Tests Evaluating Punching Shear Resistance of Prefabricated Composite Bridge Units Made with Inverted Steel T-Beams

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Design information is developed for a special type of prefabricated composite superstructure unit for bridge spans in the 30- to 70-ft range. These steel and concrete units consist of span-length, 7-in. thick reinforced concrete slabs 6 to 10 ft wide. The webs of a pair of inverted T-shaped steel beams are embedded in each slab. Horizontal steel studs welded at intervals to the beam webs act as shear connectors. Since no steel top flanges are present over the beam webs, the spacing of the studs might be critical in preventing the web from punching through the slab under wheel loads of heavy trucks. To evaluate the resistance of two particular prefabricated units to punching shear and to develop general design information for determining safe stud spacings, tests were conducted on two 10-ft wide by 5½-ft long specimens representing a section of a typical bridge. The studs were spaced at 4-in. intervals in one specimen and at 10-in. intervals in the other. Both specimens were supported and loaded so that they would be subjected to punching shear.

Both specimens failed in a similar manner at loads that were about 5 times greater than the maximum wheel load (including 30 percent for impact) specified by AASHO for H20 or H20-S16 type trucks. The mode of failure appeared to be a combination of tension and bond failure in the concrete rather than a punching-type failure. Therefore, under actual highway loadings, failure of the slab by punching of the web through the concrete would not be expected even with large stud spacings in the portions of a bridge span where punching shear is the major force transferred between the concrete and the steel. The vertical shears created by wheel loads seem to be transferred from the slab to the beam web by both shear in the studs and bearing on top of the web. However, the amount of shear transferred by each mechanism could not be determined.

A method of determining the safe spacing of studs for resisting combined punching and horizontal shear was developed and was based on the conservative assumption that the studs carry all the punching shear. It was also assumed that the intensity of the punching shear was proportional to the deflection of the slab near the web, and that a conservative approximation of this relationship is that the shear intensity varies parabolically—from zero to maximum shearing stress to zero—over an 8-ft length. This procedure permits calculation of the maximum vertical shear per stud, thereby enabling possible use of conventional procedures in designing these prefabricated superstructure units.

Paper sponsored by Committee on Steel Superstructures.

•IN COOPERATION with the Indiana Steel Fabricators Association, the U. S. Steel Applied Research Laboratory is developing design information for a special type of prefabricated composite steel and concrete superstructure unit for highway bridges that is intended to be competitive mainly with prestressed concrete box beams for spans ranging from about 30 to 70 ft. These prefabricated bridge units are designed to be 6 to 10 ft wide and of span length. Typical details of their construction are given in the nondimensional sketch shown in Figure 1. Each unit consists of a 7-in. thick reinforced concrete slab supported by a pair of span-long steel beams that have the shape of an inverted T instead of the usual I-shape. The two beams are so placed that the top $3\frac{1}{2}$ in. of each web are embedded in the concrete. Horizontally positioned steel studs are welded to each side of each steel web along a line $1\frac{3}{4}$ in. from the top of the web. These studs transfer shear between the concrete and the steel shapes, and thereby make composite action possible. At the bridge site, adjacent slabs are connected through longitudinal grouted keyways and by transverse tie rods (Fig. 1).

Most of the problems involved in the design of bridges in which these prefabricated units are to be used can be solved by standard bridge design procedures. However, the elimination of the conventional steel top flanges suggests that wheel loads might cause the steel webs to punch through the concrete locally because no top flange is present to support heavy wheel loads, such as the 20,800-lb total wheel load (16,000-lb live load plus 30 percent for impact) specified by the American Association of State Highway Officials (AASHO) (1) for the H20 and H20-S16 loadings for which most major highways are designed.

The behavior of the slab in resisting this tendency of the web to punch through is very complex. When a wheel load is placed on a deck, vertical shears are created in the slab around the wheel and must be transferred into the beams by a combination of (a) bearing on the top surfaces of the studs, (b) bearing on the top surface of the beam webs, and (c) bond between the concrete and the adjacent vertical faces of the steel webs, although the contribution of this bond is probably small. Furthermore, near the ends of a bridge, the horizontal shear developed in the studs by composite action interacts significantly with the vertical shear transferred by the studs. The basic design problem, therefore, is to determine safe stud spacings that will resist mainly punching shear in the center of the span, where not much horizontal shear is present, and also resist combined punching and horizontal shear near the ends of the span, where horizontal shear is usually most significant. However, because the manner in which the vertical shears from the wheel loads are transferred from the slab to the studs and webs is very

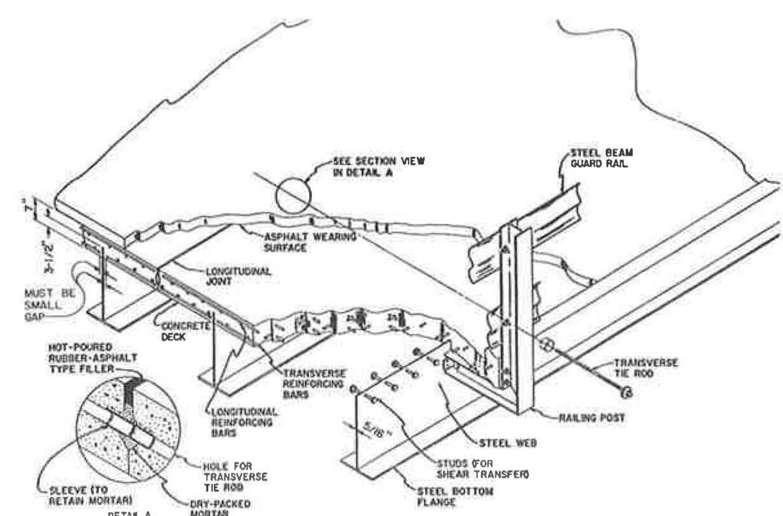


Figure 1. Prefabricated composite highway bridge unit with inverted steel T-Beams.

