Impact of shear connectors on composite action in full-depth precast concrete bridge systems

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- This study examines the impact of shear connectors on composite actions of precast concrete bridge systems.
- A parametric study was conducted to examine the overall effects of pocket spacing and the number of shear studs per pocket on the horizontal shear force in both simply supported bridges and continuous-span bridges.
- The load–deflection curves and slippage values from the nonlinear finite element analyses were recorded and compared with the experimental testing results.

In the past 50 years, after extensive research and practice, installation of full-depth precast concrete deck panels on steel girders is now common. On the other hand, full-depth precast concrete deck panel installation on steel beams has been limited. This paper describes a study to evaluate the performance of bridges constructed with precast concrete deck panels, both existing and newly cast, on steel beams.

Full-depth precast concrete deck panels combine high-strength prestressing tendons with high-performance full-depth precast concrete prefabricated in a controlled environment. The connections between the beams and precast concrete panels are provided by shear studs aligned in shear pockets that provide full composite action. This parametric study examined the effect of pocket spacing and number of shear studs per pocket on the structural response of full-depth precast concrete deck panels. The results indicate that horizontal shear strength is increased in relation to the number of shear pockets in simply supported and continuous-span models. The load–deflection curve and slippage values were compared with experimental testing using a nonlinear finite element analysis (FEA). A limited number of studies have focused on this area, and this study aimed to fill in gaps in the understanding about the performance of full-depth precast concrete deck panels.

This paper discusses the key parameters that affect the FEA modeling of a precast concrete deck panel on the shear connectors and concrete, including material models, element models, and contact elements, in addition to loading and boundary conditions. The study compared predictions from analyses carried out on the precast concrete deck panel with results obtained from a similar recommended method in terms of stress distribution in the connection and interface capacity of connection. A parametric study was conducted to investigate the influence of shear connectors on connector size, grade, and general compressive strength of concrete on the precast concrete deck panel performance.

The concrete girders for the continuous connectors have been extensively studied. The study published by Hanson[1] in 1960 was the basis for the original design provisions used in the American Concrete Institute's *Building Code Requirements for Structural Concrete (ACI 318-08) and Commentary (ACI 318R-08)*.[2] Grossfield and Birnstiel[3] noted that greater shear strengths would result if bigger slips were allowed. In the framework of providing an equation to predict horizontal shear strength, Mattock[4] authored a paper in the 1970s proposing an equation for horizontal shear strength. Loov and Patnaik[5] tested 16 samples of composite beams and further analyzed the horizontal shear strength. Developing composite action between steel girders and precast concrete deck panels requires knowledge of the details of the panel-to-girder connection, as analyzed by Issa et al.[6] Kahn and Slapkus[7] tested six composite concrete T beams to investigate the interface shear strength for sections made with high-strength concrete. Issa et al.[8] compared experimental findings with the horizontal shear strength predicted using the American Association of State Highway Officials' (AASHO's) *Standard Specifications for Highway Bridges*[9] regarding interface shear transfer between bridge surfaces and the supporting steel. To evaluate the constructibility and behavior of a precast concrete bridge deck system, Issa's team conducted a series of tests on a full-scale prototype of a continuous two-span bridge. The bridge was 82 ft (25 m) long and 18 ft (5.5 m) wide and was supported by three wide flange girders (W18x86). There were 11 precast concrete deck panels in the deck constructed with conventional mild steel. At the central support, the deck was post-tensioned to an approximate stress of 500 psi (3.4 MPa). Hatami[10] developed design equations for the discrete shear connectors found in precast concrete bridge deck systems. Xu et al.[11] concluded that for concrete slab prefabrication in steel–concrete composite bridges, clustering stud shear connectors with a narrow spacing to form the group studs was beneficial to the bridge. Hosseini et al.[12] analyzed the performance of bolted shear connectors embedded in concrete slabs subjected to static and fatigue loading. These studies were comprehensive, and their experimental findings and equations have been used as the basis for new provisions on interface shear in the ninth edition of the *AASHTO LRFD Bridge Design Specifications*.[13]

For general industry practices, the design attributes of precast concrete deck panels can be summarized as the following:

- The spacing between connectors is equal to 48 in. (1219.2 mm).
- The increase in the number of shear studs increased the ultimate strength.
- Few studies have considered the effects of pocket spacing on the system, but the typical spacing is considered to be 24 in. (609.6 mm).
- Current design provisions do not address the number and designation of shear connectors and pockets used in precast concrete bridge deck panel systems.

The main objective of the present study was to identify design procedures for shear pockets that have shear stud connectors within a full-depth precast concrete deck system, with a focus on interpreting the behavior of the shear pocket connection through modeling and FEA. This study also reviewed the effects of some relevant characteristics, such as the effect of compressive strength of concrete and the general connection on the attitude of the shear pocket dimensions.

The experimental tests validated the finite-element simulation according to a calibration process.

This study also investigated the effect of slippage on ultimate resistance and deflection. To connect two structural elements, a type of bolted structural steel connection known as a slip-critical joint depends on the friction between two connected elements. Within a composite beam, the concrete slab is connected to the steel section and associated with carrying the load. Slip between the slab and the steel section is then prevented and the connection holds out against a longitudinal shear force similar in distribution to the vertical shear force. Composite action activates when there is no slippage between the slab and upper flange. Steel shear connectors are used in modern bridges to extend the load limits permitted for composite activity. The bond between concrete and steel on the contact surface is often enough to bear the shear strengths induced by dead and live loads. Subsequently, within the FEA models for this study, the bridge systems were assumed to be composite structures.

Maximum spacing and stud spacing limits in design specifications

The current AASHTO maximum spacing requirement of 24 in. (609.6 mm) first appeared in 1944 in the fourth edition of the *Standard Specifications for Highway Bridges*.[9] At the University of Illinois Chicago, a composite design study for the Public Roads Administration was carried out. The investigators performed quarter-scale model tests on composite bridges and recommended a maximum spacing of three to four times the slab depth because of possible separation between the slab and the steel girders. The sample bridge had a slab thickness of 7 in. (177.8 mm), so the model studies used a 24 in. maximum spacing requirement, which corresponds to 3.5 times the slab thickness.

In the second edition of the AASHTO LRFD specifications,[14] the maximum allowed center-to-center spacing between shear connectors is 24 in. (609.6 mm). The minimum spacing needed in the longitudinal direction is 6*d* (where *d* is the diameter of the shear stud), whereas the minimum spacing in the transverse direction to the longitudinal axis is 4*d*. Both the American Institute of Steel Construction (AISC)[15] and AASHTO steel building specifications have similar minimum spacing requirements. For AISC, the maximum spacing is 36 in. (914.4 mm), or eight times the slab thickness. The minimum center-to-center stud spacing requirements in Eurocode 4[16] are 5*d* and 2.5*d* for longitudinal and transverse directions, respectively, and they are smaller than the previously noted requirements in U.S. practice.

The shear friction concept used in ACI 318-08[2] is based on the force transfer across an existing or anticipated crack. Strength and behavior of initially uncracked specimens differ from those with preexisting cracks. The shear key side points permit the two interfacing parts to slide over one another, and the connecting bar then begins to engage within the mechanism. Once sliding happens, the two surfaces separate from each other, giving rise to tensile and shear stress within the steel connector.

Table 1 shows the cohesion and friction factors in a composite structure specified in the fifth edition of the AASHTO LRFD specifications.[17]

Shear connectors

The shear force at the interface between the deck slab and steel section is carried by the shear connectors. In composite deck bridges, the shear connectors are provided along the length of the bridge span in positive and negative moment regions to fulfill composite action requirement, as was used in this design example. The shear connectors must permit a thorough compaction of the concrete to ensure that their entire surfaces are in contact with the concrete. Moreover, the shear connectors must be capable of resisting both horizontal and vertical movements between concrete and steel.

Design methodology for the shear connectors

Because the steel girder has been designed as a composite section, shear connectors should be supplied at the crossing point between a deck slab and the steel part to withstand the interface of the shear. For consistent composite bridges, shear connectors are usually provided throughout the bridge length. In the negative flexure section, shear connectors must be provided because longitudinal strengthening is intended to be a section of the composite region.

