Composite Beams with Stud Shear Connectors

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Testing of a composite beam structure consisting of a 6-in. deck slab 32 ft long and 10 ft 11 in. wide and two 18WF50 beams 32 ft long, is described. The shear connectors on one of the beams were ¾-in. diameter studs with upset heads, welded to the top flange. On the other beam, ½-in. diameter studs with end hooks at right angle were used.

The loading program consisted of 1 million cycles at 100 percent, an additional 250,000 at 125 percent, and a further 250,000 at 150 percent of the design live load. Final failure was produced by static loading. Both types of shear connectors behaved satisfactorily under all testing conditions.

On the basis of these experimental results tentative design recommendations concerning the use of stud shear connectors are made. The desirability of further experimental studies is indicated, leading ultimately to less conservative design rules.

● USE of composite beams as structural members has become generally accepted for bridge and building construction (1, 2, 3). Essential parts of such beams are the shear connectors providing integral action of the steel beam and the concrete deck slab. They have the double function of transmitting the horizontal shear between slab and beam and of tying down the slab to the beam. Presently, a great variety of different types of shear connectors is successfully used.

The newest addition is stud shear connectors consisting of round studs automatically stud welded to the top flange of the steel beams (2). The simplicity of such connectors is striking in comparison to more elaborate systems already in use. In the present paper the results of tests on a full-scale specimen subjected to extensive repetitive loading and an ultimate static load are briefly reported. On the basis of these results and a corresponding analysis, tentative design recommendations are made.

In this connection it may be important to point out that the design of any type shear connector must necessarily be based on experiments (or experience). The action of the connectors is much too complicated to be accessible to an exact stress analysis. Even the loading of a connector is rather indeterminate, as a considerable amount of shear is transmitted by bond. In case the latter should be broken by slip, mechanical friction is still able to carry part of the shear.

Furthermore, the stress distribution in the connector itself is so highly complicated that any analysis must be regarded as an approximation. It may be well to remember that the design of other connections (for example, riveted or welded connections) is also fundamentally experimental. The allowable shear, bearing, and pitch values, or the allowable stresses of butt and fillet welds, were derived from experience and experiments.

Essentially two possibilities of failure must be considered:

1. In bridge structures the shear acting on the shear connectors is primarily produced by live loads; hence, it is repetitive, so that the possibility of a fatigue failure exists. Even in cases where the steel beams are temporarily supported during the construction of the slab, the dead load stresses in the slab, hence the shear between slab and beam, are greatly reduced by shrinkage and creep. It is therefore essential to investigate the fatigue behavior of a shear connector. Because the actual state of stress in and around a connector is practically inaccessible to an exact analysis, only testing of connectors in actual size and under actual action will furnish conclusive results.

2. Concerning the possibility of failure under static loading, it is desirable that failure not occur in the shear connectors but rather by general yielding of the steel beam. In the latter case no sudden failure will occur. Hence, the shear connection should be able to withstand the shear under ultimate load producing failure by yielding of the steel beam and eventual crushing of the concrete deck slab.

The tests were planned primarily to get experimental information on the behavior of ½-in. and ¾-in. diameter stud shear connectors in fatigue. As no fatigue failure developed after a considerable number of overload cycles the ultimate load under static conditions was determined. It is believed that the experimental information is sufficient to recommend conservative design values for the ½-in. studs.

TEST SPECIMEN, SET-UP, AND INSTRUMENTATION

The test specimen (Fig. 1) was built according to a design prepared by the Bureau of Public Roads. It consisted of two 32-ft long, 18WF50 beams (A-7 steel), placed 6 ft apart and a deck slab 6 in. thick and 10 ft 11 in. wide (specified concrete strength f'_c = 3,000 psi). Three crossbeams, 16WF40, placed 1 ft from each end and at mid-length were bolted to the longitudinal beams by high-tensile bolts. The reinforcement of the slab (deformed bars A-305 steel) is also indicated in Figure 1.

One of the beams, referred to as north beam, carried pairs of ¾-in. diameter K S M solid fluxed shear connectors with upset heads, 4 in. long, at a constant spacing of 11½-in. The south beam was provided with rows of three ½-in. diameter K S M solid fluxed L-connectors having a height of 1⅞ in. and a short hook bent at right angle. The spacing was kept constant at 14 in. The stud connectors were welded by the manufacturer's personnel with their equipment.

No special preparation was given the beam before welding. The slab was poured without temporary support of the steel beams, that they carried the entire dead load. Figure 2 shows the slab during pouring.

For testing, the new installation for testing of large assemblies under static and fatigue loading (5) at Fritz Engineering Laboratory, Lehigh University, was used. The specimen was simply supported over a span of 30 ft with supports under each steel beam. Identical loads were applied at mid-span by two hydraulic jacks acting directly above the two steel beams. For cyclic loading the jacks were connected to an Amsler pulsator producing a cyclic, sinusoidally varying load at 250 cycles per minute. In the static test for ultimate load the Amsler pendulum dynamometer applied and measured the pressure in the jacks. The test set-up is shown schematically in Figure 3. Figure 4 shows a picture of the specimen under cyclic loading.

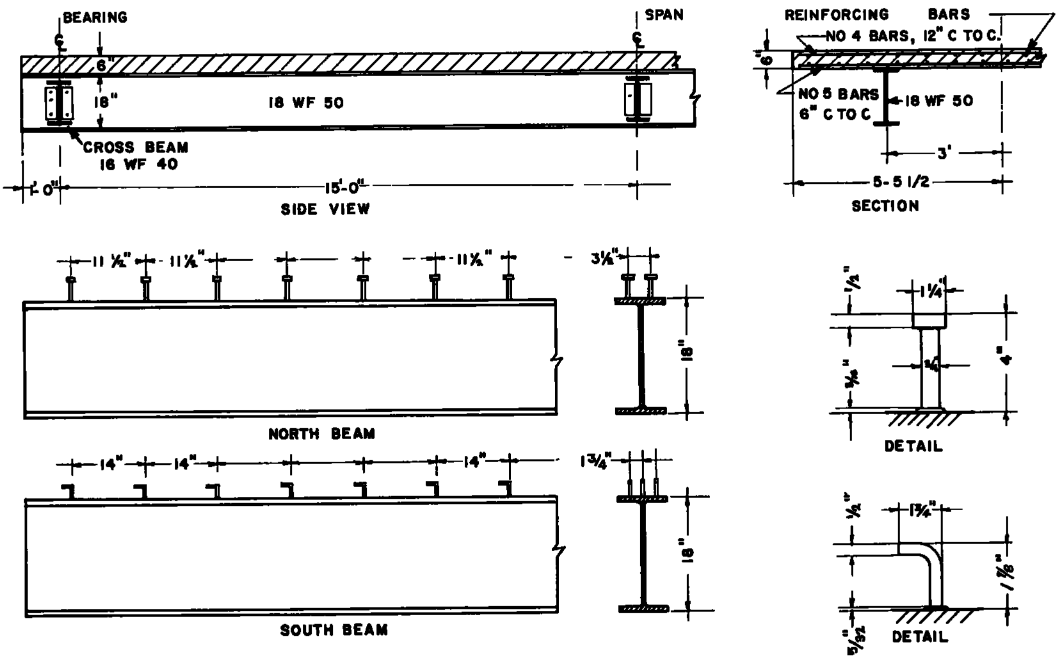


Figure 1. Test specimen design detail.

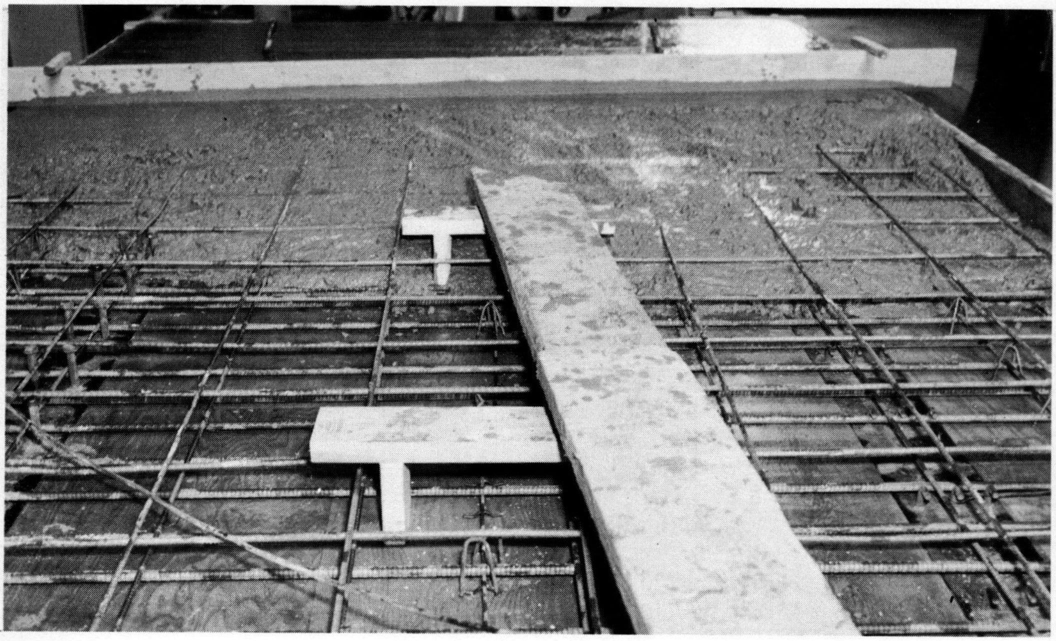


Figure 2.