complex, accurate calculations for determining the stud spacing required to resist punching shear alone or punching shear in combination with horizontal shear due to composite action could not be made on a theoretical basis.

Consequently, the simple tests described herein were performed to determine experimentally whether a 4- and a 10-in. stud spacing would be adequate for resisting punching shear due to an H20 or H20-S16 loading, and if possible, to establish rules for calculating safe stud spacings to resist combined punching and horizontal shear. The results of the tests and the development of design information for determining these safe stud spacings are described.

MATERIALS AND EXPERIMENTAL WORK

For the testing program, two double-web units, differing only in the number of steel studs welded to the webs, were constructed at the U. S. Steel Applied Research Laboratory. The steel T's with studs attached were furnished by the Indiana Steel Fabricators Association. As shown in Figure 2, each test specimen was 10 ft wide by $5\frac{1}{2}$ ft long. This specimen size represented a portion of a 10-ft wide prefabricated unit between two transverse cross-sections $5\frac{1}{2}$ ft apart. The 10-ft width was selected for the tests because, theoretically, localized shears are greater in a 10-ft wide unit than in a narrower unit. The $5\frac{1}{2}$-ft length was selected because before the tests were performed it was believed that the punching shears from a wheel load would be most critical within such a length, and because of the size limitations in the laboratory test setup. The beams were fabricated T's consisting of $7\frac{1}{2}$- by $\frac{5}{16}$-in. webs and $7\frac{1}{2}$- by $\frac{5}{16}$-in. flanges of ASTM A441 steel, the thickness of the web being the least that would be used in an actual bridge. These particular flange and web dimensions were used for convenience in testing and are different from those that would be used in an actual bridge; however, these dimensional differences would not affect the type of test performed. The transverse distance between centers of the webs (interior span) was 6 ft, and each cantilever

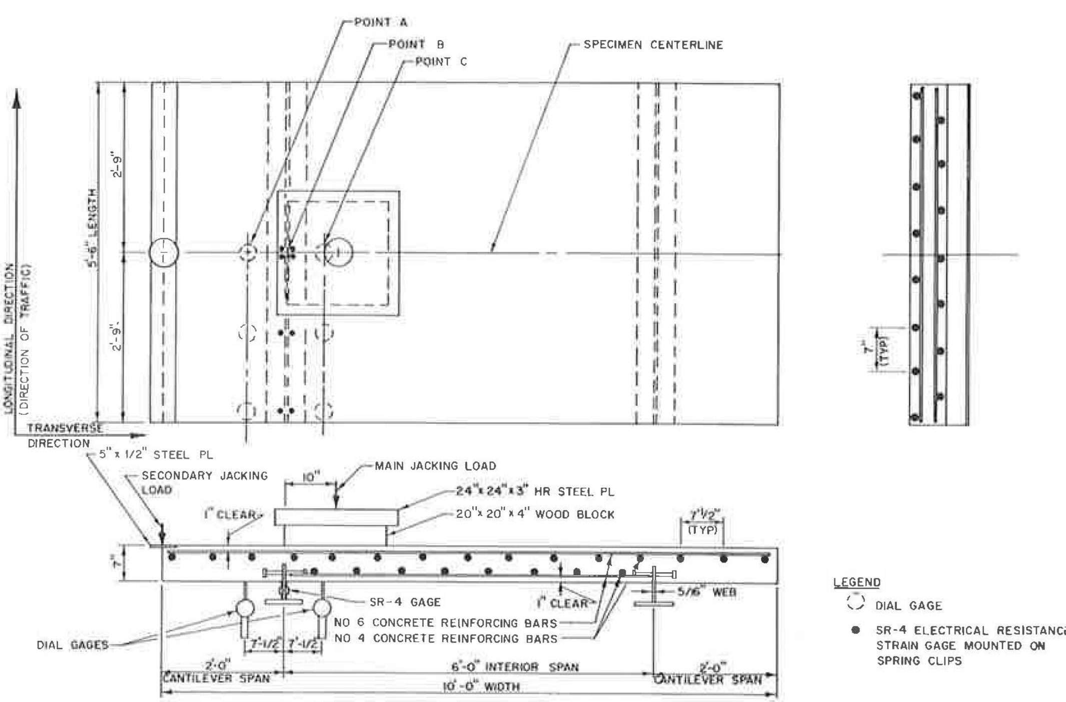


Figure 2. Testing arrangement for experimental T-cast units and details of their construction.