The number of studs per pocket is equal to the total number of studs divided by the total number of pockets along each beam. The center-to-center shear pocket spacing should be between a minimum of 18 in. (457.2 mm) and a maximum of 24 in. (609.6 mm). The eighth edition of the AASHTO LRFD specifications[18] provides the following equations in articles 5.7.4.2 and 5.7.4.3 for horizontal shear resistance.

$$A_{vf} = 0.05\frac{A_{cv}}{f_y} \qquad \text{(AASHTO 5.7.4.2-1)}$$

$$\upsilon_{nh} = cA_{cv} + \mu(A_{vf}f_y + P_c) \qquad \text{(AASHTO 5.7.4.3-3)}$$

where

A_{vf} = area of shear reinforcement crossing shear plane

A_{cv} = area of concrete engaged in shear transfer

f_y = yield strength of reinforcement

υ_{nh} = nominal horizontal shear resistance

c = cohesion factor

μ = friction factor

P_c = permanent net compressive force normal to shear plane

The calculated nominal horizontal shear strengths for experimental test specimens without shear reinforcement and with

Table 1. *AASHTO LRFD Bridge Design Specifications* 5.8.4.3: Cohesion and friction factors

Type of structure	c, ksi	μ	K_1	K_2, ksi
Cast-in-place concrete slab on clean concrete girder surfaces, free of laitance with surface roughened to an amplitude of 0.25 in.	0.28	1.0	0.3	Normalweight concrete: 1.8 Lightweight concrete: 1.3
Normalweight concrete placed monolithically	0.40	1.4	0.25	1.5
Lightweight concrete placed monolithically or not monolithically	0.24	1.0	0.25	1.0
Normalweight concrete placed against a clean concrete surface	0.24	1.0	0.25	1.5
Concrete placed against a clean concrete surface, free of laitance but not intentionally roughened	0.075	0.6	0.2	0.8

Source: Data from AASHTO (2010).

Note: Use c = 0.075 and μ = 0.6 for concrete placed against a clean concrete surface. AASHTO = American Association of State Highway and Transportation Officials; c = cohesion factor; K_1 = fraction of concrete strength; K_2 = limiting interface shear resistance; μ = friction factor. 1 in. = 25.4 mm; 1 ksi = 6.895 MPa.

three studs, six studs, and nine studs were 72.1, 145, and 196 kip (320.7, 645, and 871.8 kN), respectively.

To investigate the influence of the yield strength and concrete strength as well as shear size and connector size on the interface shear capacity of shear connectors in the precast concrete deck panels, different stud numbers were applied to the model with identical cross section and side views for the parametric studies summarized in **Table 2**. Parametric studies were conducted using the AASHTO Eq. (5.7.4.3-3)[18] and the FEA for the evaluation of the effect of shear connector sizes as well as number of shear studs for distances of 24 and 48 in. (609.6 and 1219.2 mm) between shear pockets, as follows:

- bonded no-slip with a space of 24 in. (609.6 mm) between pockets
- three shear studs with a space of 24 in. (609.6 mm) between shear pockets
- three shear studs with a space of 48 in. (1219.2 mm) between shear pockets
- six shear studs with a space of 48 in. (1219.2 mm) between shear pockets
- nine shear studs with a space of 48 in. (1219.2 mm) between shear pockets

Table 2. Shear stud models

Designation	Number of studs	Size of shear pocket length × width, in.	Spacing between pockets, in.
Model 1	Full bond	11 × 6	24
Model 2	3	11 × 6	24
Model 3	3	11 × 6	48
Model 4	6	11 × 6.5	48
Model 5	9	11 × 10	48

Note: 1 in. = 25.4 mm.

The shear connector, shear connector diameter, and pocket size are given after the shear connectors are designed using an interface shear resistance approach. The format of the shear connectors inside the shear pocket can be decided based on girder type and flange width. **Figure 1** shows a shear connector–shear pocket configuration with a considerable number of shear studs. In addition, Table 2 presents the models and the sizes of the shear pockets used in the support analysis.

Finite element analysis

Model setup

Standard elements as defined in the Ansys software were used in creating the model and are further explained in the model properties section. Precast concrete was modeled using the SOLID65 element, which has three translational degrees of freedom at each node as well as the capabilities of cracking and crushing. The element has up to three reinforcement materials in three directions and one solid material. Therefore,

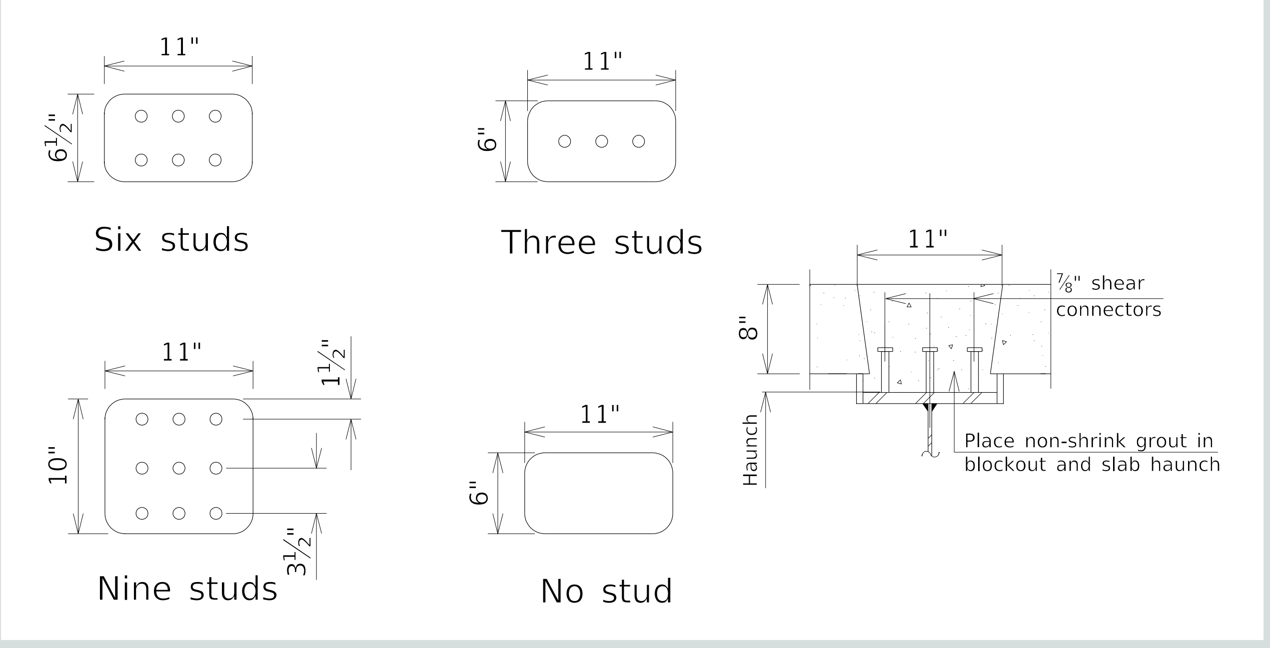

Figure 1. Cross sections showing shear stud configurations. Note: 1" = 1 in. = 25.4 mm.

this component is commonly used to suit nonlinear material properties. Fanning[19] used Ansys software to model prestressed concrete systems incorporating physical experiments for a 29.5 ft (9 m) long prestressed concrete T beam and ordinary reinforced 9.8 ft (3 m) long concrete beam.

The target element TARGET170 and the contact element CONTACT175 were used to model the contact between the precast concrete and the grout surface. In a similar manner, the target and the contact elements were used to model the connection between the steel flange of beams and the grout surface.

The contact and target elements in Ansys were used to simulate the stiffness, separation, and bonding between the grout and the concrete at the joint of the section. The CONTACT175 element parameters included the following:

- standard contact surface behavior
- friction coefficient of 0.6 for cement grout
- cohesion value c_1 for contact cohesion between cement grout and flange of a steel beam of 0.025 ksi
- cohesion value c_2 for contact cohesion between the precast concrete beam and cement grout of 0.075 ksi
- initial penetration of 0
- closed gap for automatic contact adjustment
- selection of asymmetric contact

Figure 2 shows the relationship between the idealized stress and strain of the cement grout.

In addition, shear stud connectors were anticipated to be elastic-plastic materials having a yield strength f_y of 36 ksi (248.2 MPa), modulus of elasticity E of 29,000 ksi (199,955 MPa), and Poisson's ratio v of 0.3. The element BEAM188 was used to model the shear stud connector because it is appropriate for slender to moderately stubby and thick beam structures. Moreover, the BEAM188 elements were based on the Timoshenko Beam Theory,[20] which consists of shear-deformation effects. Each element consisted of six or seven degrees of freedom.

The yield strength of the post-tensioning bars was 121 ksi (834.2 MPa). The ultimate tensile strength of the post-tensioning bars was 156 ksi (1075.5 MPa). The post-tensioning bars had a cross-sectional area of 1.58 in.^{2} (1019 mm^2), a modulus of elasticity E of 30,000 ksi (206,850 MPa), a Poisson's ratio ν of 0.3, and 3.1% relaxation. The post-tensioning was modeled using the LINK8 element having a 0.00331 initial strain.

