PCI/ NATIONAL BRIDGE CONFERENCE 2006

Topic area: Precast bridge deck solutions, **Paper #42**

UTILIZATION OF LARGE-SIZE STUDS WITH PRECAST CONCRETE DECK PANELS

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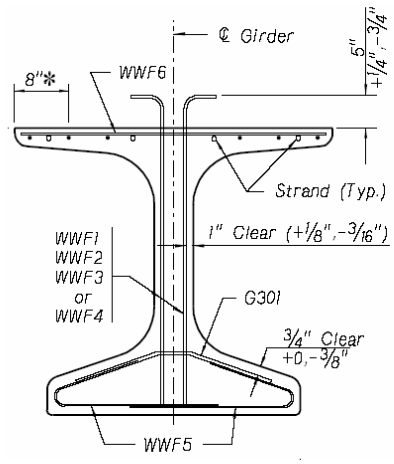
Introduction and background

Majority of bridges in the United States are typically made composite with the concrete deck slab. In general, composite construction provides bridges with high span-to-depth ratio that allows longer spans to be covered without affecting the vertical clearance, wider girder spacing that reduces the cost of the super structure, and stiffer structures with less deflection and vibration under service loads compared to non-composite construction.

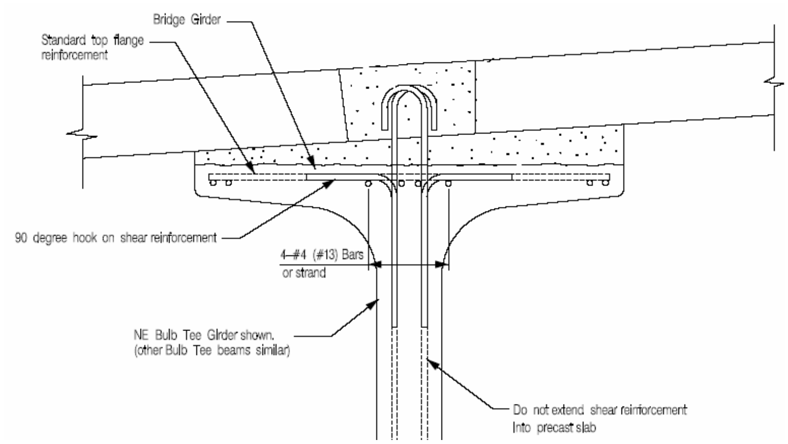
Composite action is typically created by using shear connectors that are extended outside the top surface of the girder and embedded in the deck slab. The shear connectors resist the horizontal shear that is caused by the superimposed loads, at the girder-deck interface. For concrete girders, the shear connectors can be provided by:

(1) Extending the vertical shear reinforcement of the web outside the top flange, where it takes an L-shape, as shown in Figure 1-a. Although this method provides for an inexpensive way for creating the composite action, it puts the shear connectors under corrosion risk because the vertical shear reinforcement of the girder is usually made of black bars. The majority of the bridge owners require that all the reinforcement in the deck should be corrosion resistant by epoxy coating the reinforcement or using clad or stainless steel reinforcement. Applying this requirement on the vertical shear reinforcement of the girder has been found relatively expensive and complicates fabrication of the precast girder.

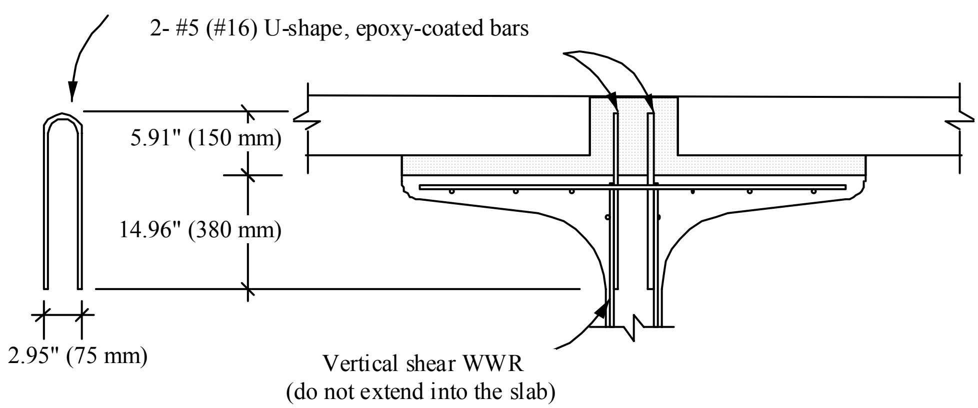
(2) Inserting individual U-shape bars in the top part of the girder, as shown in Figures 1-b and 1-c. The individual shear connector pieces can be made corrosion resistant by epoxy coating the reinforcement or using clad or stainless steel reinforcement.