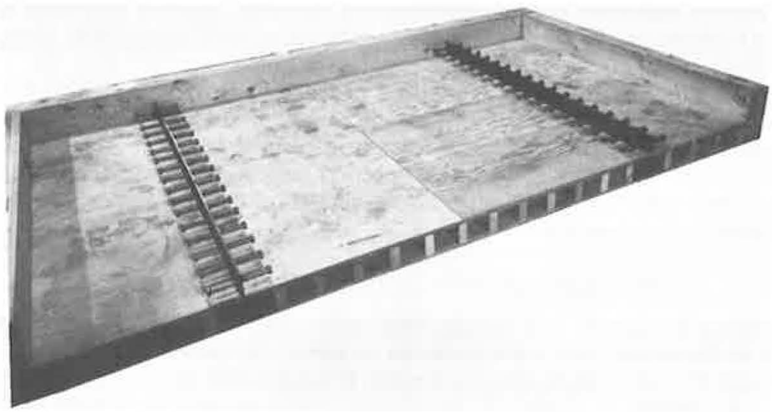


Figure 3. Casting form (one side removed) showing arrangement of studs (spaced at 4-in. intervals) on webs in experimental unit made for test 1.

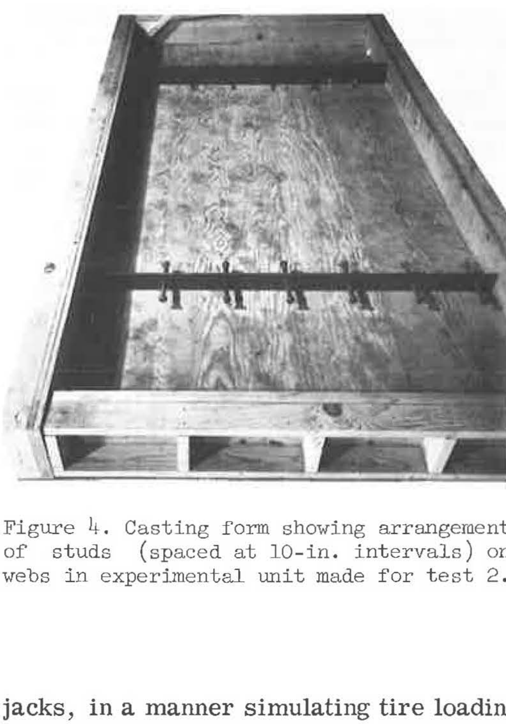


Figure 4. Casting form showing arrangement of studs (spaced at 10-in. intervals) on webs in experimental unit made for test 2.

span projected 2 ft beyond the web. The stud spacing on each side of each web was 4 in. for the specimen used in test 1 and 10 in. for the specimen used in test 2, as shown in Figures 3 and 4, respectively. These standard welded studs (1) were 4 in. long, with a ¾-in. diameter shank and a 1¼-in. head diameter. In accordance with the formulas in the highway bridge specifications (1), both slabs were reinforced transversely and longitudinally with intermediate grade steel reinforcing bars of sizes No. 6 and No. 4, respectively. Their locations in the slab are shown in Figure 2. The average ultimate compression strength of the concrete at the time the bridge units were tested was 3,710 psi, as determined by compression tests on four 6-in. diameter by 12-in. high concrete cylinders.

In each test, the specimen was placed with the steel T-flanges resting on a firm support so that no horizontal shear would occur. As shown in Figures 5 and 6, vertical loads were then applied by hydraulic jacks, in a manner simulating tire loadings from heavy H20-S16-type trucks positioned to produce maximum punching shear in the specimen. That is, the main vertical load was applied on a 20- by 20-in. wood block that simulated the imprint area of dual H20-type tires and was centered on the specimen centerline so that the edge of the wood block lined up with the interior face of a web. Positioning of the main load at this point should produce maximum punching shear in the interior span. However, in bridges built with these prefabricated units, consideration must also be given to the effect of loads on adjacent units. For example, when two trucks are simultaneously in adjacent lanes, and wheel loads of each truck are located so that they are 4 ft apart across the longitudinal joint between units, a downward reaction from the adjacent unit may occur along the joint edge. This reaction, theoretically as much as about 30 percent of a wheel load, increases the maximum punching shear in the interior span of the unit re-

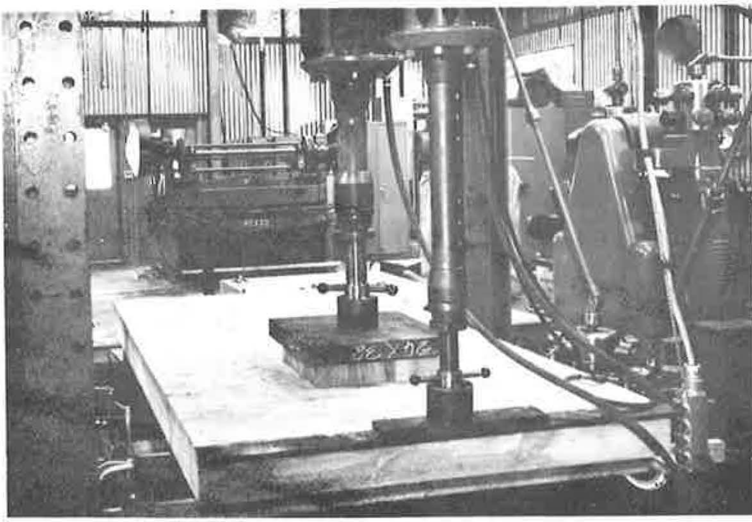


Figure 5. General view of loading arrangement for testing experimental units.