A full-depth precast concrete panel tested by Issa et al.[8] was used to validate the FEA modeling. That bridge system had two spans and one lane; the dimensions were 18 × 82 ft (5.5 × 24 m), with two spans of 40 ft (12 m) each. The bridge consisted of 11 prefabricated full-depth precast concrete panels. The supporting mechanism had three W18x86 steel beams with shear connectors and shear studs to secure the connection. The panels were 18 ft long and 8 in. (200 mm) thick; the development length of steel bars extending from every panel and connection of joints can be handled by lon-

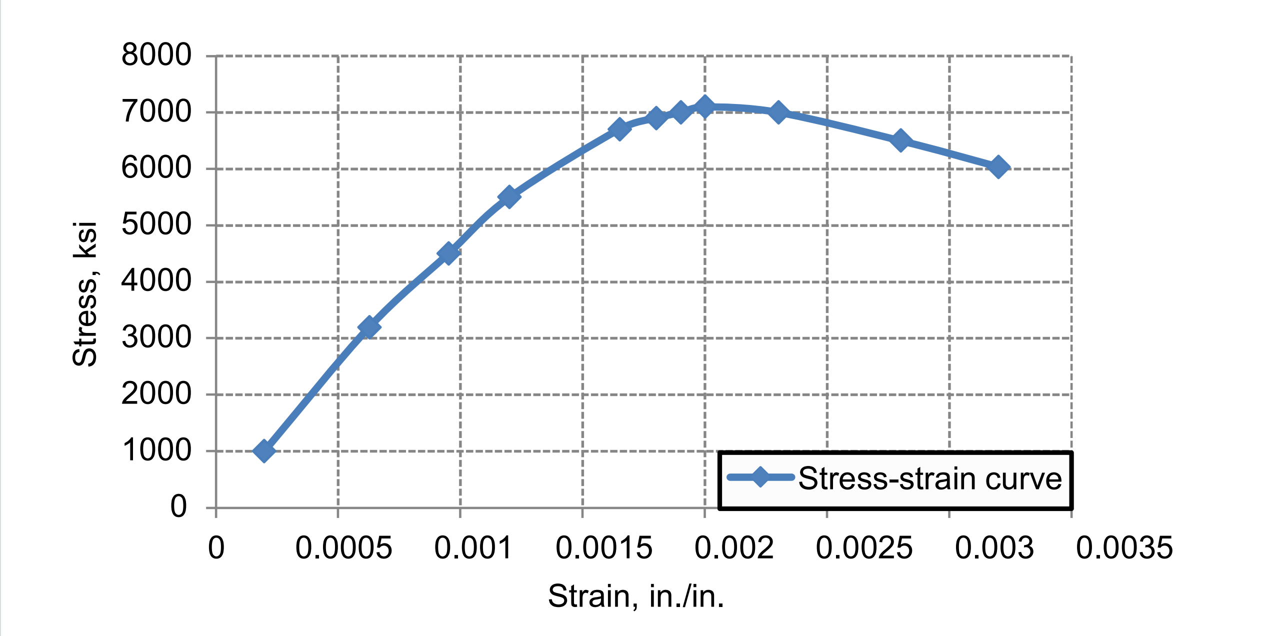

Figure 2. Idealized stress-strain relationship of cement grout used for finite element analysis. Note: 1 in. = 25.4 mm; 1 ksi = 6.895 MPa.

gitudinal joints between the staged construction with enough width, which is important for the flow of traffic. The two 4 ft × 9.5 in. (1.2 m × 241.3 mm) end panels (P1) were identical, whereas the nine 8 ft (2.44 m) middle panels (P2, P3, P4, P5, and P6) referred to in **Fig. 3** had different configurations due to the post-tensioning requirements and sequence of construction.

To find the optimum mesh density, a sensitivity study was conducted. The meshing density increased around the critical locations (**Fig. 4**) where the perfect bond between the concrete and the steel reinforcement was anticipated. The total applied load was distributed into small load increments, and the modified Newton-Raphson equilibrium iteration technique was used to examine the convergence at the end of every load increment for a tolerance of 0.001. The boundary conditions were applied to the full models of the surfaces. Three types of loading were used: a service load of 92 kip (410 kN), an overload test with 184 kip (818 kN), and an ultimate load test with 553 kip (2460 kN).

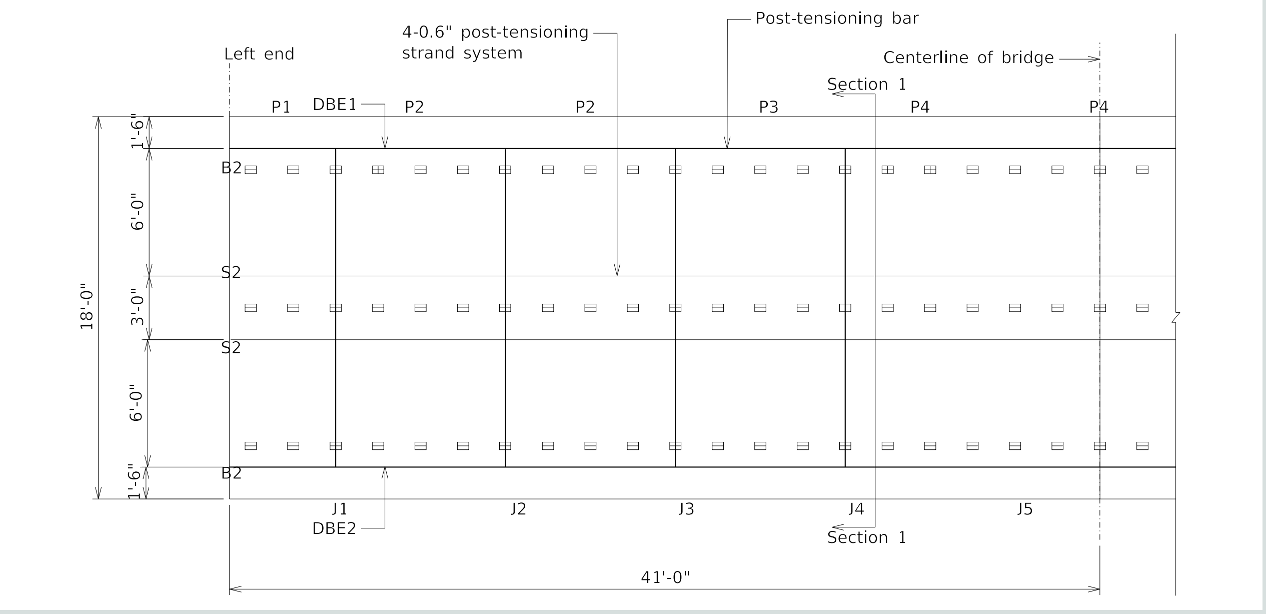

Figure 3. Arrangement of full-depth precast concrete bridge deck system components. Note: B2 = end of post-tensioning bar; DBE1 = north post-tensioning bars with length of 82 ft.; DBE2 = south post-tensioning bars with length of 82 ft.; J1 = joint 1; J2 = joint 2; J3 = joint 3; J4 = joint 4; J5 = joint 5; P1 = panel 1; P2 = panel 2; P3 = panel 3; P4 = panel 4; S2 = end of post-tensioning strand. 1" = 1 in. = 25.4 mm; 1' = 1 ft = 0.305 m.

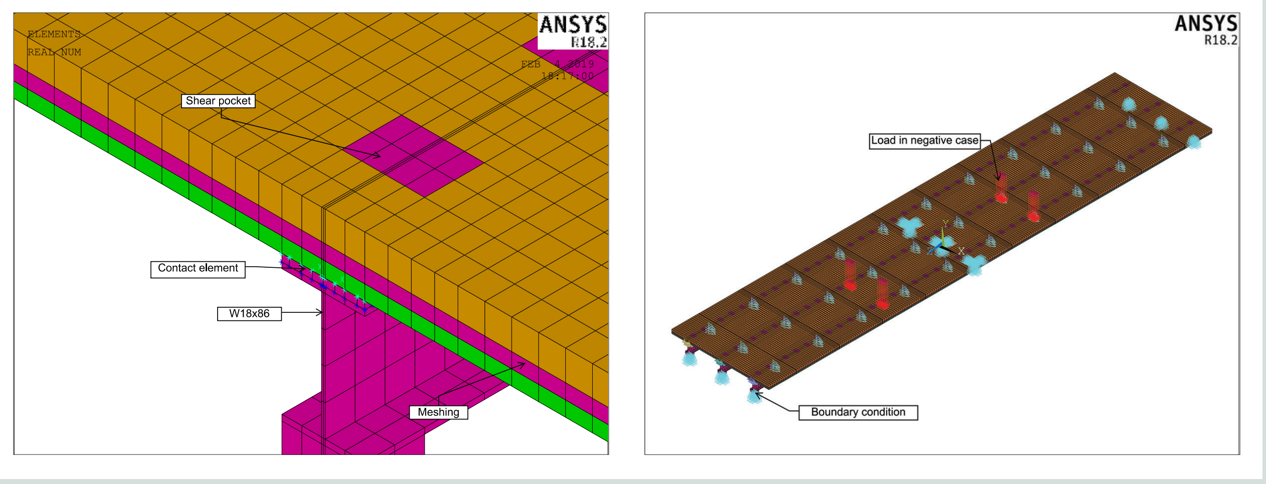

Figure 4. The geometry and connector configuration of a continuous span with three shear studs for 24 in. spacing. Note: 1 in. = 25.4 mm.