The instrumentation was intentionally concentrated on a few essential measurements. Ames dials were mounted at the four ends of the beams to measure the relative movement slip between beam and slab. Deflection measurements were taken by means of a level instrument reading 0.001-in. scales attached to the steel beams over the supports and at mid-span.

On each steel beam two SR-4 electrical strain gages were mounted 9 in. from the center (in order not to interfere with the connection of the cross member). One gage was placed in the middle of the bottom flange, the other 16 in. above the bottom flange on the outside of the web (Fig. 5). Readings of the gages were recorded graphically by a "Brush" oscillograph.

PROGRAM, TESTS, AND RESULTS

The test program comprised the following parts:

1. 1,000,000 load cycles alternating between a minimum of 3,700 and a maximum of 33,800 lb (total for both jacks). The maximum load corresponded to the design live load.
2. 300,000 cycles between 4,000 and 42,200 lb. The maximum load corresponded to 125 percent of the design live load.
3. 250,000 cycles between 6,000 and 50,700 lb, the latter being equal to 150 percent of the design live load.
4. Destruction test under static loading.
5. Auxiliary tests for determining the tensile strength of coupons from the 18WF50 beams, the tensile and shear strength of the stud shear connectors, and the cylinder strength of the concrete in the deck slab.

The results of the auxiliary tests are summarized in Tables A to D in the Appendix. The static yield stress indicated in Table A was determined on the tensile coupons at practically zero loading rate within the yield level between the yield point and strain hardening. This value is much more representative for computation of the ultimate load than the upper yield point obtained at a considerable loading rate (for example in mill tests). The double shear tests on the studs reported in Table D produced shear failure on the side opposite to the weld, hence showing a strength of the weld superior to the base material.

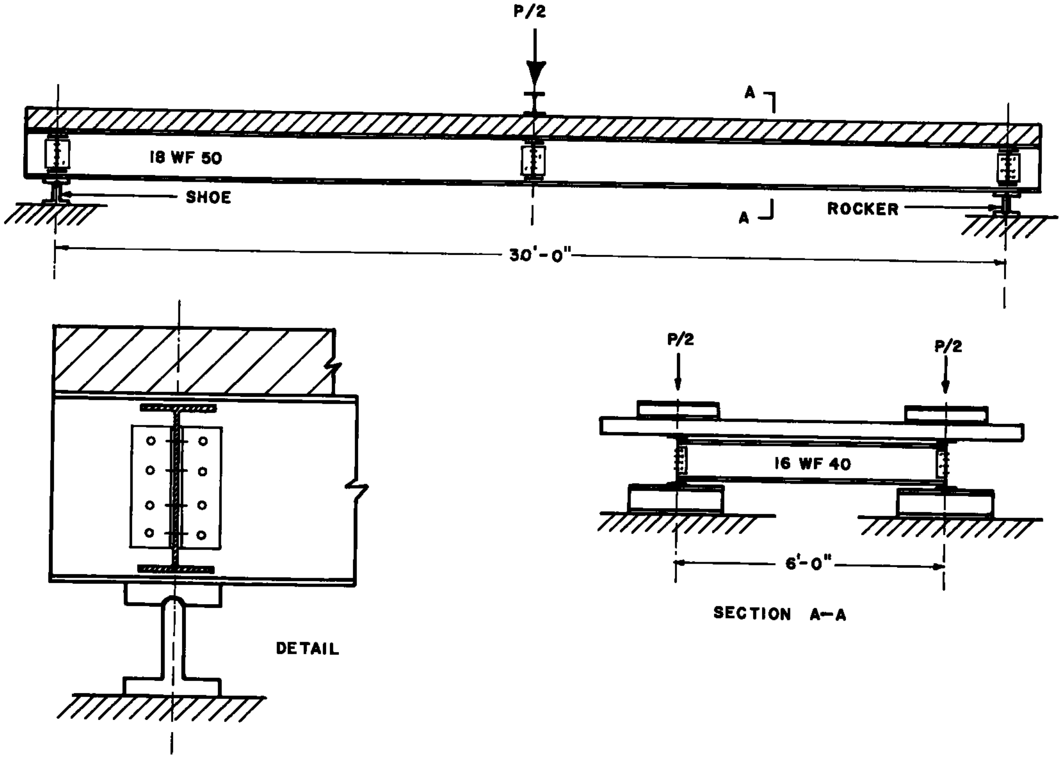


Figure 3. Schematic test set-up.

TABLE 1
INFLUENCE OF REPETITIVE LOADING ON DEFLECTIONS AND SLIP

TEST NO	AGE OF SPECIMEN (IN DAYS)	LOADING	LOADS (IN POUNDS)		C_L DEFLECTION (IN INCHES)		END SLIPS (IN INCHES X 10^4)			
							NORTH BEAM 3/4 INCH STUDS		SOUTH BEAM 1/2 INCH STUDS	
			MINIMUM	MAXIMUM	MEASURED	THEORETICAL(a)	EAST END	WEST END	EAST END	WEST END
1	30	Static	—	33,800	0.235	0.222	0	0	0	2 b
2	33 to 35	1,002,000 Cycles	3,700	33,800	—	—	—	—	—	—
3	36	Static	—	33,800	0.215	0.222	0	0	2	0
4	36	Static	—	44,000	0.280	0.289	0	0	4 b	3
5	36 to 37	296,500 Cycles	4,000	42,200	—	—	—	—	—	—
6	37	Static	—	44,000	0.270	0.289	0	0	2	3
7	41	Static	—	52,000	—	—	0	0	4	6
8	41 to 42	256,800 Cycles	6,000	50,700	—	0.341	—	—	—	—
9	47	Static	—	49,800	0.330	0.327	0	0	4 c	10 c

Notes (a) With $E_s = 30 \cdot 10^6$ psi, $n = 10$
(b) Not recovered
(c) Recovery not checked

The fatigue tests on the composite beam specimen are summarized in Table 1. The load, P, was applied in equal parts, $\frac{P}{2}$, over each steel beam at mid-span (Fig. 3). In order to study the influence of the repetitive loading on the behavior of the specimen—and especially on the deflection and possible slip between steel beam and concrete deck—static loading tests were interposed between the different phases of the fatigue loading program. A total of nine tests were performed in the following sequence (see Table 1):

Test 1 was a static loading test up to the design live load, P = 33,800 lb, producing together with the dead load stresses a computed maximum fiber stress of 18,000 psi. The measured deflection at mid-span and the slip at the four ends of the beams are given in Table 1. The difference in deflection between the north and south beam was less than 0.003 in. and never exceeded this value in the subsequent static tests (tests 3, 4, 6, 7, and 9). As the loads applied to the beams were identical at any time, it can be concluded that no interaction took place between the north and south beams.

Test 2 consisted of 1,002,000 cycles between a minimum load, P_{min} = 3,700 lb, and a maximum load of P_{max} = 33,800 lb. These loads were the actual effective loads, taking into account the influence of the inertia forces. It amounted under the given conditions to 16 percent of the load amplitude. During testing no unusual observations were made.

The subsequent static test (test 3) indicated that 1,000,000 repetitions of full live load did not break the bond or produce any significant slip. The decrease in deflection from tests 1 to test 3 is rather remarkable, indicating an increase in the bending stiffness of the specimen. It is probably attributable to the increase of the modulus of elasticity of the concrete within the six days between test 1 and 3.

An additional 296,500 cycles at 125 percent live load (test 6) and 256,800 cycles at 150 percent live load (test 8) produced neither fatigue failure nor important slip. Concerning a comparison between the slip data for the north beam with ¾-in. studs, showing absolutely no slip, and the south beam with ½-in. studs, exhibiting a maximum slip of 0.001 in., it is shown subsequently that the shear area provided by the studs on the south beam was considerably less than that on the north beam.