(a) L-shape shear connectors (adopted from NDOR Manual, 2005[1])



(b) Two-individual U-shape bars placed in the transverse direction (adopted from PCINER-01-PDPG[2])



(c) Two-individual U-shape bars placed in the longitudinal direction (adopted from Tadros et al, PCI 2002[3])

Figure 1. Various types of shear connectors used with concrete girders

For steel girder bridges, headed steel studs are commonly used to create for the composite action, as shown in Figure 2. The studs are welded to the top surface of the flange using a semi-automatic arch-shielding procedure. The procedure involves using a welding gun that holds the stud and has a trigger-activated circuit to initiate the weld[4]. The welding gun has a lifting mechanism to draw the stud away from the base material and initiate the welding arc. Research conducted on composite systems using headed steel studs has proven the feasibility and cost effectiveness of this system[5-8].

The steel studs are made with various sizes ranging from 1/4 in. (6 mm) up to 7/8 in. (22.2 mm) in diameter. In bridges, the 3/4 in. (19 mm) and 7/8 in. (22.2 mm) diameter studs are typically used due to the heavy superimposed dead and live load exist on bridges. In high shear areas of steel girder bridges, as many as three 7/8 in. (22.2 mm) diameter studs per row are used to satisfy design requirements, as shown in Figure 2. The relatively high number of studs has many disadvantages, such as: (1) long installation time; (2) little or no room is left on the top flange for the construction workers to walk, which raises safety concerns, and (3) difficult deck removal that may damage the studs as well as the girder top flange. For these reasons, a girder-to-deck connection that reduces the number of shear studs could be advantageous.

In 1997, a group of researchers developed a large size, 1¼ in. (31.8 mm) diameter stud. The stud development was initiated on the NCHRP 12-41 research project titled "Rapid replacement of Bridge Decks"[9] and then the stud was used on demonstration bridges in Nebraska[3,10,11].

Recently, the 1¼ in. (31.8 mm) diameter stud has been considered for further investigation, for use with full-depth precast concrete panels made composite with steel or concrete girders, on the ongoing NCHRP 12-65 research project titled "Full-Depth, Precast-Concrete Bridge Deck Panel Systems"[12] .

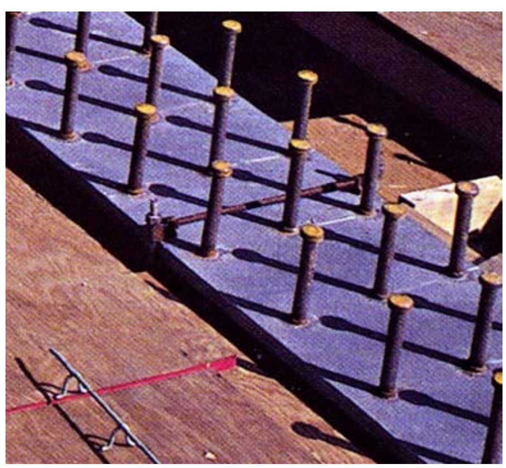


Figure 2. Three 7/8 in. (22.2 mm) diameter studs per row used on a steel beam

This paper provides information on: (1) development of the 1¼ in. (31.8 mm) stud; i.e. dimensions, manufacturing and welding processes, and quality control testing, (2) use of the 1¼ in. (31.8 mm) stud on concrete girders, (3) advantages of using the 31.8

mm (1¼ in.) stud with precast concrete deck panel systems, and (4) discussion on the 24-in. (600 mm) maximum stud spacing mandated by the AASHTO specification. Also, the paper provides a discussion on the applicability of the current AASHTO specifications[13,14].