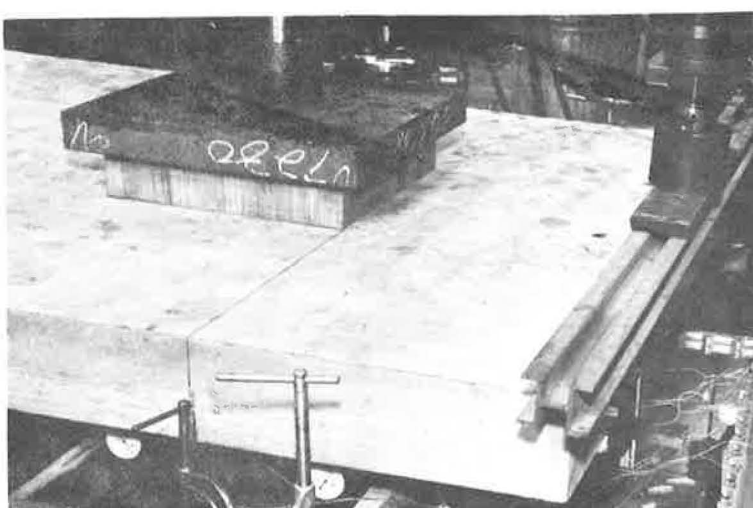


Figure 6. Close-up of experimental unit showing positions of main and secondary loads with respect to edge of beam web (pencil line on concrete); two of the dial gages for measuring deflections shown beneath slab.

ceiving the downward reaction. To simulate this situation in the test specimen, a secondary vertical load was applied at the edge of the specimen. Because it was not known how critical such a downward reaction at this point would be, the behavior of the specimen was studied under secondary vertical loads of up to 30 percent of the main load.

As shown in Figure 2, instrumentation for each test consisted of six dial gages, three in line with point A and three in line with point C, placed $7\frac{1}{2}$ in. from the web nearest the load, and eight electric resistance strain gages mounted on specially fabricated spring clips that were bolted to the web in line with point B. Since the top of each clip was in contact with the concrete, the vertical movement of the clip, which was calibrated with respect to the strain measured in its bent portion, was equal to the vertical slip of the concrete relative to the steel web.

The main load was applied in increments ranging from 5,000 to 10,000 lb. After each increment was applied, the slab was inspected for cracks, and both dial- and strain-gage readings were recorded. Then, with the main load constant, the secondary load was increased from zero to 30 percent of the main load. After another set of gage readings was recorded, the secondary load was reduced to zero, and the main load was increased one increment. This procedure was repeated until failure of the specimen occurred.

RESULTS AND DISCUSSION

The behavior of the two experimental superstructure units was almost identical. Because of this similarity of behavior, the test results for both units will be discussed together throughout the remainder of the paper.

As shown in the load-deflection curves in Figures 7 and 8, the vertical deflection of the slabs at point A on the specimen centerline (in cantilever span, 7½ in. from web) increased as the main load increased up to about 50,000 lb and then decreased slightly for greater loads. At point C on the specimen centerline (in the interior span, 7½ in. from web), increasing the main load continuously increased the deflection up to the maximum load applied, as would be expected. The deflections at point C were considerably greater than the deflections at point A and were, therefore, considered to be of much significance in analyzing the behavior of the slab. The increase of the deflection at point A and the decrease of the deflection at point C when the secondary load was applied are readily explained by the fact that the web between these points acts as a fulcrum. Also, as expected, the deflection of the slab was less at points away from the specimen centerline. For example, at a 90,000-lb main load and zero secondary load,