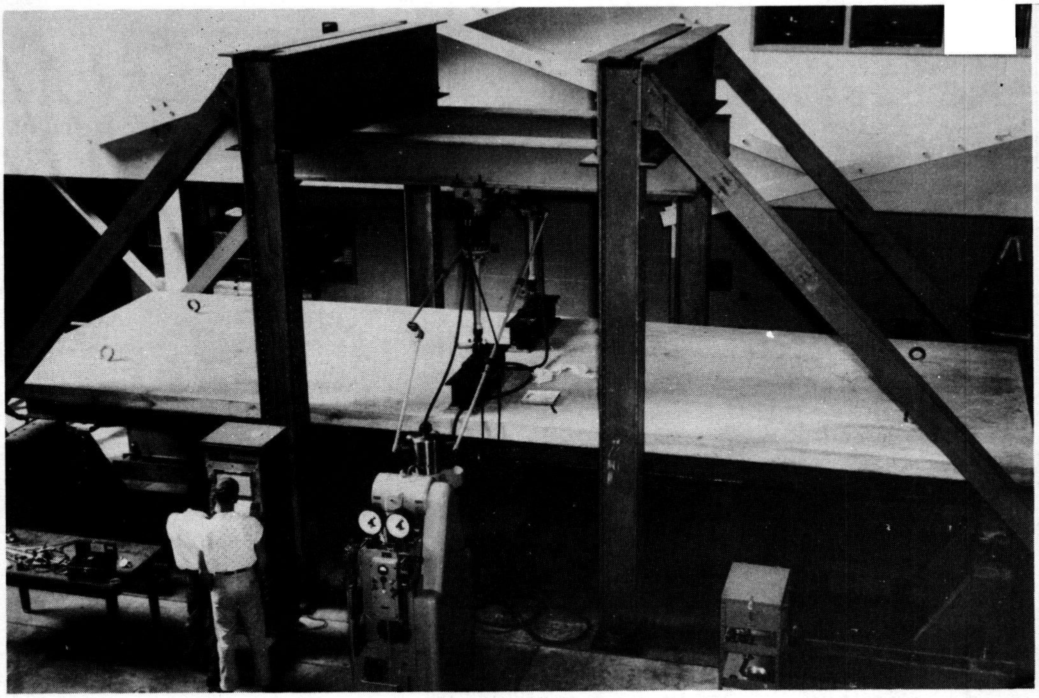


Figure 4.

TABLE 2
SECTION PROPERTIES AND LOADING

Steel Beams: Pair of 18 WF50

Handbook properties

Area	A_s	=	2 x 14.71	=	29.42 in.^{2}
Moment of Inertia	I_s	=	2 x 800.6	=	1601.2 in.^{3}
Section modulus	S_s	=	2 x 89.0	=	178.0 in.^{4}

Deck Slab:

Width	b	=	131 in.
Thickness	t	=	6 in.
Area	A_c	=	786 in.^{2}

Composite Section:

Reduced area	A	=	108.02 in.^{2}
Reduced moment of inertia	I	=	4912.0 in.^{4}
Neutral axis, from top	x	=	6.26 in.
Distance to top concrete fiber	c_c	=	- 6.26 in.
Distance to top steel flange	c_u	=	- 0.26 in.
Distance to bottom steel flange	c_L	=	17.74 in.
Statical moment of steel area about N.A.	m	=	257.2 in.^{3}

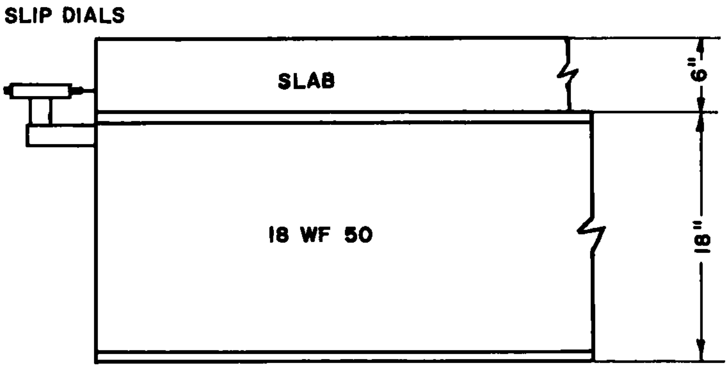
Loading:

Dead load, distributed	w	=	920 lb/ft
Design live load concentrated at mid-span	P	=	33,800 lb

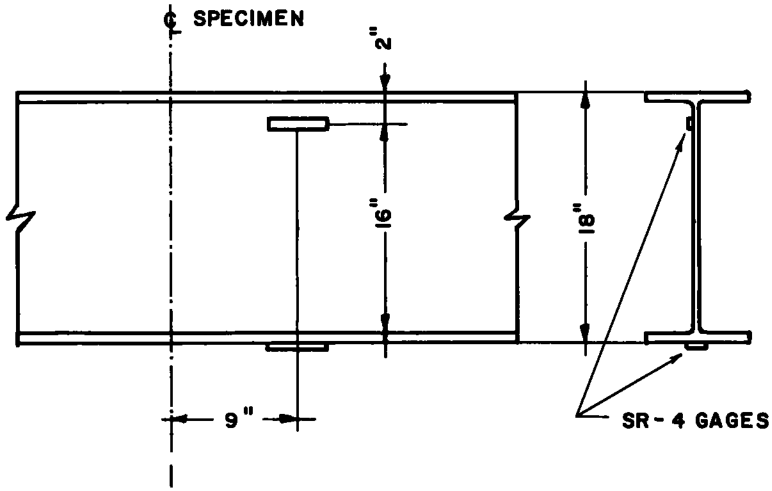
Test 9, the initial phase of the final static destruction test, was performed in two steps. At a slab age of 47 days the specimen was loaded to a maximum of 145,600 lb with a corresponding deflection of 4.15 in. Ten days later the specimen was brought up to the ultimate load. Figure 6b shows mid-span deflection of the south beam. At deflections of 4.15 and 6.90 in., respectively, the specimen was completely unloaded in order to reset the loading jacks. This is shown in Figure 6b by the unloading curves resulting in permanent sets of 3.00 and 5.50 in., respectively. Also indicated are the following points: Working load first yielding observed from SR-4 readings, first yielding determined from flaking of mill scale, first cracks in concrete slab. The maximum observed load was 172,800 lb, or 5.1 times the design live load, resulting in a mid-span deflection of the south beam of 6.93 in. For comparison the load limits under cyclic loading are shown in Figure 6a.

Figure 7 shows the specimen at ultimate load. The high curvature of the steel beams at mid-span forced the concrete slab to separate from the beam. This was especially pronounced on the south beam, resulting in actual clearance between beam and slab. However, at the locations of the studs the slab was held down effectively. Final failure was brought about by crushing of the concrete slab. The slip measurements are shown in Figure 8. Up to the theoretical yield load, $P = 80,100$ lb, no slip exceeded 0.0025 in.

Again, it should be stressed that the north beam, with the ¾-in. studs, showing practically no slip, had considerable more shearing area than the south beam with ½-in. studs. Therefore, no direct conclusions as to the influence of stud diameter on the slip should be made from comparison of the slip curves in Figure 8, but an interpretation should be made in the light of the analytical results given in following sections. Important slip on the south beam was setting in at a load of about 120,000 lb, corresponding to



a. SLIP DIALS (1/1000 IN.) AT ALL BEAM ENDS



b. SR-4 GAGES ON EACH BEAM

Figure 5. Instrumentation.

about 3.5 times the maximum design live load. At ultimate load the end slips took the following values:

Beam	Studs	Slip, in. West End	Slip, in. East End	Slip, in. Average
South	½ in.	0.0458	0.0634	0.0546
North	¾ in.	0.0108	0.0040	0.0074

Finally, Figure 9 shows the SR-4 strain gage readings. It can be seen that yielding in the bottom flanges occurred at loads of about 90,000 and 120,000 lb for the south and north beam, respectively. Yielding reached the upper gages, 16 in. above bottom flange, at approximately 135,000 lb. An interpretation of the test results becomes only meaningful in conjunction with an analysis. Such a procedure is absolutely necessary, especially if any valid extrapolation beyond the specific conditions of the test is contemplated. Therefore, a short analysis and interpretation of the results is presented in the following.