Development of the 1¼ in. (31.8 mm) stud

(a) Material and dimensions

The size of the large stud was determined to satisfy the following conditions: (1) cutting the number of the 7/8 in. (22.2 mm) diameter studs by as much as 50 percent, (2) using available material in the market, (3) using the same technique and equipment currently used for welding the 7/8 in. (22.2 mm) studs on steel girders, and (4) maintaining a competitive price of the large stud with the 7/8 in. (22.2 mm) diameter stud. To fulfill these conditions, the researchers conducted a search in cooperation with stud manufacturers in order to find the steel grade and row material that can be used in producing the large stud. The study revealed that Society of Automotive Engineering (SAE) 1008 steel, cold drawn, currently used for the 7/8 in. (22.2 mm) studs, or SAE 1018 steel is available in 1¼ in. (31.8 mm) diameter. The 1¼ in. (31.8 mm) diameter was chosen because a 1¼ in. (31.8 mm) diameter circle has almost twice the cross-sectional area of a 7/8 in. (22.2 mm) diameter circle, which will reduce the number of 7/8 in. (22.2 mm) studs by 50%. Also, SAE 1018 was considered because it has higher tensile strength than the SAE 1008, which will result in higher reduction of the number of 7/8 in. (22.2 mm) studs. Table 1 gives a comparison of the mechanical properties between the SAE 1008 and SAE 1018 steel grades.

Table 1. Minimum Mechanical Properties of SAE 1008 and 1018

	SAE 1008 (cold drawn)	SAE 1018 (cold drawn)
Tensile strength, MPA (ksi)	340 (49.0)	440 (64.0)
Yield strength, MPa (ksi)	290 (41.5)	372 (54.0)
Elongation %	20	15
Reduction in area %	45	40
Brinell hardness	95	126

The study also revealed that the 1¼ in. (31.8 mm) stud can be produced as a headed stud, similar to the 7/8 in. (22.2 mm) studs, or as a headless stud by threading the top part of the stud and adding a hexagonal nut. The headless 1¼ in. (31.8 mm) stud with hexagonal nut was used in the early stages of development and the implementation projects due to the lack of forging equipment at the local stud manufacturers[9]. Recently, after the 1¼ in. (31.8 mm) stud has gained popularity among the bridge design engineers, many stud manufacturers have been able to produce the headed stud with a competitive price compared to the headless stud[12].

The stud's head plays an important role in developing the full tensile capacity of the stud. When horizontal shear forces are provided at the interface, the deck slab starts to move vertically away from the girder applying upward vertical thrust on the bottom surface of stud head and tensile force in the stud stem. As a result, the stud's head resists this force by applying high compressive stress on the concrete mass surrounding the stud. The compressive stresses helps to confine the concrete around the stud's stem, protecting it form premature failure and helping the stud to develop its full tensile capacity. The effect of enhanced confinement has been recognized in some design specifications and building codes in the formulas used to determine development length of bars in tension. For example, see Equation (12-1) of the ACI Building Code[15], where the effect of enhanced confinement is represented by the factor K_{tr} provided in the denominator.

The size of the large stud's head was determined by almost doubling the head-to-stem cross sectional area ratio of the 7/8 in. (22.2 mm) stud. This decision was taken based on the fact that the 1¼ in. (31.8 mm) stud has twice the tensile capacity of a 7/8 in. (22.2 mm) stud. This resulted in a 2½ in. (64 mm) diameter of the stud's head. Figure 3 show the dimensions of headed and headless 1¼ in. (31.8 mm) studs. Regarding the length of the 1¼ in. (31.8 mm) stud, the researchers decided to follow the AASHTO specifications recommendations, where: (1) the stud's head should be higher than the bottom layer of reinforcement of the concrete slab, (2) the ratio of the stud length after installation to its diameter should not be less than four, and (3) the 2-in. (50-mm) minimum concrete clear cover on the stud should be satisfied.