In the first span and over the interior central support, a maximum positive moment and a maximum negative moment were noticed as a result of the truck loading. AASHTO HS20 truck loading was simulated using an equivalent two-axle vehicle loading of 6 ft (1.83 m) wide with a distance of 17 ft (5.18 m) between the two axles (Fig. 4). A maximum load value of 92 kip (410 kN), approximately representing the AASHTO truck service load plus 30% impact, was applied in this case; for HL-93, the truck service load is 119.77 kip (532.7 kN).

The model included the deck, steel girder, post-tensioned bar and strand, plate, shear connectors, haunch, and pocket (Fig. 4).

Model properties

The input of material properties was assigned to model elements. The material characteristics were input to represent concrete, steel, and the grout used in the prototype. Each type of element can reinforce particular aspects of the material. The elements used in the full-scale study included SOLID65, LINK8, SOLID45, TARGET170, CONTACT175, and BEAM188.

SOLID65 is used in grout and concrete in the FEA model. It is a three-dimensional (3-D) solid element and has crushing and cracking options. The SOLID65 element is defined by eight nodes consisting of three degrees of freedom for each node. The entrenched reinforcement can be improved by the introduction of a support ratio into SOLID65. The real constant input describes the support ratio as a volumetric ratio linked to a group of material properties and two distinctive orientations.

The properties of the materials can be linked to element types. In this study, the concrete compressive strength f_c' was set to 7100 psi (48.9 MPa). Shear transfer coefficients were added for both closed and open cracks. The crack coefficients ranged from 0 to 1. A crack coefficient of 0 shows a smooth crack minus shear transfer, whereas a crack coefficient of 1 represents a rough crack consisting of shear transfer.

The grout compressive strength f_g' was taken to be 5000 psi (34.4 MPa). The Poisson ratio of the grout v was taken as 0.2. Furthermore, the coefficients of shear transfer were the same as the coefficients of concrete elements. The Ansys program also used the key options to define the features of the forms of elements. Two types of critical options were applied for the SOLID65 elements to model the grout and the concrete. KEYOPT(7) and KEYOPT(3) were used to assist in model convergence. KEYOPT(3) is associated with the characteristics of completely crushed unsupported elements; it was assigned a value of 2 to reduce mass and applied load and, in turn, introduce a steady Newton-Raphson load vector.

In contrast, a value of 1 was assigned to KEYOPT(7) to represent tensile stress relaxation after cracking. For example, when an element is reduced to zero, that element receives enough tensile stresses to result in a crack and the element may experience difficulties in converging. KEYOPT(7) enables stress to decrease after crack progress and, as a result, permits elements to converge quickly.

A two-node, 3-D spar element characterizes the LINK8 element. Each of its nodes consists of three degrees of freedom. It only assesses compression and tension forces. It encompasses large deflection abilities, stress stiffening, swelling, and creep. The LINK8 element was applied in the modeling curve strand. It used two inputs of real constants to describe the strand. In the Ansys program, LINK8 elements are produced on the lines of transverse volumes. Usually, the volumes are joined so that they can function as a one solid concrete.

The lines denoting the curved strand are also joined to show one solid strand. The strand in the model is joined to the concrete. In addition, it works in tandem with the concrete. It was established that the laboratory units lacked strand in unswerving contact with the concrete. As a result, they generated primary stress in concrete. This happened because the strand only passed through the conduit. Again, the strand only exerts a force on the concrete due to the bearing plates at each end of the strand. This can make FEMs stiffer than actual test units.

The SOLID45 element was used in bearing plates and comprised eight nodes, each with three degrees of freedom. The SOLID45 element reinforces the features existing in the steel. In addition, it does not require any real constant inputs. The FEM represents bearing plates using the SOLID45 element and involves stress control for reaction bearing and during application of loads. Normally, it is advisable to introduce loads to bearing plates in the Ansys program to enable accurate modeling of the load stresses. Furthermore, applying loads directly to the concrete element may increase the difficulty in the convergence of the model.

The target and contact elements are applied to trigger separation, stiffness, and bonding between grout and concrete at the joint of the section. Pairs of contacts include a contact element and a target element, which share the same real constant.

TARGET170 was used as the target element. The target elements overlay the already created solid elements, such as the element SOLID65 in this research. The TARGET170 elements consist of more-rigid material than the contact pair. Hence, in the current model, SOLID65 elements characterize grout that was covered with TARGET170. In the contact pair, the contact element (CONTACT175) was used. It covered SOLID65 elements representing the concrete. CONTACT175 has the capability of modeling the surface. In addition, CONTACT175 can model sliding that occurs between solid 3-D elements.

The FEA strategies for the connection of precast concrete deck panels were incorporated into the material models, boundary conditions, contact elements, shear connector element models, loading, and concrete. The slip was measured and recorded for each load increment and plotted versus the corresponding load; all models, including the flanges of the precast concrete girder segment, were not intentionally roughened. The strength of the frictional bond between the haunch and the concrete surfaces was shown in this test. This test therefore permitted investigation of the contribution of the bond of the material of haunch grout to the strength of the connection system.

For the various numbers of shear studs represented in the load-deflection relationship (**Fig. 5**), the model was assessed with the full bond in the first model with 24 in. (609.6 mm) spacing between pockets, no shear stud, and a service load of 119.77 kip (532.7 kN). The findings show that slippage was zero and deflection was 0.1793 in. (4.5 mm). In the second model, when the three shear studs with a distance of 48 in. (1219.2 mm) between the pockets were used, deflection was 0.396 in. (10.1 mm), the slippage value was 0.0023 in. (0.058 mm), the load was 119.7 kip (532.6 kN), and failure occurred at 459.13 kip (2042.31 kN). When six shear studs with a distance of 48 in. between the pockets were used, the slippage value was 0.003 in. (0.0762 mm) under the same service load of 119.7 kip and deflection was 0.421 in. (10.7 mm). Failure occurred at 499.5 kip (2221.8 kN). Finally, for nine shear studs with a distance of 48 in. between the pockets, deflection was 0.41 in. (10.4 mm), slippage was 0.00185 in. (0.047 mm), and the in-service load was 119.7 kip. **Figure 6** depicts the general relationship between deflection and maximum sliding. The significant behavior for nine studs with a distance of 48 in. and failure at 659 kip (2931.3 kN) occurred when the separation happened. Because the bond at the interface between the precast concrete girder and the haunch was unsuccessful and was followed by flexural yielding of the steel bolts, which avoided the abrupt failure of association, failure was observed in all models.

Figure 7 shows model 5 with nine shear studs with a distance between shear pockets of 48 in. (1219.2 mm) and model 2 with three shear studs with a distance between shear pockets of 24 in. (609.6 mm).

Model geometry, boundary conditions, and loading

The model considered the shear connectors, pocket, and haunch; the bond between the concrete and shear connectors; and the concrete haunch with pocket. The FEA was carried out for a vertical structural load applied at the surface of the bridge. This load was a combination of the truck load and the lane load applied step by step until the final failure of the connection occurred.

The value of the load applied for truck HL-93 (AASHTO[2]) was 119.7 kip (532.4 kN), and the overload was 240 kip (1067.5 kN). For a two-time truck load, the ultimate load was 924 kip (4110 kN), a 7.7-time service truck loading.

In the simply supported case, there was no negative moment in the analysis of moment because the two post-tensioning bars were removed from the model (**Fig. 3** and **8**).