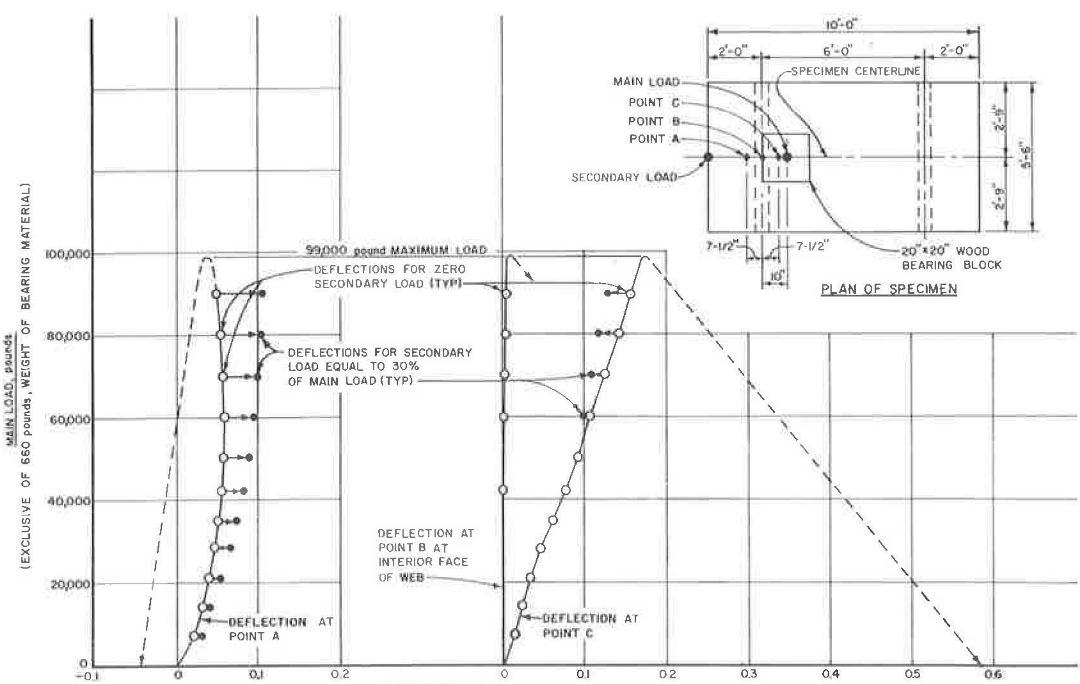


Figure 7. Specimen centerline deflections for test 1 (4-in. stud spacing on webs).

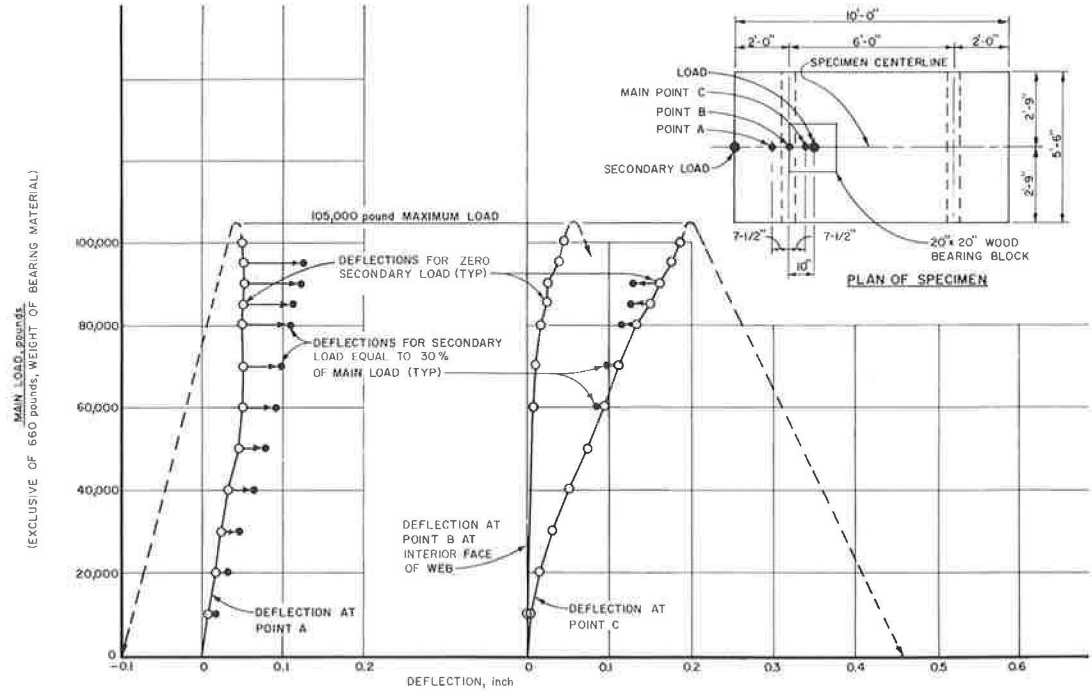


Figure 8. Specimen centerline deflections for test 2 (10-in. stud spacing on webs).

TABLE 1

VERTICAL DEFLECTIONS ALONG LONGITUDINAL LINE THROUGH POINT C[a]

Test No.	Stud Spacing (in.)	Deflections (in.)		
		At Centerline	Halfway Between Centerline and Edge	At Edge
1	4	0.160	0.147	0.094
2	10	0.163	0.139	0.085
Avg.		0.162	0.143	0.090

[a]At 90,000-lb main load and zero secondary load.

the distribution of vertical deflection along the longitudinal line through point C was as given in Table 1. It is seen that the edge deflections were about 50 to 60 percent of the deflections at the specimen centerline. The magnitude and distribution of the deflections did not appear to depend on the stud spacing.

As Figures 7 and 8 also show, the vertical movement (slip) between the steel web and the adjacent concrete at point B (the web itself) on the specimen centerline also increased progressively, but at a slower rate than the deflections at points A and C, as the load increased. At a main load of 90,000 lb, the slip at the specimen centerline was about 0.005 in. for the specimen with a 4-in. stud spacing and about 0.03 in. for