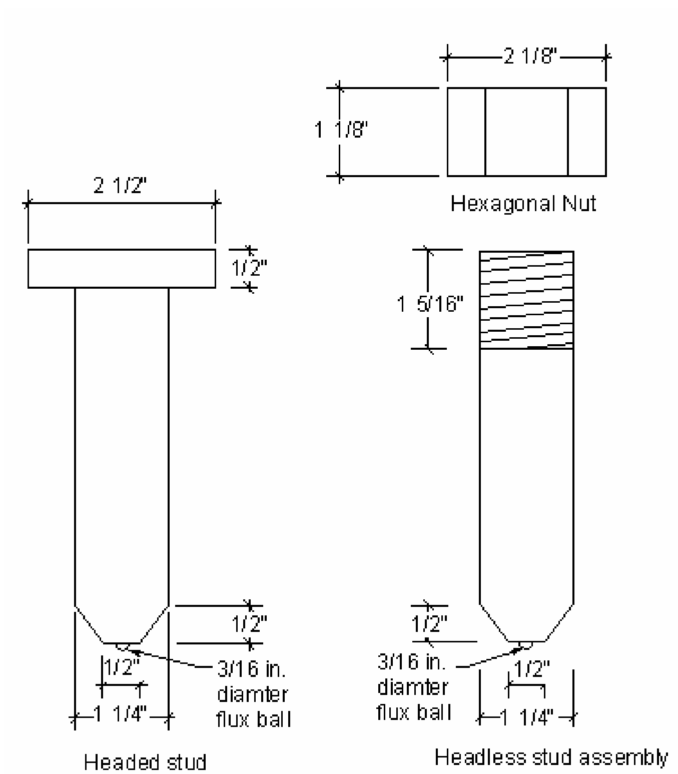


Figure 3. Dimensions of the larger diameter stud

(b) Welding of the large diameter stud on steel girders

The researchers determined that the arc stud welding process that is currently used in welding the 3/4 in. (19.1 mm) and the 7/8 in. (22.2 mm) studs could be used for the larger stud, because of its availability, productivity, and familiarity. During this welding process, a controlled electric arc is used to melt the base of the stud and a portion of the base metal. The stud is thrust automatically into the molten metal and a high-quality fusion weld is produced.

The ‘‘chuck’’ of the welding gun that grips the stud was modified to fit the large headed or headless stud, as shown in Figure 4. Many welding trials were conducted to determine the factors that may affect the welding quality. Three factors were the slope of the stud chamfer, amount of flux, and power supply. During early welding trials, it was evident that steeper chamfer and more flux than those used with the 7/8 in. (22.2 mm) studs would facilitate the welding process and lead to high-quality welding. Thus, the 1¼ in. (31.8 mm) stud was provided with a steep chamfer and the amount of flux material was tripled compared to that used with the 7/8 in. (22.2 mm) studs. Since the 1¼ in. (31.8 mm) stud has a larger cross-sectional area than the 7/8 in. (22.2 mm) studs, it was expected that welding would require a power source with higher amperage. Welding trials showed that a power source of 2,400 minimum amperage, which is currently available from many commercial vendors, would produce enough heat to melt the stud base and lead to good welding quality. Note that welding the 7/8 in. (22.2 mm) stud usually requires amperage in the range of 1,800–2,000. With the above medications, excellent welding quality was achieved.

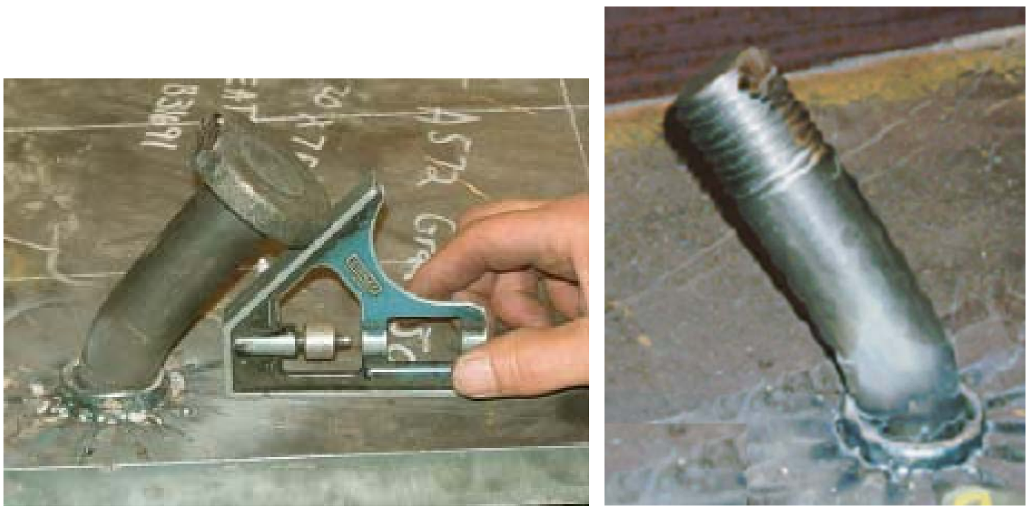
Figure 4. Welding gun with the special chuck used for the large diameter stud