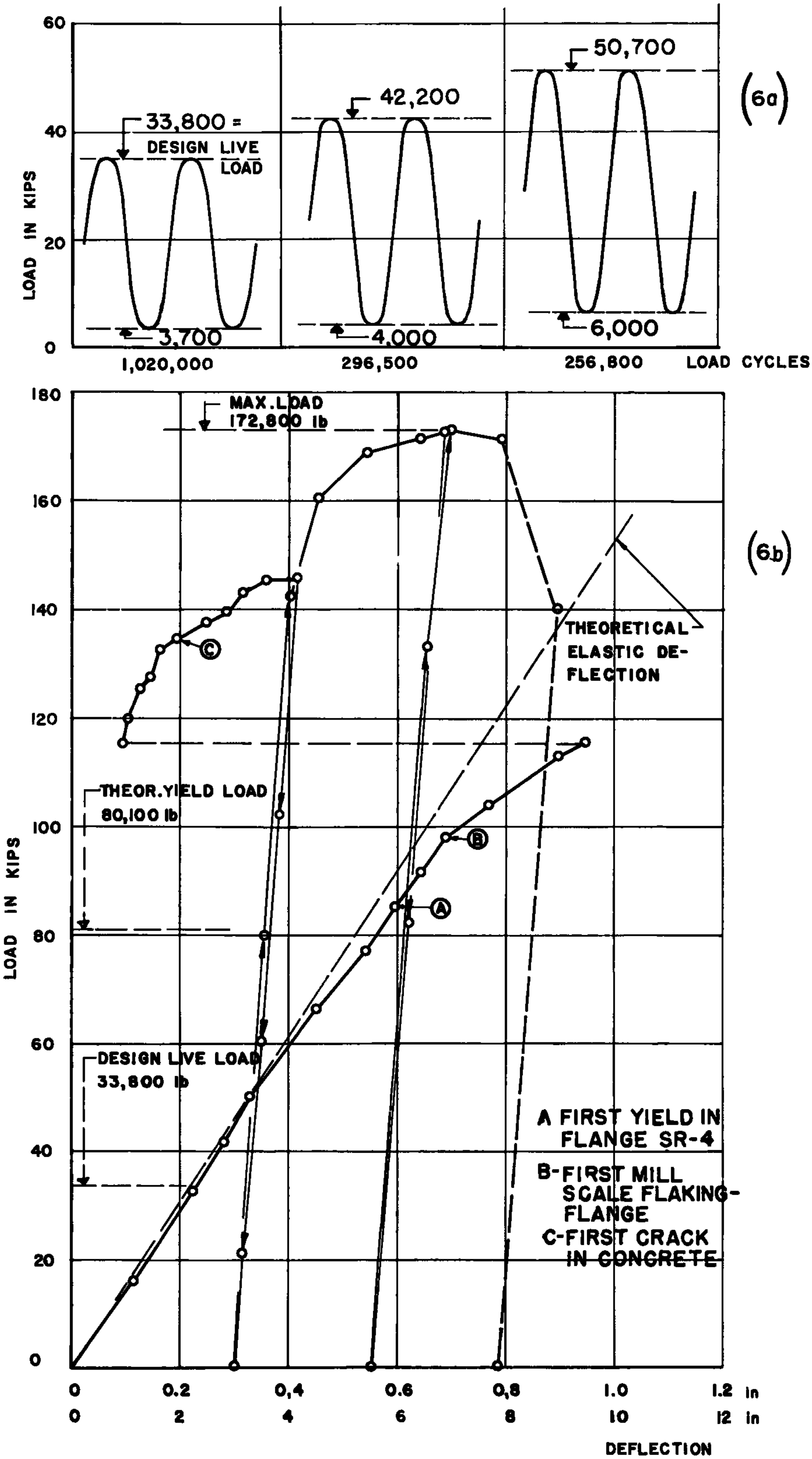


Figure 6. (a) Load limits under cyclic loading; (b) Measured center deflection.

ANALYSIS AND INTERPRETATION OF THE RESULTS

The analysis is based on the following generally accepted assumptions:

In the elastic range:
1. Complete interaction between steel beam and concrete deck; that is, no slip.
2. Modular ratio between steel and concrete, n = 10.
3. All the horizontal shear between slab and beam is transmitted by the studs only.
4. The entire width of the slab is fully effective.

In the plastic range:
5. At ultimate load the steel beam is fully yielded.
6. The compressive stress block in the concrete slab is rectangular in shape.
7. · Concrete takes no tension.

The elastic stress and deflection calculations are based on the method of transformed section. In the present case the transformation is made with the steel modulus, E_S, as a basis, hence reducing the concrete section by the modular ratio. Table 2 summarizes the section properties and the loading. The dead load and live load moment are given by:

$$M_{DL} = \frac{wL^2}{8} = \frac{920x30^2}{8} = 103,400 \text{ ft-lb} \tag{1}$$

$$M_{LL} = \frac{PL}{4} = \frac{33,800x30}{4} = 253,500 \text{ ft-lb} \tag{2}$$

Keeping in mind that the dead load is entirely carried by the steel beams (no temporary supports), the dead load stresses are:

$$f_{DL} = \frac{M_{DL}}{S_S} = \frac{103,400x12}{178} = \pm\ 6,980 \text{ psi} \tag{3}$$

The negative sign corresponds to compression in the top flange of the steel beam, the positive sign to tension in the bottom flange.

The live load produces stresses in the composite sections as follows:

$$f_{LL} = \frac{M_{LL}c}{I} \tag{4a}$$

Top flange of steel beams, with $c = c_u = -0.26$ in.

$$f_{LL} = \frac{253,500x12(-0.26)}{4,912} = -161 \text{ psi} \tag{4b}$$

Bottom flange of steel beam, with $c = c_L = 17.74$ in.

$$f_{LL} = \frac{253,500x12x17.74}{4,912} = 10,980 \text{ psi} \tag{4c}$$

The concrete stress is obtained from Eq. 4a by dividing with the modulus ratio, n = 10.

Maximum concrete stress, with $c = c_c = -6.26$ in.

$$f_{LL} = \frac{253,500x12\ (-6.26)}{4,912} \times \frac{1}{10} = -388 \text{ psi} \tag{4d}$$

The horizontal shear force, S, per unit length between one steel beam and the concrete slab is determined from:

$$S = \tfrac{1}{2} \times \frac{V\ m}{I} \tag{5a}$$

with V being the total vertical shearing force at the section under investigation and in the statical moment of the steel area about the neutral axis. The factor $\frac{1}{2}$ attributes half of the total horizontal shear to each of the two beams. Concerning the value of V,

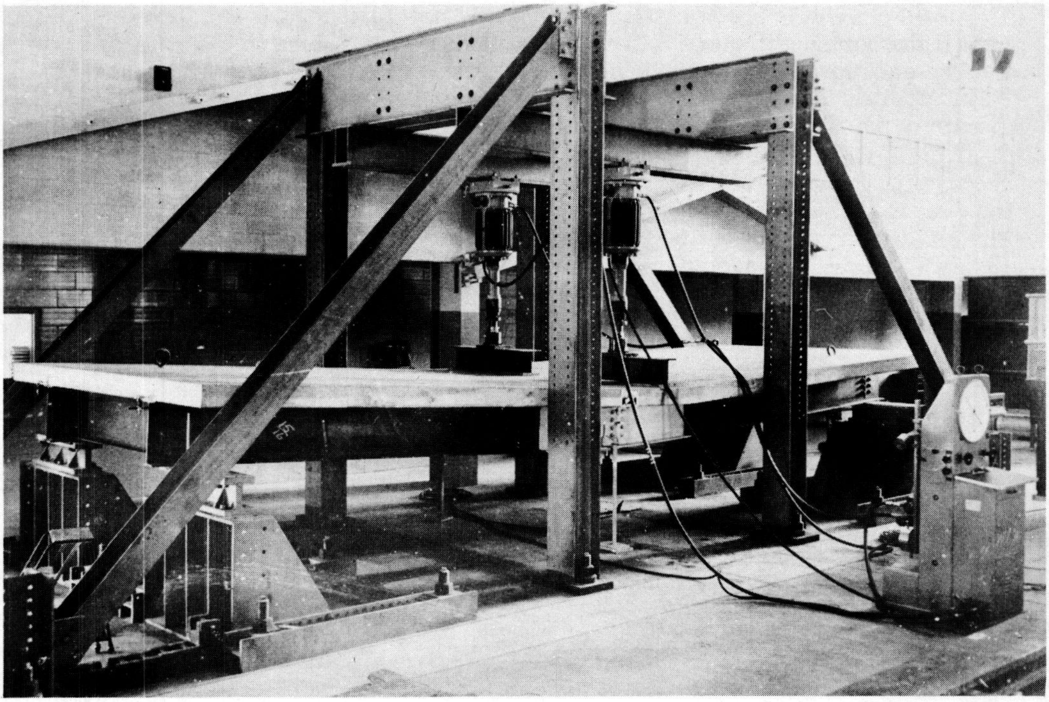


Figure 7.

dead load did not produce any shear. The live load shear was uniform over one-half the span length, with a change equal to P at the loading point.

$$V_{LL} = \frac{P}{2} = 16,900 \text{ lb} \tag{6}$$

Hence,

$$S = \frac{1}{2} \times \frac{16,900 \times 257.2}{4,912} = 442 \text{ lb/in.} \tag{5b}$$

It may be added that this shear is acting on the top flange of the steel beam in an outward direction from mid-span. Following the assumption that all shear is transmitted by the studs exclusively, the force Q per connector can readily be calculated. If the spacing is s and the number of shear connectors in one row is n,

$$Q = \frac{S\,s}{n} \tag{7}$$

Applying Eq. 7 to the present case gives:

$\frac{3}{4}$-in. studs (north beam): $s = 11.5$ in.
$n = 2$

$$Q = \frac{442 \times 11.5}{2} = 2,540 \text{ lb} \tag{8}$$

½-in. studs (south beam): $s = 14$ in.
$n = 3$

$$Q = \frac{442 \times 14}{3} = 2{,}060 \text{ lb} \tag{9}$$

It should be emphasized that under actual conditions the connector forces are considerably smaller, as most of the shear is transmitted by bond or mechanical friction. The same applies also to riveted connections, where under working conditions considerable shear is transmitted by friction between the plates, created by the clamping action of the rivets. However, the computation is by no means meaningless; it presents an index of comparison and can also be used for design, as shown later.