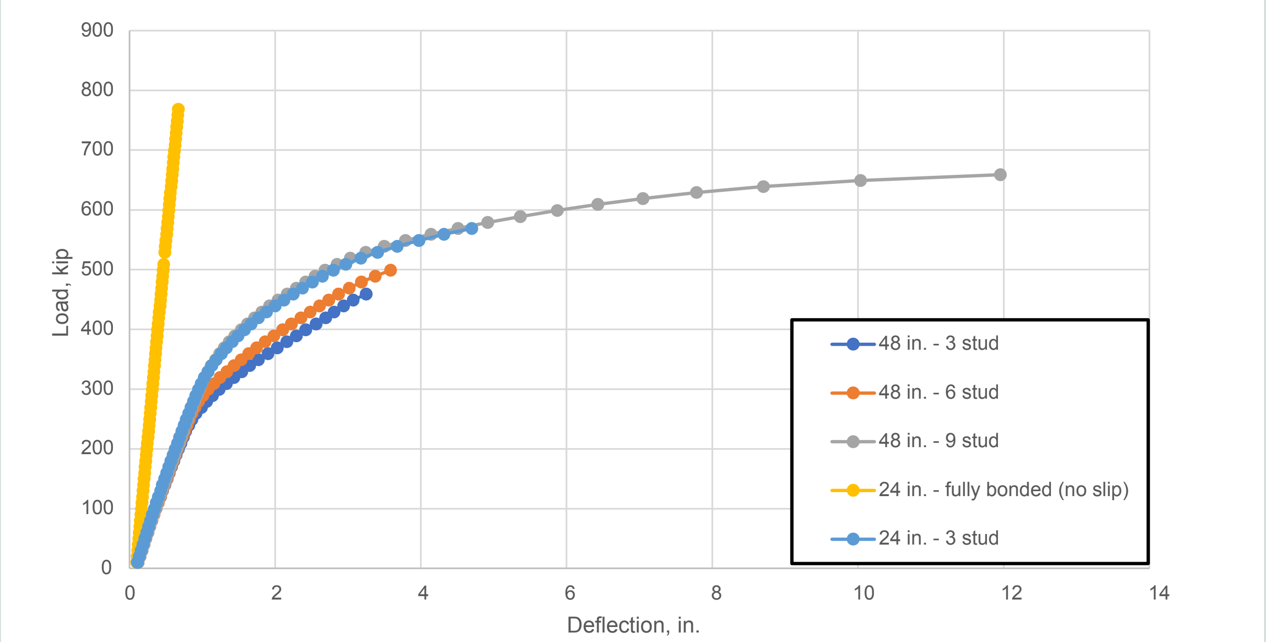

Figure 5. Load–deflection curves for various numbers of studs at spacings ranging from 2 to 4 ft. Note: 1 in. = 25.4 mm; 1 ft = 0.305 m; 1 kip = 4.448 kN.

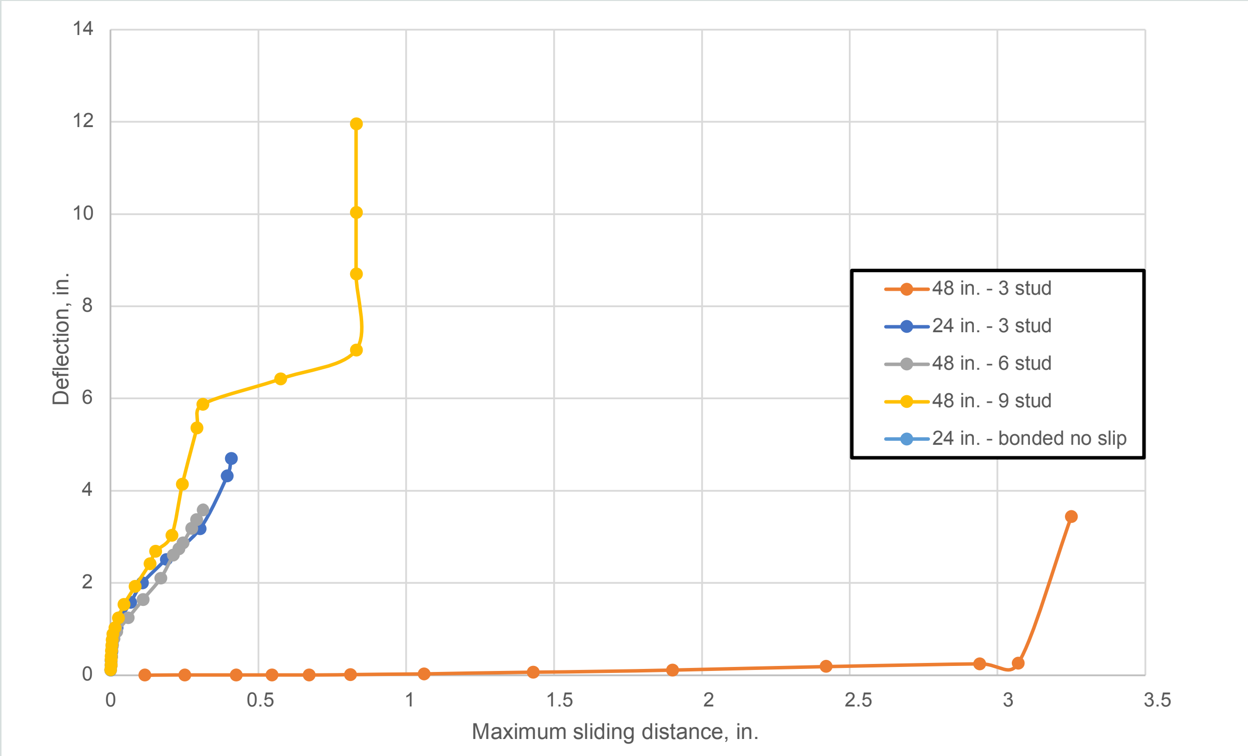

Figure 6. Deflection–maximum sliding distance curve for various numbers of studs at spacings ranging from 2 to 4 ft. Note: 1 in. = 25.4 mm; 1 ft = 0.305 m.

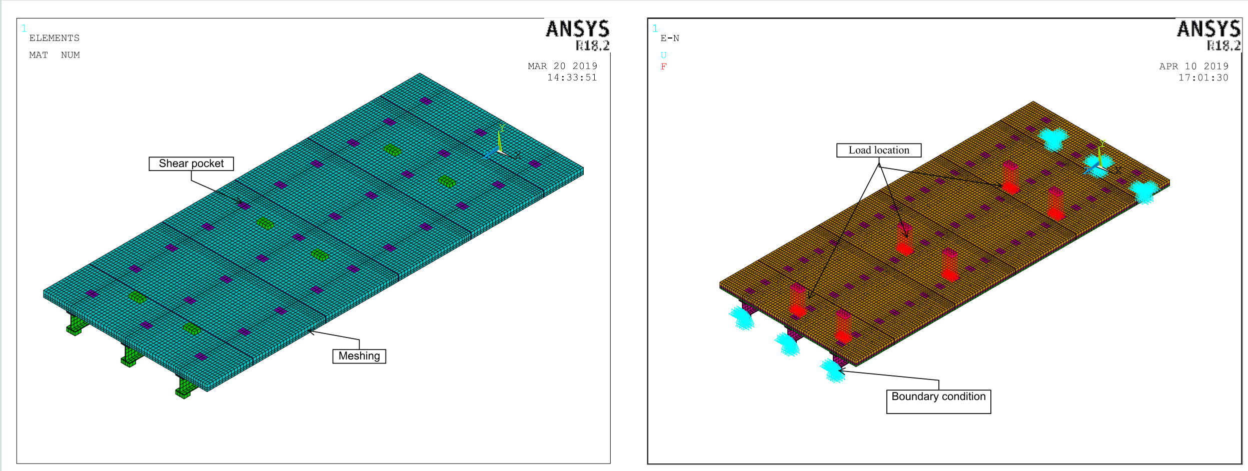

Figure 7. Geometry and connector configuration for the finite element analysis. Note: 1 in. = 25.4 mm.