(c) Quality control testing of welding

Bridge owners require testing of studs for quality assurance. Most specifications require that studs welded to the steel girders be bent at a 45-degree angle, using a sledge hammer, without failure at the weld. This procedure has been successfully used with the 1¼ in. (31.8 mm) as shown in Figure 5. However, it was noticed that the top part of the stud was damaged during testing, due the large force that is needed to bend the stud.

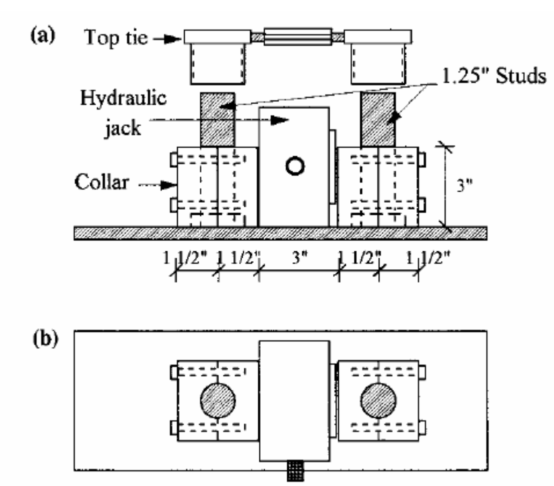
The researchers developed a portable hydraulic jacking system that could be used in the shop or in the field for testing pairs of studs, as an alternative to the stud bending procedure. The device, shown in Figure 6, consists of two collars placed around two adjacent studs, a small hydraulic jack, and a top tie. The collar consists of two steel blocks tied together with four screws. By tightening the four screws, the collar is in full contact with the stud. The base of the collar is chamfered to accommodate the weld at the stud base. A hydraulic jack is placed between the collars to provide lateral shearing force at the stud base. The top tie, which consists of two hooks and a turnbuckle, is used to protect the studs from bending.

The quality-control test is conducted by applying a horizontal force to cause a tension failure in the stud. The force is calculated by analyzing the studs with the top tie as a frame structure, where the studs are fixed at the base and hinged at the top. By equating the principal stresses at the stud base with the stud yield strength, a relationship between the applied force and the stud yield strength is derived. To protect the stud from damage during the quality-control test, an appropriate factor of safety may be applied. This device has been used successfully in addition to the bending test on the demonstration bridge projects[11,16].



(a) Headed stud (b) Headless stud

Figure 5. Quality control test by bending the large diameter stud to 45 degrees



(c)

Figure 6. Quality-control test using a portable hydraulic jacking system: (a) elevation, (b) plan, (c) general view

(d) Experimental investigation

In order to investigate the applicability of the design procedure given by the current ASSHTO bridge specifications[13,14], the research team conducted a comprehensive experimental program, which consisted of: (1) 20 push-off specimens for ultimate strength investigation; (2) 25 push-off specimens for fatigue resistance investigation; and (3) one full-scale beam test. The experimental investigation has proven that the large diameter stud can be conservatively designed using current AASHTO Specifications. For more information, please see References 9 and 16.

(e) Demonstration projects

The 1¼ in. (31.8 mm) stud was used on two projects in Nebraska. The first project was a three-span continuous bridge in western Nebraska, on Highway 71 in Gering South, Nebraska, consists of three continuous spans of 45, 60, and 45 ft (13.7, 18.28, and 13.7 m)[16]. The cross section of the bridge consists of five W30x99 rolled steel girders spaced at 8 ft, 9 in. (2.67 m) made composite with a 7.5 in. (190 mm) thick cast-in-place slab. Headless 1¼ in. (31.8 mm) stud with hexagonal nut was used on the south span, one stud per row welded directly over the girder web, with spacing from 7–10 in. (177 to 254 mm). The 7/8 in. (22.2 mm) stud was used on the center and north span, three studs per row at spacing from 10–16 in. (254 to 407 mm).