The average shearing stress in a stud follows:

$$v_{st} = \frac{Q}{A_{st}} \tag{10}$$

where A_{st} is the cross-sectional area of one stud. Again, v_{st} should be taken as a nominally computed stress. For the test specimens the corresponding values are:

¾-in. studs: $A_{st} = 0.441$ sq in.

$$v_{st} = \frac{2{,}540}{0.441} = 5{,}750 \text{ psi}$$

½-in. stud: $A_{st} = 0.196$ sq in.

$$v_{st} = \frac{2{,}060}{0.196} = 10{,}500 \text{ psi}$$

TABLE 3

SUMMARY OF THEORETICAL STRESS COMPUTATION

NO	LOADING	CONCRETE STRESSES		STEEL STRESSES		CONNECTORS			
						3/4 INCH STUDS		1/2 INCH STUDS	
		TOP (in psi)	BOTTOM (in psi)	TOP FLANGE (in psi)	BOTTOM FLANGE (in psi)	FORCE PER STUD (in lbs)	AVERAGE SHEARING STRESS (in psi)	FORCE PER STUD (in lbs)	AVERAGE SHEARING STRESS (in psi)
(1)	Dead Load W = 920 lbs/ft	—	—	−6,980	−6,980	—	—	—	—
(2)	Design Live Load P = 33,800 lbs	−388	−16	−161	+10,980	2,540	5,750	2,060	10,500
(3)	125% Live Load P = 42,200 lbs	−485	−20	−201	+13,710	3,180	7,190	2,580	13,100
(4)	150% Live Load P = 50,700 Lbs	−582	−24	−242	+16,480	3,810	8,620	3,090	15,700
(5)	DL+LL=(1)+(2)	−388	−16	−7,141	+17,960	2,540	5,750	2,060	10,500
(6)	DL+125%LL=(1)+(3)	−485	−24	−7,181	+20,690	3,180	7,190	2,580	13,100
(7)	DL+150%LL=(1)+(4)	−582	−24	−7,222	+23,460	3,810	8,620	3,090	15,700

It is now apparent that the ½-in. studs on the south beam were under much more severe loading than the ¾-in. studs on the north beam, regardless of the exact values of the connector forces, Q.

For the higher live loads of 125 and 150 percent, the same equations apply, as the specimen is still within the elastic range. The corresponding results are summarized in Table 3.

The mid-span deflection, y, due to live load is readily computed from

$$y = \frac{PL^3}{48\, E_s I} \tag{11}$$

with $E_s = 30 \times 10^6$ psi being the modulus of elasticity of steel. Therefore,

$$y = \frac{33{,}800 \times 30^3 \times 12^3}{48 \times 30 \times 10^6 \times 4{,}912} = 0.222 \text{ in.} \tag{12}$$

At ultimate (maximum) load, consideration must be given to the inelastic behavior of steel and concrete. The following analytical considerations are based on an assumed stress distribution over a cross-section as shown in Figure 10. With the entire steel section yielded, the maximum resultant tensile force, T, is determined in magnitude and location at the centroid of the steel section.

$$T = f_y A_s = 38{,}700 \times 29.42 = 1{,}140{,}000 \text{ lb} \tag{13}$$

where f_y = 38,700 psi is the average static yield stress of the beam and A_s the cross-sectional area of the two WF beams. The resultant compressive force, C, in the con-

TABLE 4

COMPARISON BETWEEN BEHAVIOR OF ¾ INCH STUDS AND ½ INCH STUDS

NO	APPLIED SHEAR V (in lbs)	MEASURED END SLIP (IN INCHES X 10⁴) NORTH BEAM 3/4 INCH STUDS EAST END	NORTH BEAM 3/4 INCH STUDS WEST END	SOUTH BEAM 1/2 INCH STUDS EAST END	SOUTH BEAM 1/2 INCH STUDS WEST END	AVERAGE SLIP (IN INCHES X 10⁴) 3/4 INCH STUDS	1/2 INCH STUDS	COMPUTED VALUES 3/4 INCH STUDS FORCE Q PER CONNECTOR (in lbs)	3/4 INCH STUDS AVERAGE SHEARING STRESS V_{st} (in psi)	1/2 INCH STUDS FORCE Q PER CONNECTOR (in lbs)	1/2 INCH STUDS AVERAGE SHEARING STRESS V_{st} (in psi)
(1)	0	0	0	0	0	0	0	0	0	0	0
(2)	7990	0	0	0	0	0	0	1,200	2,710	971	4,960
(3)	16250	0	0	2	6	0	4	2,440	5,540	1,980	10,100
(4)	20800	0	0	4	8	0	6	3,140	7,080	2,530	12,930
(5)	24900	0	0	4	10	0	7	3,740	8,470	3,030	15,480
(6)	33300	0	0	4	12	0	8	4,980	11,280	4,030	20,600
(7)	38600	0	0	8	22	0	15	5,810	13,180	4,700	24,000
(8)	42600	0	4	12	26	2	19	6,710	14,500	5,200	26,550
(9)	52000	0	8	20	38	4	29	7,830	17,700	6,340	32,400
(10)	60000	0	16	30	50	8	40	9,040	20,400	7,310	37,400
(11)	66400	0	26	46	70	13	58	10,000	22,600	8,100	41,300
(12)	68800	6	32	82	102	19	92	10,320	23,400	8,370	42,700
(13)	72600	12	40	150	172	30	161	10,920	24,750	8,850	45,100
(14)	72800	18	56	244	234	37	239	10,980	24,800	8,880	45,300

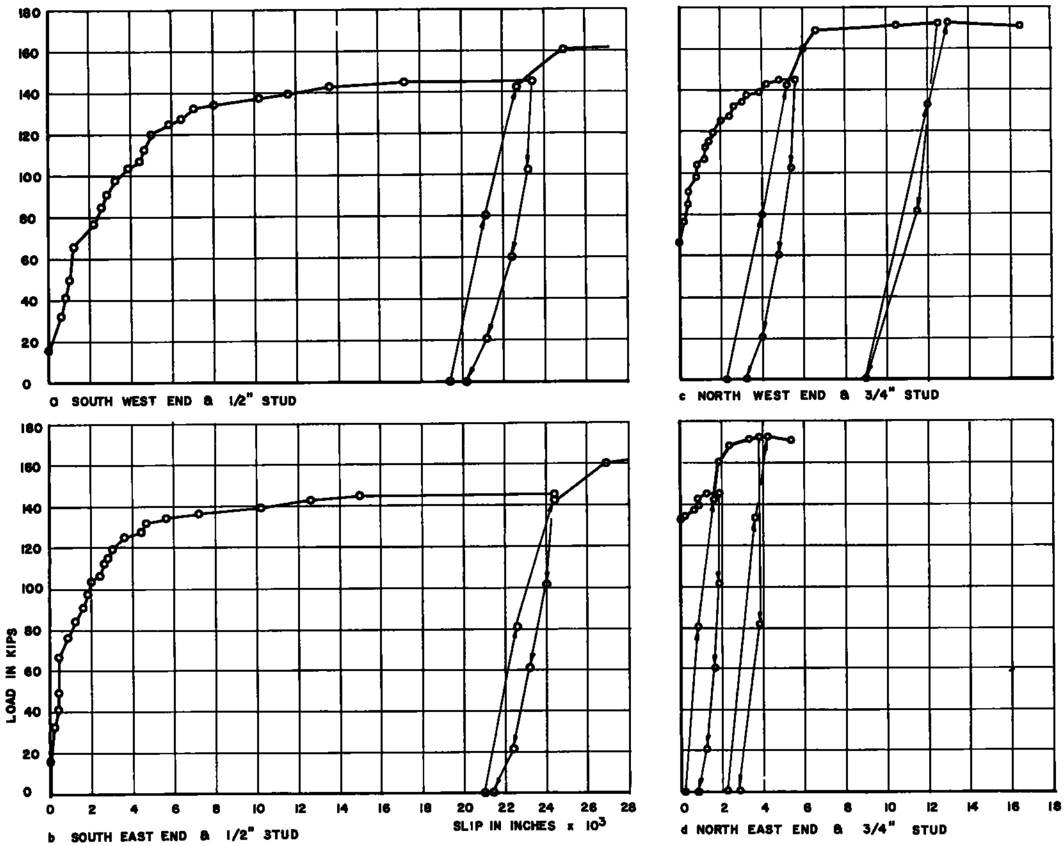


Figure 8. Measured end slips.

crete is equal but opposite to T. It is also determined by the slab width, the cylinder strength of the concrete, and the depth of the rectangular stress block; hence,

$$C = b f_c x$$

with b = slab width, f_c = cylinder strength of concrete, and x = depth of rectangular stress block (see Fig. 10).

