
\( f_y \) = yield strength of reinforcement
\( P_e \) = permanent net compressive force normal to the shear plane
\( \nu_{sh} \) = nominal shear stress
\( \mu \) = coefficient of friction