The bridge construction was completed in the fall of 1999. The researchers and NDOR designers took deflection measurements of the bridge using a three-axle dump truck. Deflection measurements were taken at the maximum positive moment section of the center girder of the exterior spans. Both exterior spans showed the same amount of deflection 0.12 in. (3 mm). Continuous visual inspection of the bridge deck has shown no cracks or distress on the south span where the large diameter stud was used.

The second project is the Skyline Bridge, Omaha, Nebraska[11]. The bridge carries the Skyline Drive traffic over US 6 Expressway. The bridge has two unequal spans 89 and 124.5 ft (27150 and 37950 mm) and the superstructure has five steel plate girders spaced at 10 ft, 10 in. (3300 mm) made composite with a full-depth precast concrete deck system. The headless 1¼ in. (31.8 mm) stud with hexagonal nut was used on the entire bridge to create for the composite action. Continuous open channels were created in the precast concrete panel over the girder lines to provide space for the shear connectors, as shown in Figure 7.

The large studs were provided on one line over the web of the steel girder at a uniform spacing of 6 in. (152 mm). The studs were welded by the steel fabricator at the steel shop at a rate of 40 to 50 seconds per stud. The quality control test was conducted by bending the stud to 45 degrees and using the hydraulic jacking device. Both tests have shown a high quality welding. The studs were arranged so that they did not interfere with the transverse reinforcement of the precast panel passing over the girder lines.

Figure 7. Large diameter studs used with precast concrete deck panels

The continuous open channels were filled with Type-K non-shrinkage cement mortar and cured in-place using wet burlap. Construction of the deck was complete early on 2004 and the bridge was open to traffic by March 2004. The bridge has been under continuous monitoring for about one year, where deflection measurements have been within the planed design limits. Routinely visual inspection of the deck has shown neither any separation between the deck and the steel girders nor any signs of cracks or distress.

USE OF FULL-DEPTH PRECAST CONCRETE DECK PANELS MADE COMPOSITE WITH THE SUPER STRUCTURE

Creating a composite action between the precast deck and the supporting girders has been one of the challenges that faced the design engineers in design of precast concrete panel deck systems. Intermediate pockets over the girder lines have to be created in the panel to accommodate the shear connectors extending from the supporting girders into the precast deck. Also, the shear connectors have to be clustered in groups lined up with these pockets. For a bridge owner, it is advantageous to reduce the number and size of the shear pockets for the following reasons:

(1) To simplify and speed up the fabrication process of the panels. Forming of the shear pocket typically slows down the fabrication process of the panels and eventually raises the fabrication cost.

(2) To reduce the volume of the non-shrink grout used to fill the shear pockets, which results in reducing the cost of the deck panel system and increasing the construction speed, especially for over night deck replacement projects.

(3) To reduce the possibility of water leakage at the interface of the shear pocket and the grout filling it.

(4) To give the design engineer more flexibility in laying the transverse reinforcement of the panel.

Typically, two issues affect the number and size of the shear pockets. These are: (1) the size of the shear connector, and (2) the maximum spacing allowed by the specifications between the stud clusters.

Size of the shear connector

The literature review of NCHRP 12-65[12] revealed that 7/8 in. (22.2 mm) steel studs clustered in shear pockets were used on the majority of the bridges built with precast deck panel systems. The number of studs per pocket ranged from three to twelve studs depending on the amount of the horizontal shear stresses that is needed to be resisted at the interface, as shown in Figure 8. Although the horizontal shear stress varies over the span of the girder, where it is maximum at the girder end and minimum at midspan section, shear pockets at constant spacing and with fixed dimensions were used to simplify and speed up the fabrication process of the panels.