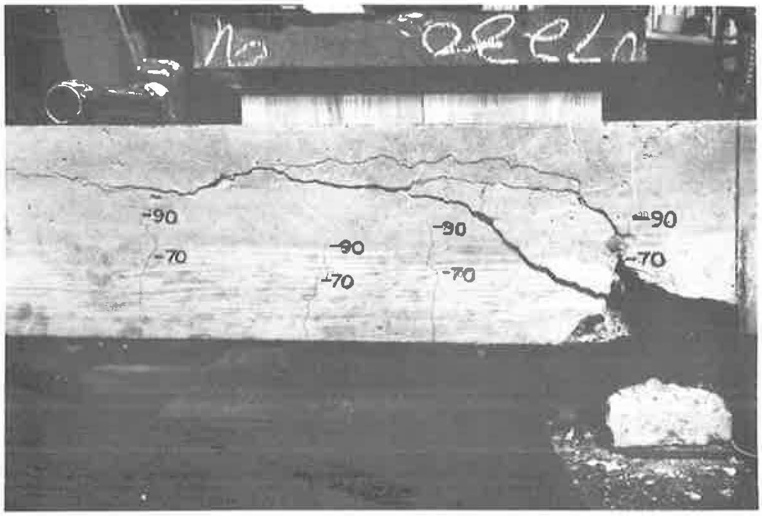


Figure 9. View of transverse face of failed test unit with 4-in. stud spacing; large failure cracks developed suddenly at main load of 99,000 lb.

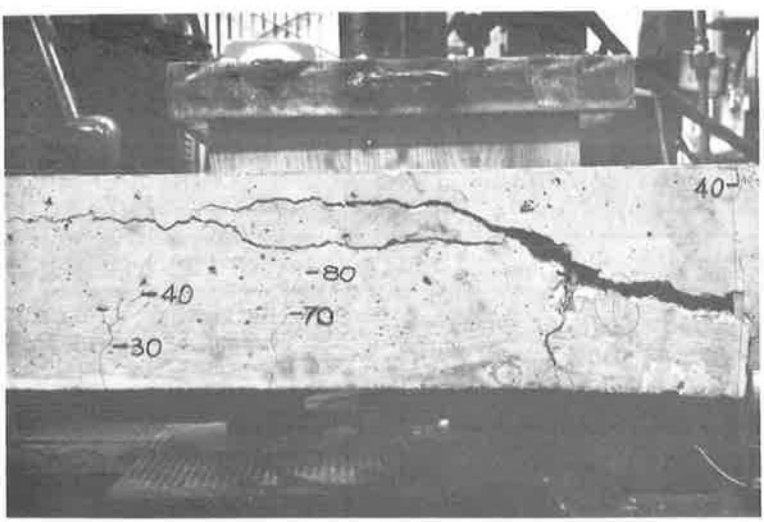


Figure 10. View of transverse face of failed test unit with 10-in. stud spacing; large failure cracks developed suddenly at main load of 105,000 lb.

the specimen with a 10-in. stud spacing. The edge slips were about 0.004 and 0.02 in., respectively, for the two specimens. It is, of course, logical that the specimen with the 10-in. stud spacing slipped more than the specimen with the 4-in. stud spacing. However, even at a 90,000-lb load, no cracking of the concrete in the vicinity of the steel web was observed in either test, and the slight slip at that load was apparently not detrimental to the specimens.

At about a 40,000-lb main load, vertical hairline cracks began to form at the bottom of the interior span and at the top of the cantilever span near the web. As the main load increased, these cracks progressed farther into the slab and extended beyond the mid-

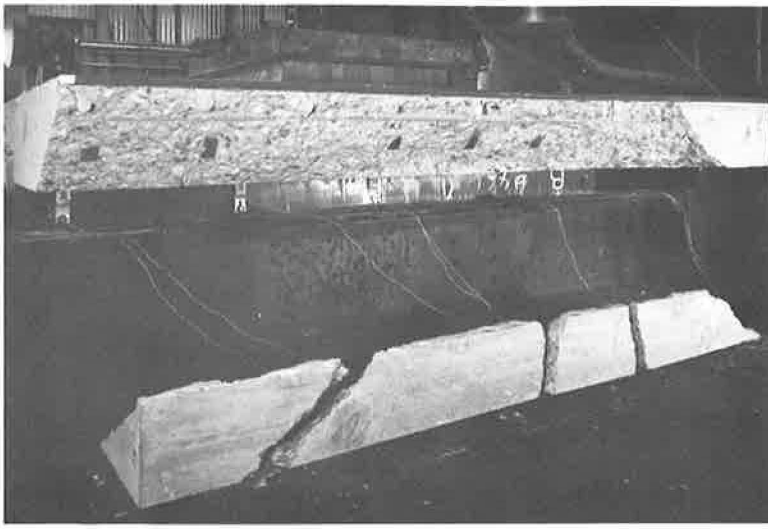
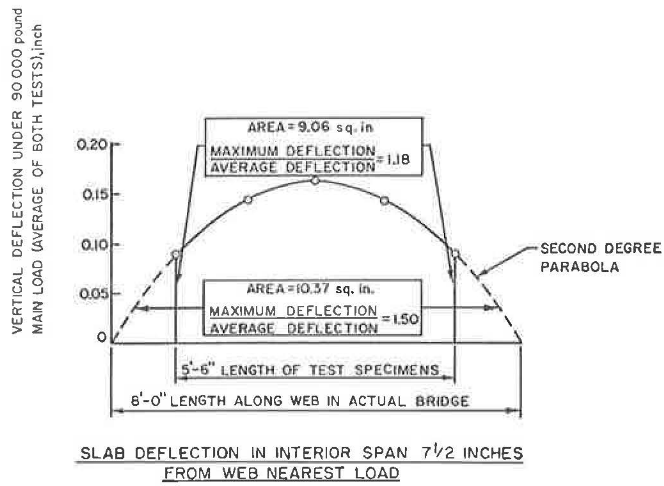