FEA results

Observations were made throughout the FEA models to understand the distortions and modes of failure of the various models. Various correlations and comparisons of the outcomes were made to examine the efficiency of shear stud connectors. The load used and the corresponding slip between the deck and the supporting system were recorded for each load increment. Each specimen demonstrated a comparable tendency in behavior throughout the testing process. When the applied load approached the range of 500 to 800 kip (2224 to 3558.5 kN), failure occurred due to full separation of the haunch from the basic structure of the model. For most of the tested models, the load at this point was considered to be

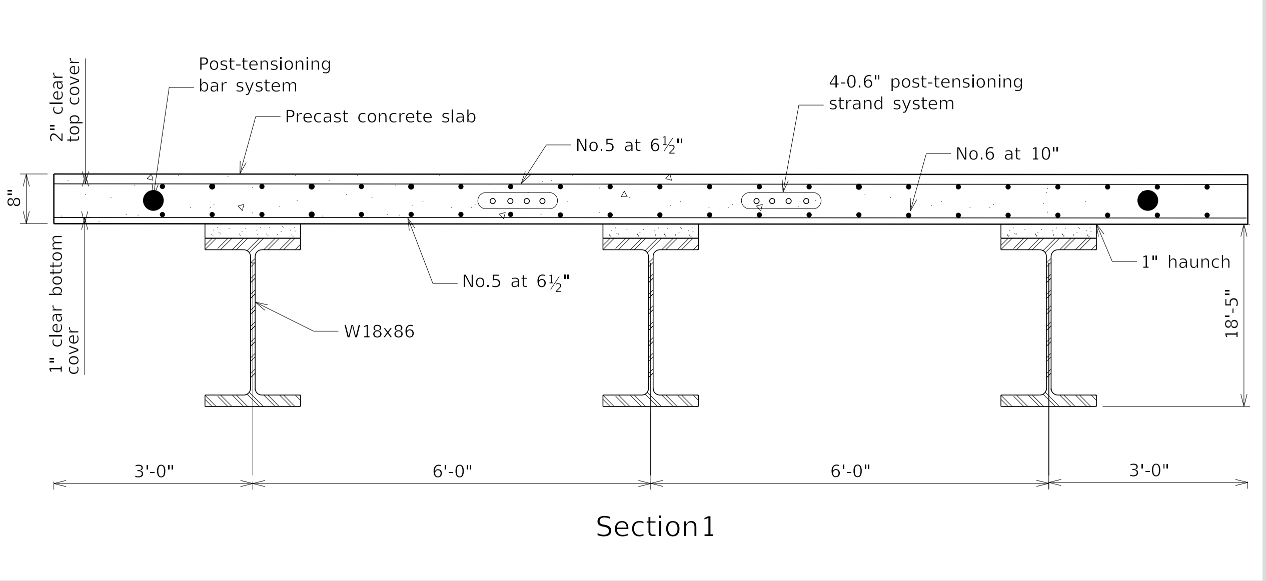

Figure 8. Cross section showing deck reinforcement details. Note: no. 5 = 16M; no. 6 = 19M; 1" = 1 in. = 25.4 mm; 1' = 1 ft = 0.305 m.

the service load. Afterward, the studs at the junction carried the applied load. At this point, the studs revealed that their flexural bending tension and yielding features obstructed the shear connection from slipping at more significant loading proportions.

The measurement of slip was determined and recorded for every load addition and plotted versus the corresponding load. Neither laboratory specimens nor FEA projections of the precast concrete girder section were deliberately roughened. The model without studs demonstrated a curve with nearly no slip. The analyses simply illustrated the strength of the hypothetical bond between the backside and the concrete surfaces. This analysis permitted investigation of the contribution of the bond of the haunch grout material to the connection of system strength. Entire composite action was apparent throughout testing of the initial bridge model because no slippage occurred at the concrete-steel interface, which was between the steel beams and the concrete panels at the haunches. Entire composite action was apparent in the bridge model because no slippage happened at the concrete-steel interface until the load attained roughly 100 kip (445 kN). When the load achieved is 100 kip, the comparison showed a good correlation between the relationship predicted using FEA and the experimentally measured load-displacement relationship. It confirmed the efficiency of using the FEA tool to perform parametric studies for further examinations. The parametric study was conducted to investigate the effect of the number of studs and shear pocket dimensions on shear pocket connection strength. The best model used nine shear studs and 48 in. (1219.2 mm) spacing, which indicates that the spacing between shear pockets has a greater effect than the number of studs. **Figure 9** shows the failure modes in models 4 and 5 when the load was 659 kip (2931 kN).

Comparison of FEA results with experimental test results

Negative flexure case

Figure 10 presents results of measured slippage at various stages of loading for the composite member in the flexural test. The FEM curves show that slippage at the interface of the deflection was 0.095 in. (2.413 mm), and slippage was 0.0085 in. (0.2 mm) for a service load of 93.75 kip (417 kN) in a noncomposite girder. Cracks were evident at a load of 600 kip (2668.9 kN) at the interface between the girder and the trough in the test, and there was an indication of noncomposite action failure due to slippage observed at the maximum load applied. In the experimental test under the service load of the maximum negative moment load test of 93 kip (413.6 kN), the load–deflection curve was linear for the middle beam at 14 ft (4.2 m) from the central supports. The maximum deflection was 0.08 in. (2 mm), and a maximum slippage of 0.007 in. (0.18 mm) was recorded (Fig. 10).

For the bonded composite member in the FEM, the deflection was 0.093 in. (2.36 mm) with a 93 kip (413.6 kN) service load, and no slippage was recorded. It was observed that when the applied service load approached 600 kip, failure occurred because the backside was completely separated from the model and, in most of the tested models, the load at this stage was assumed to be the service load. Later, the applied load was carried at bolts at the interface. For truckload HL-93 under a service load of 119.75 kip (532.6 kN), the deflection was 0.145 in. (3.7 mm) and the slippage was 0.0091 in. (0.23 mm) (Fig. 10).

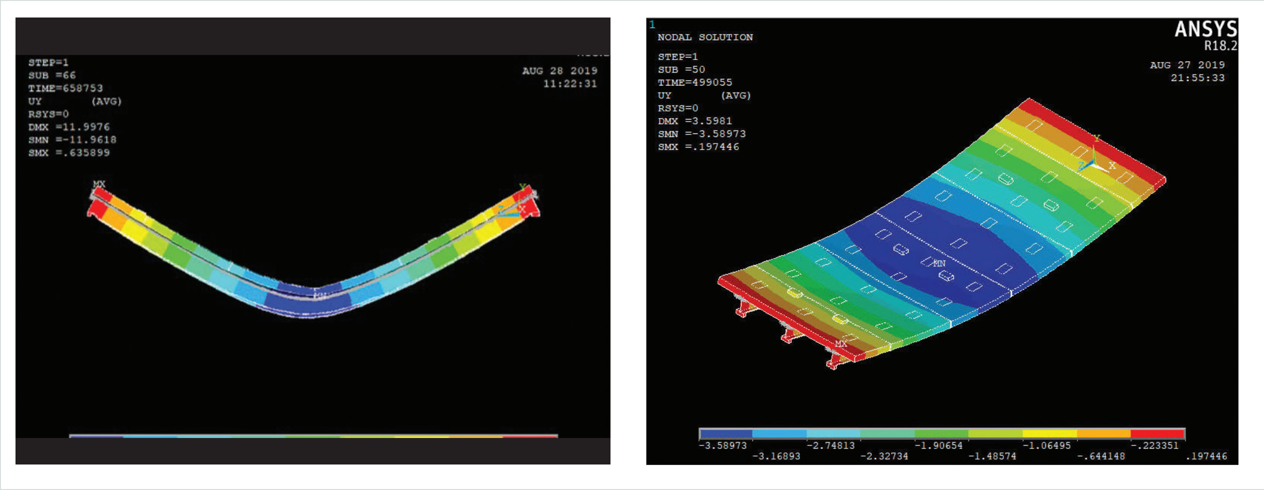

Figure 9. Model views in failure from the finite element analysis. Note: 1 in. = 25.4 mm.

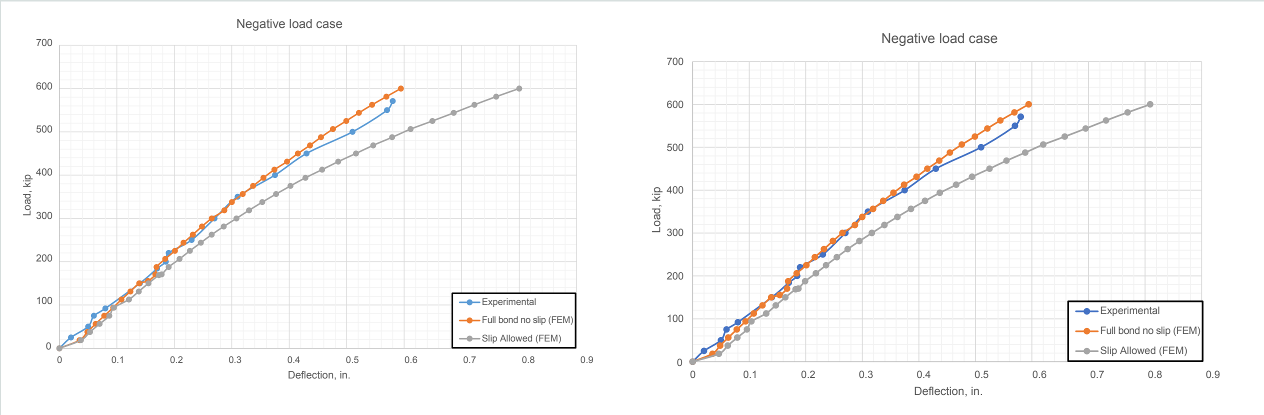

Figure 10. Deflection versus negative flexure load under truckloads HS-20 and HL-93. Note: FEM = finite element method. 1 in. = 25.4 mm; 1 kip = 4.448 kN.