With the pertinent numerical values the depth is

$$x = \frac{T}{b f_c} = \frac{1,140,000}{131 \times 3,930} = 2.22 \text{ in.} \tag{14}$$

Having x, the distance d between T and C is $d = 9 + 6 - \frac{2.22}{2} = 13.89$ in.

The ultimate moment is

$$M_{ult} = d\,T = 13.89 \times 1,140,000 = 15,800,000 \text{ in/lb} \tag{15}$$

and

$$P_{ult} = \frac{M_{ult}}{L} = \frac{4 \times 15,800,000}{30 \times 12} = 175,800 \text{ lb} \tag{16}$$

For the computation of the horizontal shear force, S, per unit length of one steel beam Eq. 5a still applies at the ends of the specimen where the stresses are within the elastic range. However, near mid-span this equation would give much too high results.

An average S can be computed by considering equilibrium of one-half the concrete slab as a free body, as shown in Figure 10.

Then

$$S = \tfrac{1}{2}\,\frac{C}{L/2} = \tfrac{1}{2} \times \frac{1,140,000}{16x12} = 2,970 \text{ lb/in.} \qquad (17)$$

Again, the factor ½ distributes the horizontal shear evenly to the two beams. The actually measured ultimate load, P = 172,800 lb, coincided closely with the predicted value given by Eq. 16. Assuming that the distance d was at its computed value, the actual resultant compressive force in the concrete was

$$C = \frac{172,800}{175,800} \times 1,140,000 = 1,120,000 \text{ lb}$$

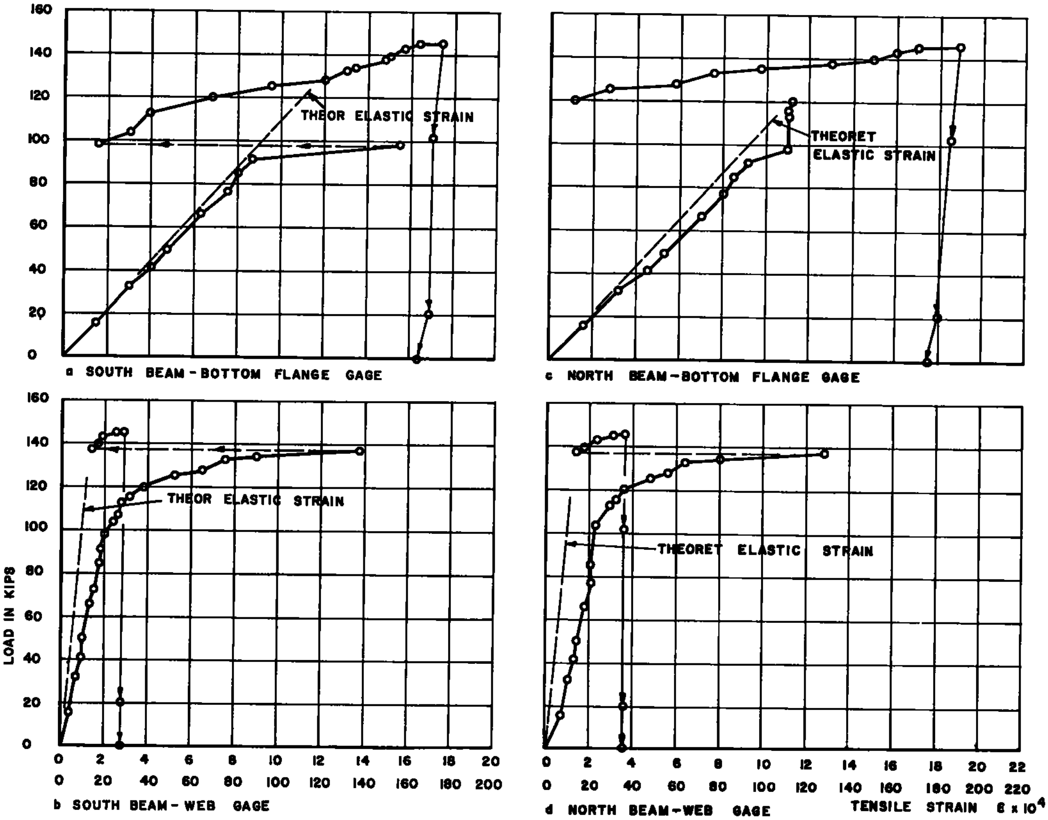


Figure 9. Measured strains.

and the corresponding average shear force in the test

$$S = \tfrac{1}{2} \times \frac{1,120,000}{16x20} = 2,915 \text{ lb/in.}$$

Using this latter value the connector forces and the average shearing stress in each connector at ultimate load can be computed from Eqs. 7 and 10.

$$\tfrac{3}{4}\text{-in. studs:} \qquad Q = \frac{2,915x11.5}{2} = 16,770 \text{ lb}$$

$$v_{st} = \frac{16,770}{0.441} = 38,000 \text{ psi}$$

$$½\text{-in. studs:} \quad Q = \frac{2{,}915 \times 14}{3} = 13{,}600 \text{ lb}$$

$$v_{st} = \frac{13{,}600}{0.196} = 69{,}400 \text{ psi}$$

Once more it is stressed that for instance the average shearing stress v_{st} = 69,400 psi represents a nominal value, the actual maximum shearing stress being considerably smaller because in the analysis the frictional resistance between steel beams and slab was neglected. Nevertheless this hypothetical value is not useless. With proper interpretation it allows the derivation of safe design values.

Comparison of the theoretical results for deflections and bending stresses of the steel beams with the corresponding experimental results are made in Figures 6 and 9, which in general show fair agreement. Keeping in mind the local influence of the loads on the strain distribution in their immediate neighborhood, the SR-4 readings are in satisfactory agreement. Even under cyclic loading these readings were in fair correspondence with the theoretically computed values. Figure 11 compares an oscillogram taken at 125 percent live load (P_{max} = 42,200 lb, P_{min} = 4,000 lb) with theoretically computed values.

No attempt was made to measure the strains in and around the studs. Besides the fact that such measurements would probably offer extreme difficulties, it is believed that they would not be of much value in arriving at design recommendations. There are

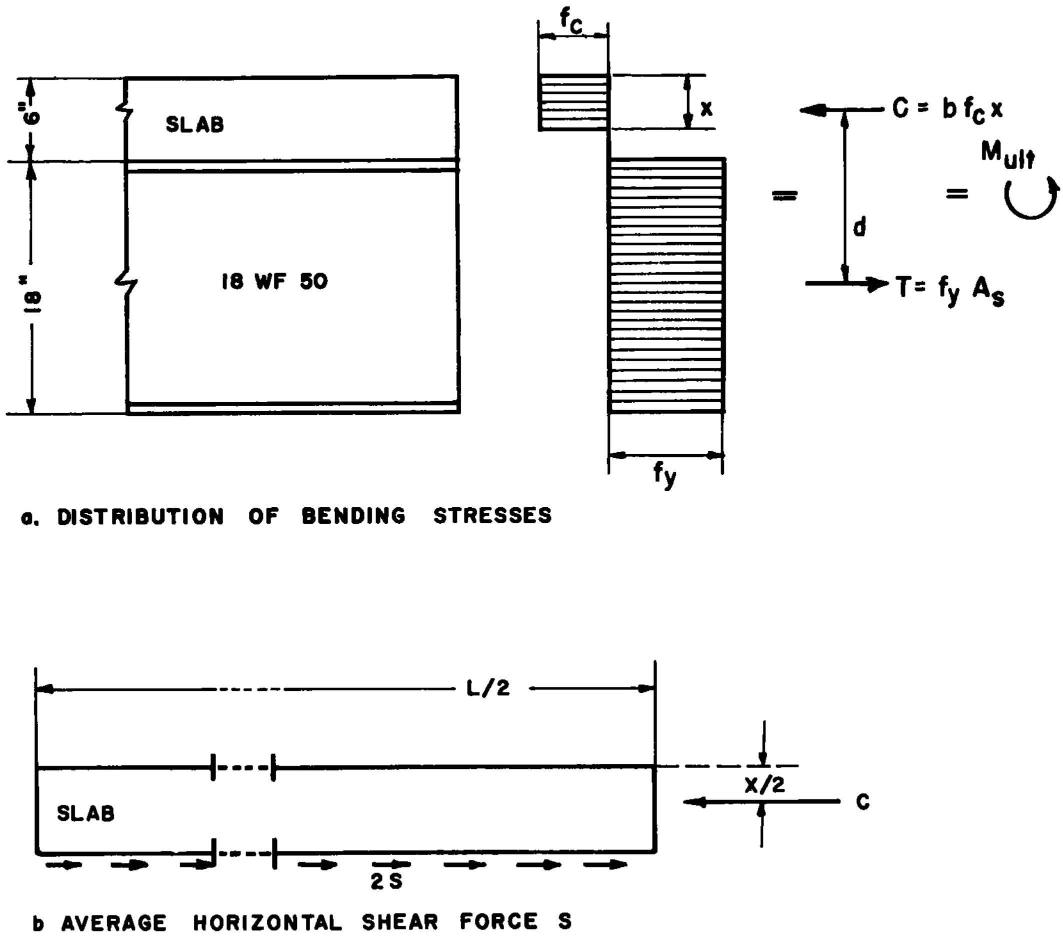


Figure 10. Assumed conditions at ultimate load.