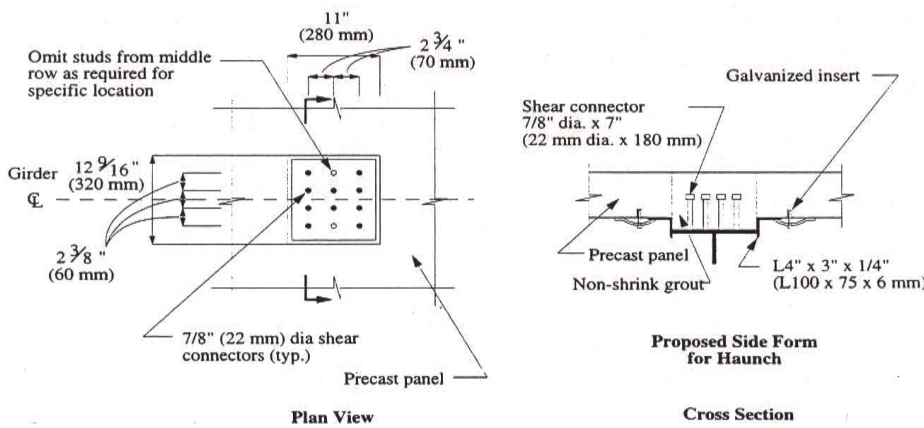


Figure 8. Details of the panel-to-girder connection used on Queen Elizabeth Way-Welland River Bridge, Ontario, Canada[12]

Maximum spacing allowed by the specifications between the stud clusters

The AASHTO Standard and LRFD Specifications[13,14] state that spacing between the shear connectors for steel or concrete girders should not exceed 24 in. (610 mm). Investigation of the origin of the 24 in. (610 mm) maximum spacing has revealed the following facts:

(1) The first composite concrete slab on steel I-beam bridges in the United States was constructed in the early to mid 1930s in Iowa.

(2) Newmark and Siess[17] in their paper in 1943 stated that *the spacing of the shear connectors shall be not more than 3 to 4 times the depth of the slab*. While this limit did not appear in the AASHO provisions currently used at that time, it appears to have been used as a convention or rule-of-thumb.

(3) The 24-in. (610 mm) maximum limit on shear connector spacing first appeared in the 4th Edition of the AASHO Standard Specifications for Highway Bridges in 1944 without commentary.

(4) A 1953 paper by Viest and Siess[18] contained a discussion of why mechanical connectors are needed. Their arguments include: 1) to prevent relative movement (either horizontal or vertical) between the beam and the slab during all loading levels up to ultimate and 2) to transfer horizontal shear from the slab to the beam. The discussion that supports these roles for shear connectors is primarily directed at insuring linear-elastic behavior of the composite system.

(5) Viest and Siess returned to this subject in a 1954 paper[19] that reports conclusions made from their experimental results and makes design recommendations. It should be noted that these experiments where carried out using the channel-type shear connectors that were conventional at the time. Although they did not comment on the origin of the 24-in. (610 mm) maximum connector spacing in the AASHO provisions, the experimental results support retaining the limit. The testing considered connector spacing of 18 in. (457 mm) and 36 in. (914 mm). While the 18-in. (457 mm) spaced connectors performed as necessary, the 36-in. spaced connector specimens experienced lift-off between connectors under load in the experiments. This result motivated the authors to recommend that "the maximum spacing of channel shear connectors be not greater than four times the thickness of the slab, but in no case greater than 24 inches."

The reader should note that all of the above mentioned research, that lead to the 24-in. (610 mm) limit, was conducted using cast-in-pace concrete deck slabs where the shear connectors are not clustered in groups. Yet, no research specifically addressing the maximum spacing of clustered shear connectors has been conducted yet.

Use of 1¼ in. (31.8 mm) stud system for composite deck/girder systems

The literature review, conducted in the on going NCHRP 12-65 project[12], has revealed that the majority of the precast concrete deck panel systems made composite with the superstructure were used with steel girders for the following reasons:

(1) Lack of information on how to create the composite action if concrete girders are used as most of the research conducted on this type of construction has been done using steel girders. Also, no guidance is provided from the AASHTO specifications on what type of shear connectors should be used (bars, bolts, etc.) and how to anchor them with the top flange to develop their yield strength.

(2) For new construction projects, expected camber of a prestressed girder provides a challenge for the precast producer as it affects the length of the shear connector that is embedded in the deck slab. This problem may force the design engineer to use shear connectors with various lengths and installed them according to a preplanned scheme, where the short pieces will be installed around the midspan area and long pieces installed close to the girder's end areas. Also, tight tolerance on the expected camber of the girder at time of installing the deck panels has to be imposed on the precast concrete producer.

(3) For deck replacement projects, rearranging the shear connectors used with concrete girders is an expensive and time consuming task because the new shear connectors

have to be installed in holes predrilled on the top flange of the girder. The holes have to be inspected by the bridge owner before installing the new shear connectors to make sure that they are clean from debris and have the right length. Also, drilling holes may interfere with the draped prestressing strands especially towards the end of the girder.