Positive flexure case

Deflection versus positive flexure load curves were used to compare the FEA results and the experimental laboratory testing. **Figure 11** presents the positive load case. The figure shows experimental results for the middle steel beam load–deflection curve at 16 ft (4.88 m) for the test of the service load at the maximum positive moment. The load–deflection curve demonstrates a linear relationship. A maximum deflection of 0.22 in. (5.6 mm), as shown in the curve, indicates that the deflection corresponded to length of the span *L*/2180, which is less than the AASHTO serviceability limit for continuous spans. A maximum limit of 0.02 in. (0.508 mm) in the composite section was recorded.

The FEM curves show that slippage at the interface was 0.00227 in. (0.056 mm) and the deflection was 0.243 in. (6.1722 mm) for the service load of 93.75 kip (417 kN) in the composite girder. Cracks were observed at an ultimate load of 600 kip (2668.9 kN) and a deflection of 2.028 in. (51.5 mm) at the interface between the girder and load test, an indication of non-composite action failure due to maximum load applied (Fig. 7).

At the full bond, a service load of 93.5 kip (415.9 kN) was provided by the FEA program, resulting in a deflection of 0.22 in. (5.588 mm), and failure occurred at a load equal to 571 kip (2539.8 kN), with no slippage recorded.

For the service load value under truckload HL-93, 119.75 kip (532.6 kN), the deflection was 0.292 in. (7.4168 mm), with the slippage being recorded as 0.0031 in. (0.07874 mm). **Figure 12** shows negative and positive moments of load–strain curves in a relationship between the FEA and experimental test results.

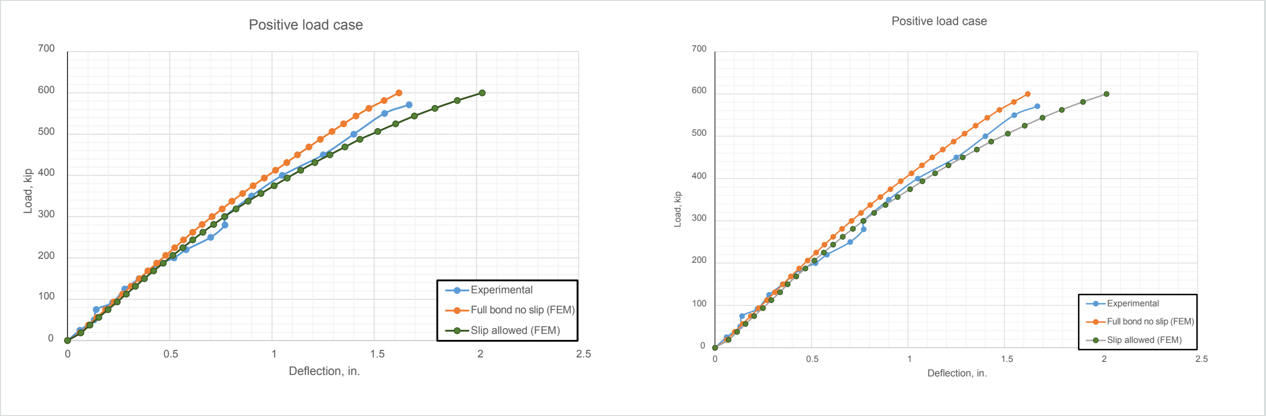

Figure 11. Deflection versus positive flexure load under truckloads HS-20 and HL-93. Note: FEM = finite element model. 1 in. = 25.4 mm; 1 kip = 4.448 kN.

Figure 13 shows the cracks from the actual test conducted by Issa et al.[8] The authors of that study noted there was a "transverse crack (approximately 0.5 mm wide) across the entire width of the bridge deck at the ultimate load level. The recorded compressive strain at the bottom of the slab over the central support was 1000 με. In addition, a tensile strain of 600 με was reached at the bottom of the slab at 14 ft (4.27 m) from the central support."[8]

Simply supported case

This case mainly supported the development of the span for the similar full-scale prototype. In the simply supported case, there was no negative moment because the two post-tensioning bars were removed from the model.

Comparison of FEA results with AASHTO specifications and findings from other research

Load–slip curves and measured slippage for all models (Table 2) were plotted for each loading increment until failure occurred within the load. The relationship between load slippage within the range of 0.02 to 0.8 in. (0.508 to 20.32 mm) was observed in all models, though for most models the critical slippage was about 0.02 in. Critical slippage is characterized as an increment in the slippage with a light increment in load. To create clamping stresses over the shear interface and improve horizontal shear capacity, a certain amount of relative horizontal slip happens.

Table 3 presents the load and slippage for full-scale models. As the load increased, separation occurred between the steel beam and haunch, and this progressive increment in slippage was recorded at each region. At the final stage of loading, there was a sharp increase in slip until failure was imminent. As the stud started yielding, cracking and local crushing of the concrete occurred. With continued loading, the stud would fracture with the structure.

The minimum area of interface shear reinforcement specified in article 5.8.4.4 of the eighth edition of the AASHTO LRFD specifications[18] must be satisfied. The equations from the AASHTO LRFD specifications were used to find the shear strength.

The AASHTO LRFD specification equation was applied to all models using a value for contact cohesion between the precast concrete beam and cement grout c_2 of 0.075 ksi (0.52 MPa), a friction factor μ of 0.6, an area of concrete considered to be engaged in interface shear transfer A_{cv} of 24 × 12 in. (609.6 × 304.8 mm) or A_{cv} of 48 × 12 in. (1219.2 × 304.8 mm), and if the force is tensile, a permanent net compressive force normal shear plane P_c of 0. The area of the bolts used was 0.785 in.^{2} (506.5 mm^2) with a yield strength of 36,000 psi (248.2 MPa) and an ultimate strength of 58,000 psi (400 MPa). The contact surface area was 12 in. wide and 24 in. long. The shear connector blockouts were also spaced at 24 in. on center and 48 in. and, in all cases, the predicted horizontal shear strength using AASHTO Eq. (5.7.4.3-3)[18] was less than the observed values from the tested specimens. **Table 4** further compares the shear strength as observed from FEAs for three models from this study with predicted values calculated from the AASHTO LRFD specifications.

Summary and conclusion

A prototype constructed by Issa[8] was selected for evaluation because it represented the geometry of the deck panels with full-depth precast concrete. The aim of the research was to investigate the constructibility and structural behavior of a prefabricated full-depth precast concrete bridge deck slab system. The comparison between live load test data and FEA models revealed a better provision of structural integrity. This was a result of modular slab systems that are tightly integrated, with high forces of post-tensioning. The load–deflection results from the FEA models were closely related to data from field tests. Furthermore, the comparison between field test data and measured data from the FEA model leads to the