very likely relatively high local stresses in the studs and in the surrounding concrete. However, it is known that such local stresses as they occur in many other cases (for example, around rivet holes, in welds, under concentrated reactions in concrete) can safely be sustained without damage to the member as such. Therefore, it is felt that nominal stress computations for the average shearing stress in the studs, neglecting probable bond and friction, are sufficient to derive safe working values.

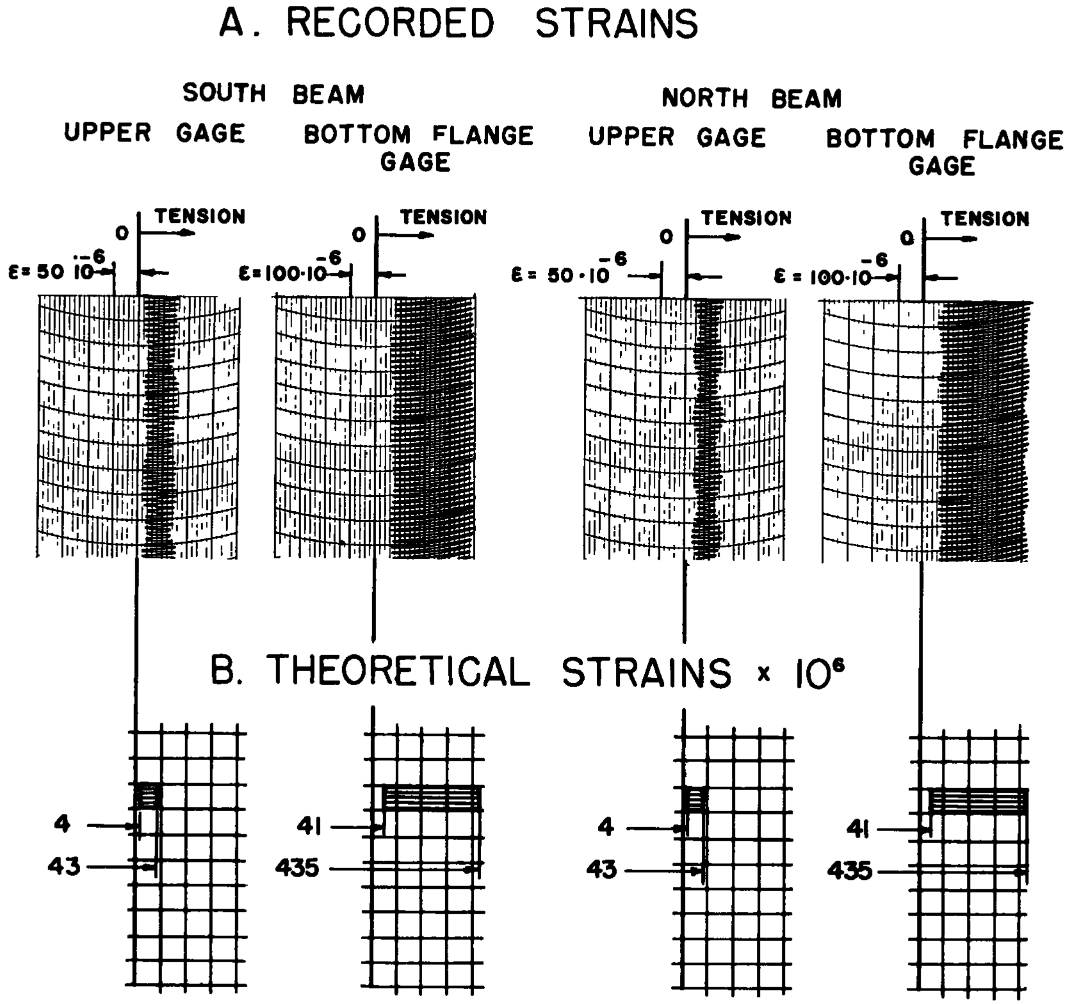


Figure 11. Comparison between recorded and calculated strains under cyclic loading; P_{min} = 4,000 lb, P_{max} = 42,200 lb.

A final comparison between the behavior of the ¾-in. studs and the ½-in. studs may be useful. Rather than to plot a slip vs load diagram, as in Figure 7, the average shearing stress in the stud (from Eqs. 5a, 7, and 10) corresponding to the measured slip is computed in Table 4 and presented in Figure 12. It can be seen that ½-in. studs started to slip at an average shearing stress of about 5,000 psi, compared to a stress of about 13,000 psi for the ¾-in. studs. However, the smaller studs seemed to "hang on" better, surpassing the larger studs at a slip of about 0.0008 in.

The geometric shape of the connectors could be the possible explanation of this rather surprising behavior. The hook of the small studs comes into action at the instant bending of the stud starts. It has the tendency to press the slab more firmly against the top flange and hence increase the mechanical friction. The larger studs with the upset head are longer and may have less tendency to increase this contact pressure. Push-out tests may be extremely helpful to clarify this point.

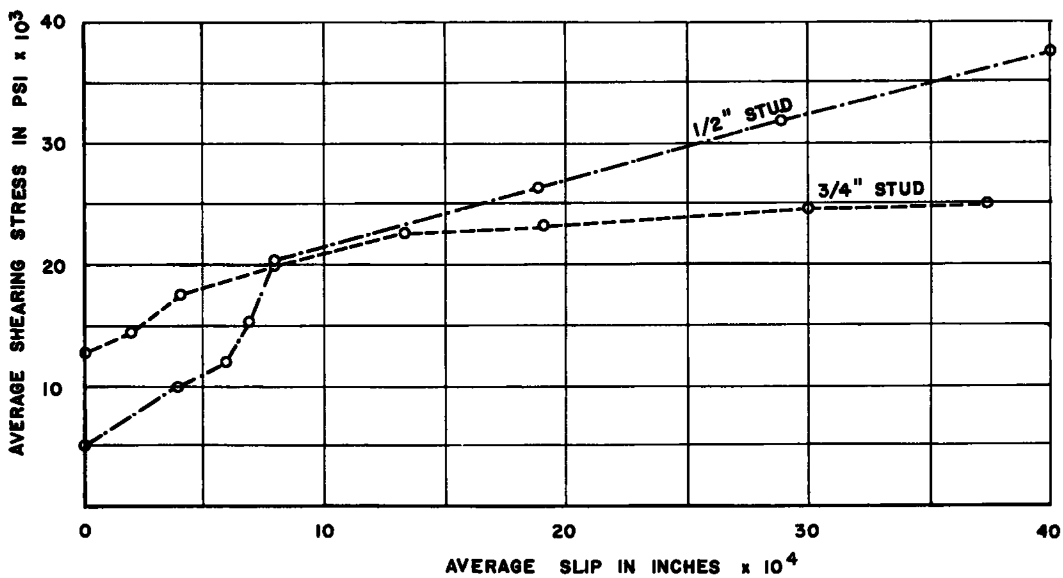


Figure 12. Comparison between behavior of 3/4-in. stud and 1/2-in. stud.

DESIGN RECOMMENDATIONS

The described test is one of the very few full-scale fatigue tests of composite beams, although a few fatigue tests on specimens of about 12-ft length have been reported (3). It is believed that the results obtained warrant the recommendation of design values for ½-in. stud shear connectors of the type tested. As the north beam with ¾-in. shear connectors offered more shear area per unit length than the south beam with the ½-in. studs (0.077 sq in. vs 0.042 sq in. per inch of length) the recommendations are only made for ½-in. studs. However, comparison of the behavior of the ¾-in. studs with design recommendations proposed by AASHO (4) is made.

The recommendations are stated first, followed by the necessary explanations.