Figure 11. View of failed test unit with 10-in. stud spacing showing diagonal tension failure of the cantilever span under a secondary load of 28,500 lb; failure occurred when main load was 95,000 lb and did not affect main results of test.

thickness when the main load was 90,000 lb. The point to which a particular main load caused these cracks to progress was marked on the slab adjacent to the cracks. Failure of the specimen with the 4-in. stud spacing occurred suddenly at a main load of 99,000 lb. The specimen with the 10-in. stud spacing failed in a similar manner at a main load of 105,000 lb. As the views of the failed slabs in Figures 9 and 10 indicate, however, the main failure cracks, which formed suddenly just beyond the ends of studs under the main load and across part of the interior span, did not join the previously formed vertical hairline cracks. (The two-digit numbers adjacent to the hairline cracks shown in Figures 9 and 10 are the main-load values expressed in thousands of pounds that marked the progress of the cracks during the tests.) The 99,000-lb failure load for the specimen with the 4-in. stud spacing and the 105,000-lb failure load for the specimen with the 10-in. stud spacing were, respectively, 476 and 505 percent of the 20,800-lb AASHO design load that includes 30 percent of the live load for impact. The secondary loads were zero when the failures occurred. A diagonal tension failure, shown in Figure 11, occurred in the cantilever span of the specimen with a 10-in. stud spacing when the main load was 95,000 lb and the secondary load was 28,500 lb. This failure did not influence the strength of the interior span because the interior span did not fail until an additional main-load increment of 10,000 lb was applied. Before the formation of the main cracks, each specimen had successfully sustained a 90,000-lb main load simultaneously with a 27,000-lb secondary load. Thus, even for positions of truck wheels that would cause edge loading, the results indicated that a $5\frac{1}{2}$-ft long portion of a bridge with the test configuration could sustain at least about 430 percent of design load including impact.

As shown in Figures 9 and 10, the large failure cracks were vertical near the bottom (at the stud location), inclined at about 45 deg to the horizontal at mid-depth, and were almost horizontal near the top of the slab. Corresponding views on the opposite side of each specimen indicated the same positions of failure cracks. Because these cracks occurred approximately where the planes of maximum diagonal tension would theoretically occur, it can be concluded that failure was due to diagonal tension possibly in conjunction with some bond slippage, rather than to punching shear. This supposition appears logical because, at the failure loads, the theoretical bond stresses exceeded 800 psi, which is in the range of values of ultimate bond strengths obtained in many independently conducted flexural tests (2) on deformed bars with diameters less than or



ASSUMPTIONS			MAXIMUM UNIT SHEARS FROM TOTAL SHEAR V	
CASE	LENGTH OF WEB RESISTING VERTICAL SHEAR	VARIATION OF SHEAR INTENSITY	SHEAR PER INCH OF WEB	SHEAR PER FOOT OF WEB
I	5'-6"	DIRECTLY PROPORTIONAL TO SLAB DEFLECTIONS	$\frac{1.18\ V}{66} = \frac{V}{55.9}$	$\frac{1.18\ V}{5.5} = \frac{V}{4.66}$
II	8'-0"	DIRECTLY PROPORTIONAL TO ORDINATES OF PARABOLA	$\frac{1.50\ V}{96} = \frac{V}{64.0}$	$\frac{1.50\ V}{8.0} = \frac{V}{5.33}$

Figure 12. Distribution of vertical shear intensity assumed proportional to slab deflection.

slightly greater than the ¾-in. diameter bars used in the present specimens. The maximum diagonal tension stress that would develop from shear in a reinforced slab at the failure loads was calculated to be between about 260 and 280 psi, which, as an isolated stress, would probably not be enough to cause a diagonal tension break. However, under the loads that caused bond slippage and the main cracks, the theoretical maximum tension stress at the ends of the studs for an unreinforced slab would be over 600 psi, which is probably greater than the ultimate tensile strength of the concrete. (Calculations were made for an unreinforced slab because, if there is bond slippage, the reinforcing bars in the vicinity of the slippage are usually not very effective in helping to support a load.) Therefore, it appears that a progressive failure occurred, probably starting with bond slippage and terminating with tension failure initiated at the ends of the studs. Because bond is evidently critical, it is important that the bottom transverse reinforcing bars be sufficiently long so that the gaps between the bar ends and the beam web are small (Fig. 2).