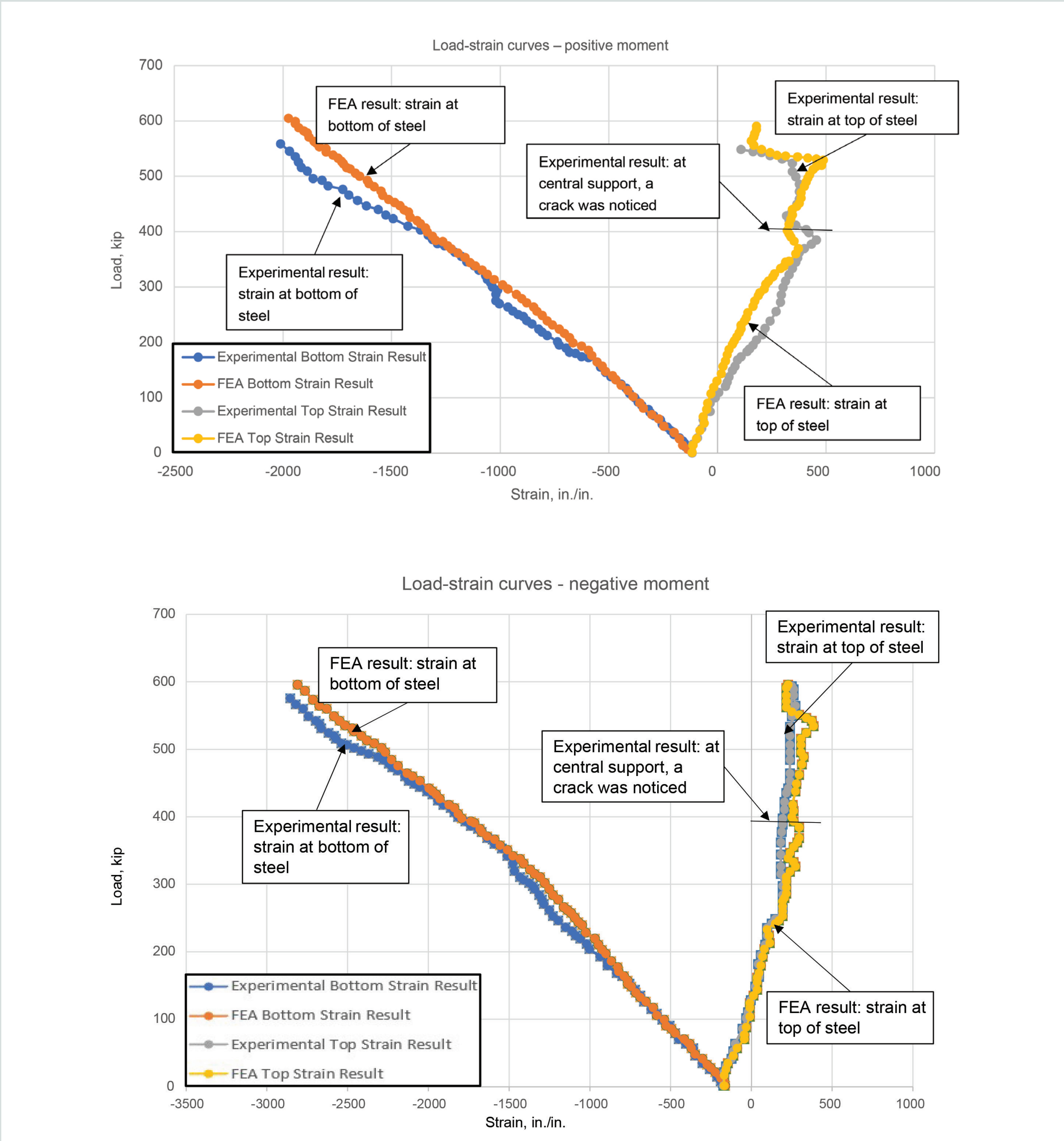

Figure 12. Load-strain curves at central support under negative ultimate load test and under positive moment ultimate load test. Note: FEA = finite element analysis. 1 in. = 25.4 mm; 1 kip = 4.448 kN.

conclusion that when the number of shear studs was 9 and the distance between the shear pockets was 48 in. (1219.2 mm), as for model 5, slippage in the bridge was improved.

With respect to the experimental and nonlinear FEA results, the following conclusions can be drawn. The shear studs and the shear pockets provide full composite action between the support system and the deck at all loading stages. From the various models of the distance between shear pockets and shear studs, the section of full-depth composite that was obtained from the experimental and FEA results confirms these conclusions:

- There was considerable agreement between the experimental and FEA results in terms of the strength capacity of the ultimate shear and the mode of failure for all types of units.

- Pocket spacing is considered to be an essential factor. The number of studs and the configuration affects the necessary load for slippage induction. The ultimate strength

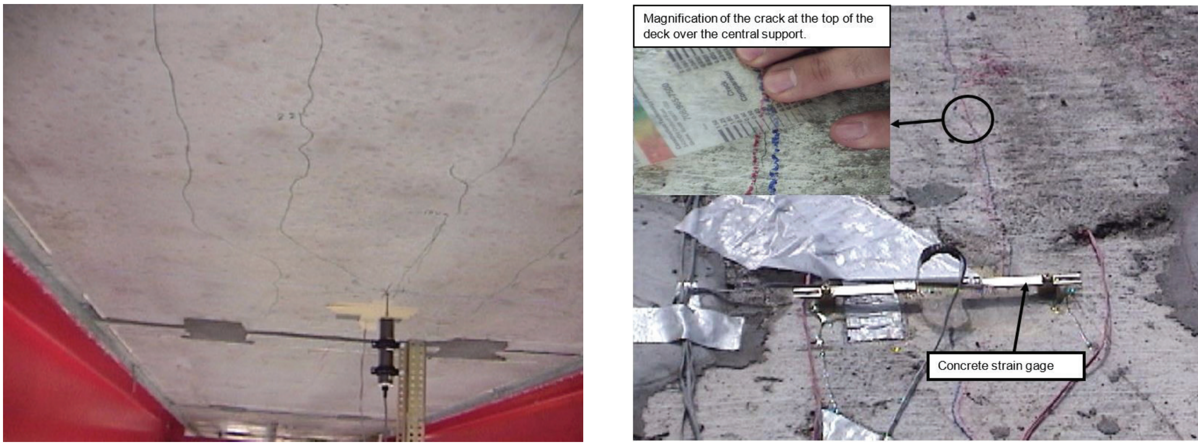

Figure 13. Observed cracks in experimental testing.
Source: Reproduced by permission from Issa et al. (2007, Fig. 12 and 16).

Table 3. Load and maximum slippage for models

Designation	Number of studs	Shear pocket spacing, in.	FEA load at maximum slippage, kip	Maximum slippage, in.	Expected failure load, kip
Calibration continuous span	3	24	n/a	n/a	420
Model 1	Full bond	n/a	768.5	0	n.d.
Model 2	3	24	568.7	0.42	n.d.
Model 3	3	48	459.1	3.43	n.d.
Model 4	6	48	499.06	0.313	n.d.
Model 5	9	48	659.1	0.80	n.d.

Note: 1 in. = 25.4 mm; 1 kip = 4.448 kN; FEA = finite element analysis; n/a = not applicable; n.d. = no data.

Table 4. Comparison of FEA results of applied load and shear strength of *AASHTO LRFD Bridge Design Specifications*

Designation	Number of studs	Ultimate load observed, kip	Horizontal shear strength from Ansys analysis, kip	Horizontal shear strength estimated using AASHTO LRFD specifications,* kip
Model 2	3	240	72.1	60.48
Model 4	6	499.06	192.1	174.096
Model 5	9	659	412.3	371.90

Note: AASHTO = American Association of State Highway and Transportation Officials; FEA = finite element analysis. 1 in. = 25.4 mm; 1 kip = 4.448 kN.
* Using Eq. (5.7.4.3-3) from the eighth edition of the *AASHTO LRFD Bridge Design Specifications*.

increased with an increase in the number of shear pockets, though the ultimate load increase became more pronounced as the studs per pocket increased.

- FEA modeling outcomes demonstrated that the use of nine shear studs with a pocket spacing of 48 in. (1219.2 mm) gives the best behavior between load, deflection, and slippage under ultimate load. Furthermore, when the shear pocket distance is doubled, the number of shear studs increases by a factor of three from the experimental number (3 shear studs).

- FEA and experimental outcomes show that concrete strength, shear connector size, and the yield stress of shear connectors substantially influence interface shear resistance of modeled concrete deck panels.

- For both negative and positive bending, the precast concrete bridge deck system showed adequate structural behavior protected from any cracking under service loads.

- Regarding FEA results, the impact of longitudinal post-tensioning was noticeable in ensuring the reliability of the bridge deck system under the load levels used, demonstrating roughly six times the service load. After releasing the applied load, the bridge system returned to its original position with minimal deformation due to the yielding of the middle steel beams.

Acknowledgments

The experiments to validate the FEA model were conducted at the structural laboratory at the University of Illinois Chicago (UIC). Reem S. Hadeed wishes to thank her UIC affiliates who provided the experimental data for their continuous help and support throughout the research period. Special thanks go to everyone who provided help and support to complete this project.

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Notation

A_{cv} = area of concrete considered to be engaged in interface shear transfer

A_{vf} = area of interface shear reinforcement crossing shear plane

c = cohesion factor

c_1 = value for contact cohesion between cement grout and flange of a steel beam of 0.025 ksi

c_2 = value for contact cohesion between the precast concrete beam and cement grout of 0.075 ksi

d = diameter of shear stud

E = modulus of elasticity

f'_c = compressive strength of concrete

f'_g = grout compressive strength

f_y = yield strength of reinforcement

K_1 = fraction of concrete strength

K_2 = limiting interface shear resistance

L = length of the span

P_c = permanent net compressive force normal shear plane

μ = friction factor

ν = Poisson's ratio

ν_{nh} = nominal horizontal shear resistance

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Abstract

This theoretical research used the finite element method to model full-depth precast concrete deck slabs. It examined the effects of pocket spacing and number of shear studs per pocket. Results indicated that horizontal shear strength increases with the number of pockets in both supported and continuous-span models. Load–deflection curves and slippage values were recorded and compared with experimental tests, confirming that pocket spacing is an important factor and that the number of studs and their configuration affects the load for slippage induction. The parametric study evaluated effects of shear connector sizes and various numbers of shear studs for distances of 24 and 48 in. (609.6 and 1219.2 mm) between shear pockets for simply supported bridges; however, there is a need for experimental evaluation of shear connector performance on design.

Keywords

BEAM188, CONTACT175, deflection, girder, post-tensioning, shear pocket, slippage.

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