The research team of the NCHRP 12-65 research project[12] has been investigating extending the maximum spacing of clustered 1¼ in. studs to 48 in. (1220 mm) for use with precast concrete deck panels. The investigation considers steel as well as concrete girders, and includes shear-off and full-scale beam specimens.

Figure 9 shows one of the proposed connection details for concrete girder bridges. A u-shape 1¼ in. (31.8 mm) headed stud is used. The u-shape stud is provided in the longitudinal direction to avoid interference with the vertical shear reinforcement of the girder. Preliminary design of this system using the shear friction theory has shown that two u-shape studs spaced at 48 in. (1220 mm) can create full composite action for 130 ft (39.6 m) long bridge with girder spacing up to 11 ft (3.4 m).

This detail and others are under experimental investigation through the ongoing NCHRP 12-65[10]. Preliminary test results have shown that the 1¼ in. (31.8 mm) headed stud can be used successfully to create full composite action.

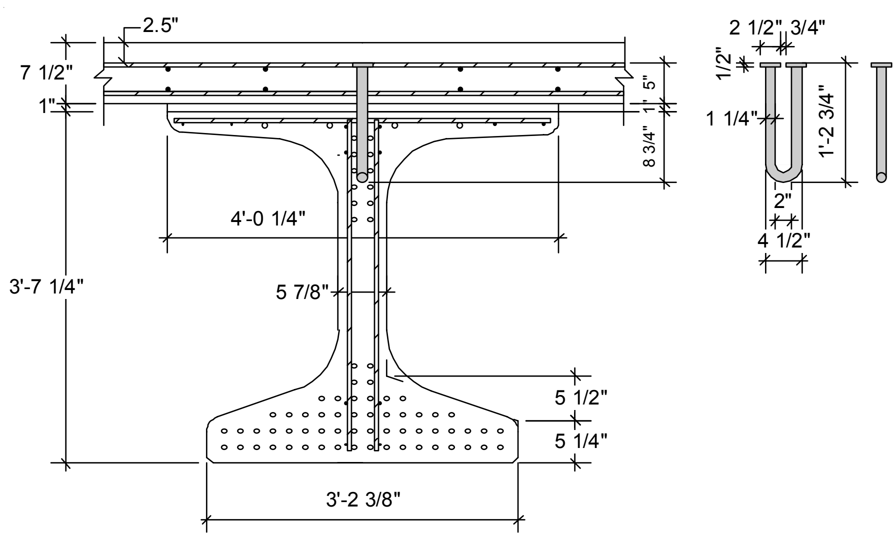


Figure 9. Composite connection detail using the large size stud

Conclusions

This paper presents information on the development of the 1¼ in. (31.8 mm) studs used for creating full composite action of deck/girder bridges. The new studs have almost double the cross-sectional area of the 7/8 in. (22.2 mm) studs. They can be produced using 1¼ in. (31.8 mm) SAE 1008 or 1018 rods that are commercially available, and can be produced as headless or forged headed studs.

Use of the 1¼ in. (31.8 mm) studs with precast concrete deck panel systems will significantly reduce the number of studs needed to achieve full composite action between the deck and the supporting girder. This will increase construction speed, ease deck replacement, reduce the possibility of damaging studs and the girder top flange during deck removal, reduce the size of shear pockets in the panel. It also will enhance the safety factor for construction workers because more space on the steel top flange will be available for the construction workers.

Acknowledgments

The research reported in this paper has been performed under the National Cooperative Highway Research Program projects, NCHRP 12-41 "Rapid replacement of bridge decks," and the ongoing NCHRP 12-65 "Full-depth, precast concrete bridge deck panel systems." Special thanks are extended to David Beal and Amir Hanna of the Transportation Research Board; Lyman Freemon, Gale Barnhill, Hussam Fallaha, Dan Sharp, and Mark Ahlman of the NDOR; Art Lerner of Master Bolts Inc., Tim Ruterkus of Tri Sales Associates, Inc., Art Lerner of Master Bolts, Inc., and Mark Lafferty of Concrete Industries, Lincoln, Nebraska. The authors would also like to thank the PCI reviewers for their constructive and thoughtful comments.

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