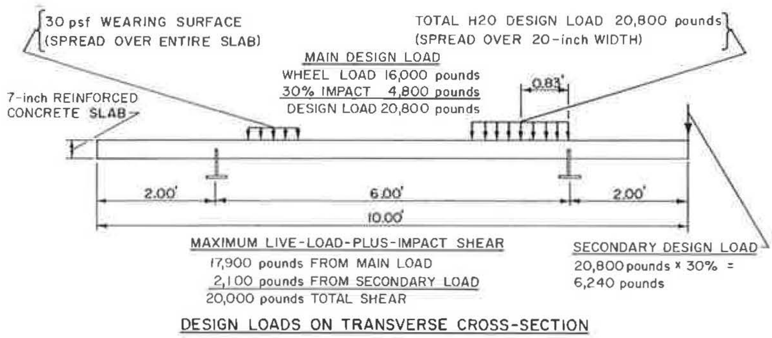
Since the failures were apparently combined tension-bond failures rather than punching-shear failures, it can be concluded that the bearing area of the top of the web in combination with the studs spaced at up to 10 in. along the web are together capable of carrying punching shears exceeding those existing at the experimental failure load. Furthermore, the occurrence of deflections at the transverse edges of the specimens indicates that a wheel load on an actual bridge would be distributed over more than a 5½-ft length. Because of this longer distribution, the resistance to punching shear in the bridge would be larger than in the tested specimens. Therefore, it is obvious that the 10-in. spacing of studs is extremely conservative for studs carrying punching shear alone, and that the spacing of studs loaded in this manner could be increased considerably without resulting in a punching type of failure under standard highway loadings.

TABLE 2

MAXIMUM INTENSITIES OF SHEAR ALONG WEB AT SPECIMEN FAILURE[a]

Test No.	Stud Spacing (in.)	Main Load (lb)	Total Shear (V) (lb)[b]	Maximum Unit Shear (V/55.9) (lb/in.)
1	4	99,000	85,000	1,520
2	10	105,000	90,400	1,620

[a]Based on case I, Figure 12.
[b]Because center of main load located 10 in. from web, maximum shear is 86.1 percent of main load.



VERTICAL SHEAR ALONG ONE SIDE OF STEEL WEB	
COMPONENT	SHEAR, pounds per inch
DEAD LOAD OF SLAB AND WEARING SURFACE	29
LIVE LOAD PLUS IMPACT (FORMULA ON FIGURE 12)	$\frac{20,000}{64.0} = 313$
TOTAL	342

SUGGESTED EXPRESSION FOR VERTICAL SHEAR PER STUD
VERTICAL SHEAR PER STUD = 342 S pounds WHERE S IS THE SPACING (IN inches) OF THE STUDS ON ONE SIDE OF THE STEEL WEB.

Figure 13. Vertical shears on studs from H20 truck loading.

To evaluate the results further and to develop design information for combined punching and horizontal shear, it was necessary to obtain an approximation of the variation of vertical shear along the web. To accomplish this, the distribution of vertical shear transferred from the slab to the beam web was assumed to be proportional to the vertical deflections of the slab near the web. Also, because stud designs are usually based on ultimate strength, it was assumed that the variation of vertical deflections under the 90,000-lb main load would be more pertinent than the variation under lesser loads. On the basis of these assumptions, the expressions for the intensity of shearing force along the web were derived (Fig. 12).

In the first group of expressions (case I) in Figure 12, it is assumed that the shear is resisted by only a 5½-ft length of web, as in the tests. The maximum intensities of shear along the web at failure of each specimen were then computed to be as given in Table 2.

It was not possible to determine from the test results how much of the shear was transferred from the slab to the web by the studs and how much was transferred by bearing on top of the webs. Because the studs deflected downward with the concrete slab at the studs, the studs were strained in bending and, therefore, supported at least part of the punching shear. If the studs had transferred all the shear, the maximum shear per stud would have been 16,200 lb for the 10-in. stud spacing, which is about 43 percent more than the maximum useful capacity of 11,300 lb specified (1) for $^3/_4$-in. diameter studs used with concrete having a compression strength of 3,710 psi. If all the shear had been transferred in bearing on the $^5/_{16}$-in. wide top of the web, the maximum bearing stress would have been 5,180 psi, which is about 40 percent more than the ultimate compression strength of the concrete, but which might not exceed the capacity of the concrete for resisting compression under the triaxial stress condition existing in the concrete over the web. It thus appears likely that both the studs and the bearing surface on top of the web participated in the transfer of vertical shear from the slab to the web.

In designing an actual bridge, however, it would be conservative and convenient to neglect the contribution of bearing on the top of the webs and to assume that all vertical punching shear, as well as horizontal shear caused by longitudinal bending, is transferred from the slab to the web by the studs. To design the studs and determine their spacing, it is, of course, necessary to know the intensity of shearing force along the web to know how much vertical shear will be applied to a given stud. The formula for determining the maximum shear intensity used here for evaluating the test data would be overly conservative for an actual bridge. Therefore, as demonstrated in Figure 12, it appears reasonable to assume that the intensity of shear would vary parabolically over at least 8 ft. This assumption is based on fitting a parabolic curve to the observed deflections. It is a conservative assumption because, in an actual bridge, the vertical deflections would not terminate abruptly within an 8-ft length but would tend to taper off more gradually.

On the assumption that the intensity of vertical shear from the wheel load varies parabolically over an 8-ft length, the second group of expressions (case II) in Figure 12 would apply. Then, as determined in Figure 13, the design maximum vertical shear per stud for H20 loading would be 342 S lb, where S is the spacing of studs in inches. To determine the maximum shear on the stud for the given spacing, this vertical shear would be added vectorially to the horizontal shear per stud, if present, and the result would be compared with the allowable shear per stud, which would be the useful capacity given in the specification (1) divided by a factor of safety.

ACKNOWLEDGMENT

Appreciation is extended to the Indiana Steel Fabricators Association for supplying the steel T's with attached studs used for the tests described herein.

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