Design Recommendation for ½-Inch "L" Shear Connectors

1. Bridge Design
   (a) The geometric shape of the connector is shown in Figure 13.
   (b) The hooks should preferably be oriented against the direction of the horizontal shear (toward middle for simple beams).
   (c) The maximum pitch should not be more than 24 in.
   (d) The useful static capacity of the shear connector in pounds is given by

$$Q_{uc} = 7,300 \sqrt{\frac{f_c'}{3,600}} = 120 \sqrt{f_c'}$$

in which f_c' is the 28-day cylinder strength of concrete, in psi.

The resistance value at working load is obtained by dividing Q_{uc} by the appropriate safety factor (4).

   (e) In any case, the allowable maximum connector force in pounds (Q_{all}) produced by live load (or dead load plus live load in case of shoring of steel beams) should be limited to

$$Q_{all} = 2,500 \text{ lb}$$

2. Building Design (Primarily Static Loading):
   (a) Useful capacity of the shear connector in pounds:

$$Q_{uc} = 120\sqrt{f_c'}$$

   (b) Allowable maximum connector force in pounds:

$$Q_{all} = \tfrac{1}{2}\, Q_{uc} = 60\sqrt{f_c'}$$

The following considerations lead to these recommendations: Recommendation (a) proposes a slightly longer shear connector than the ½-in. connectors used in the test specimen. With this increased height it is possible to accommodate the lower transverse reinforcing steel directly in the bend of the connector and still maintain a minimum concrete cover of 1 in. It is believed that fastening the reinforcement as indicated will be beneficial for composite action.

For recommendation (b), reference is made to Figure 7. In the test specimen all hooks were oriented toward the west end of the beam. The result was a smaller slip on the southeast end than the southwest end. Therefore, a proper orientation of the hooks as proposed should improve the interaction.

Recommendation (c) is taken from the AASHO Specifications (4). The useful capacity of the connectors is determined from the load slip curves, Figure 8. It should be remembered that the ultimate moment as computed was nearly reached despite considerable slip. However, it is probably desirable to limit this slip. From Table 4 it can be seen that the average slip for ½-in. connectors was 0.0040 in. at an average connector force Q = 7,310 lb. Beyond this force slip started to develop rapidly. It is felt that a total slip of about 0.0040 in. is quite tolerable. The corresponding residual slip is less than 0.0030 in. From this condition the value shown under (d) was derived. The influence of the concrete strength on the capacity Q_{uc} was assumed to vary as the square root of the cylinder strength (2, 4). At the date of testing the concrete had an age of 47 days with a cylinder strength of 3,600 psi.

The tentative AASHO Specifications (4) for the design of stud shear connectors (Art. 3.9.5—Shear) cover only straight studs with upset heads of 4-in. height. A check on the ¾-in. straight studs on the north beam shows the following behavior. At a load of 145,600 lb the average slip reached 0.0037 in. (see Table 4 and Figure 8). This corresponds to a useful capacity of approximately

$$Q_{uc} = 11,000\sqrt{\frac{3,000}{3,600}} = 10,100 \text{ lb}$$

based on a concrete strength $f_c' = 3,000$ psi. According to the AASHO Specification (4) the corresponding capacity of a ¾-in. diameter stud is

$$Q_{uc} = 332\, d^2\sqrt{f_c'} = 332 \times 0.75^2\sqrt{3,000} = 10,200 \text{ lb.}$$

The correspondence with the test value is rather remarkable.

However, in checking the ½-in. L-connectors on the south beam no such correspondence was observed. Indeed the AASHO formula covers only straight studs with upset heads of 4-in. height (15 percent reduction for 3-in. height), whereas the L-connectors introduce a new geometric shape not previously tested. The test results lead to the following useful capacity for ½- in. L-connectors:

$$Q_{uc} = 120\sqrt{f_c'} = 120\sqrt{3,000} = 6,580 \text{ lb.}$$

A rigid application of the AASHO formula to ½-in. L-connectors would yield

$$Q_{uc} = 332\ d^2 \sqrt{f_c'} = 332 \times 0.50^2 \sqrt{3,000} = 4,540 \text{ lb}$$

The considerable difference indicates that the latter formula does not cover the behavior of L-connectors. However, if applied it will result in an inequitable evaluation of the L-connector and, consequently, in a very conservative design.

Further tests are under way to substantiate the difference in behavior of the L-connectors and the straight stud connectors with upset heads.

Recommendation (e) follows directly from the behavior under cyclic loading. It is felt that the sustained cyclic loading assures a sufficient factor of safety against fatigue even under 125 percent of the design live load. The corresponding connector force for the ½-in. L-connectors was 2,580 lb (see Table 3). In an actual bridge the connectors at mid-span may be subjected to complete load reversals in short-span bridges. However, the test conditions were more extreme in the magnitude of the loads than any conditions occurring in practice. It is likely that recommendation (e), rather than (d), will generally govern a design. Further fatigue tests may lead to a relaxation of this recommendation.

Further research work on the fatigue behavior of different types of shear connectors is certainly desirable. Presently, comparative studies are restricted to static push-out tests (1, 2). It is felt that fatigue tests may substantially contribute to an investigation of the effectiveness and reliability of the different types of connectors.

Recommendation 2 (b) for the allowable connector force in building design (primarily static loading) contains a safety factor of 2. According to the "American Institute of Steel Construction Specifications," steel structures are designed with a safety factor of 1.65 against nominal yielding and 1.88 against ultimate load. As the evaluation of the stud behavior was on an ultimate basis, a safety factor of 2 is certainly adequate.

ACKNOWLEDGMENT

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The test specimen was designed by E. L. Erickson, Chief, Bridge Branch, Bureau Public Roads, and his staff. The testing procedure was planned and developed by Mr. Driscoll in conjunction with K S M Products, Inc. The tests were performed at Fritz Engineering Laboratory, Lehigh University, of which Professor W. J. Eney is director.

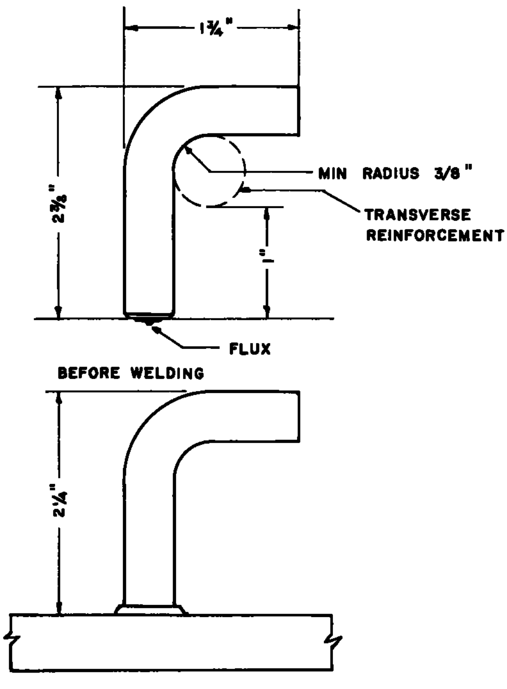

Figure 13. Detail dimensions of ½-in. diameter "L-stud connector."

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Appendix

TABLE A

Tensile Coupon Strengths of Steel in 18WF50

SPECIMEN NOS	MATERIAL	LOCATION	STATIC YIELD STRESS (psi)	ULTIMATE STRESS (psi)
1	ASTM	Flange	36,000	62,680
2	A-7	Web	41,300	64,580
3	Structural	Flange	35,500	62,460
4		Web	42,000	65,340
		Average	38,700	63,765

TABLE B

Cylinder Strength of Concrete in Slab

(Mix Design 3000 psi)

SPECIMEN NOS	AGE AT TEST (days)		STRENGTH (psi)
5	7		2,680
6	7		2,460
7	7		2,150
	7	Average	2,430
8	30		3,480
9	30		3,570
10	30		2,790
	30	Average	3,280
11	64		3,887
12	64		4,240
13	64		4,240
14	64		3,357
	64	Average	3,931

TABLE C

Tensile Tests of Stud Material

SPECIMEN NOS.	MATERIAL	NOMINAL DIAMETER (in)	YIELD STRESS (psi)	ULTIMATE STRESS (psi)	LOCATION OF FRACTURE
15	ASTM	1/2	59,700	70,500	Middle of Rod
16	A15-54T	1/2	59,300	70,000	" " "
17	Structural	3/4	65,200	71,200	" " "
18		3/4	65,000	70,500	" " "

Table D

Double Shear Test on Welded Shear Connectors

SPECIMEN NOS	MATERIAL	NOMINAL DIAMETER (in)	ULTIMATE SHEAR LOAD (lb)	ULTIMATE SHEARING STRESS (psi)	REMARKS
19	ASTM	1/2	28,300	72,300	Stud Sheared
20	A15-54T	1/2	28,600	73,000	" "
21	Structural	1/2	25,600	65,300	" "
22		3/4	52,000	58,900	" "
23		3/4	48,500	55,000	" "
24		3/4	39,600	44,800	Excessive Bending before stud sheared