Composite Open-Web Steel Joists

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THIS REPORT presents a description of research on the behavior of composite joists. In this type of structure a concrete slab is connected by shear-connectors to a steel joist. The research to be described consists of tests on beams. The purpose of the research was to determine the behavior of the shear-connectors.

REVIEW OF PREVIOUS WORK

For almost half a century composite members, consisting of a concrete slab on either a steel or a concrete beam, have been used in structural applications. In this type of structure the concrete slab is incorporated as part of the load-carrying system by the use of shear-connectors at the interface between the beam and the slab.

Considerable experimental and theoretical research has been conducted to determine the type, amount, and distribution of shear connection required to achieve full composite action.* Siess, Viest, and Newmark[1] stated that, in general, the stiffer the shear connection the greater the shear carried by it and the higher the degree of attainable composite action. These conclusions were based on a considerable number of pushout tests and quarter-scale model composite beams using I beams and a mortar slab. To help clarify what was actually occurring, an equation for incomplete interaction was developed, which provided reasonable agreement as to the shear force developed by the connectors.

Subsequently, more research was conducted, including the evolution of the stud shear-connector. Slutter and Driscoll[2] reported tests on thirteen beams of constant span length, but with different types of connectors and a considerable variation in the number and spacing of the shear-connectors. They came to the conclusion that the previously mentioned equation for incomplete interaction was a good research tool for evaluating the effect of slip within the elastic region of the load-deflection curve, but that it should not be used to design the number of shear-connectors required on a beam. They also concluded that at the ultimate load the spacing of the shear-connectors did not affect the ultimate capacity of the composite beam, provided there was an adequate number of connectors. The tests also indicated that even if there was appreciable slip it was still possible to develop the full composite ultimate moment of the beam. It must be borne in mind that these connectors were at a relatively close spacing and were attached to a comparatively stiff beam flange.

The first recorded tests on composite open-web steel joists were reported by Lembeck[3,4] who described two tests on a conventional joist and three tests on a composite joist. Each test was composed of two joists 20 ft long with a 2-ft spacing between the joists. On top of the two joists a concrete slab 20 ft $\times$ 4 ft $\times$ $2\frac{1}{2}$ in. was cast on a standard Corruform formwork. Composite action was achieved by inverting and lowering the top chord angles so that the webs extended above the top chord. The Corruform was supported on the horizontal leg of the top chord angles. As a result, the extended webs protruded about $2\frac{1}{4}$ in. into the concrete slab. Additional shear connection was achieved between the panel points by $\frac{1}{2}$-in. diameter filler rods welded to the top chord. For the conventional joist tests, 12H6 joists were used. The composite joist was similar only in the use of the same tension chord. The compression chord was considerably reduced in size to provide only for the minimum fabrication requirements and the overall depth was nearly equal to that of the standard 12H6 joist.

At design load, which was equal for both systems, the composite joist was found to have a deflection about 20 percent less than the conventional joist. The ultimate moment attained by the conventional joist was 1.72 times the design load, as compared to 1.96 for the composite joist. In the case of the conventional joist, failure was due to the buckling of the compression chord, whereas with the composite joist it was due to the rupture of the end

* *No attempt is made here to fully review this work. Only two major papers will be cited to illustrate two interpretations of composite joist behavior.*

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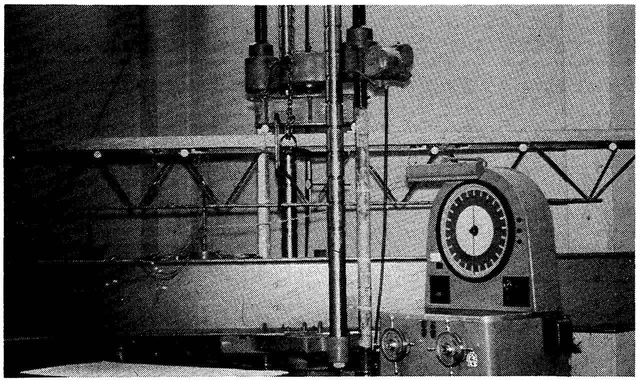


Fig. 1. Joist test set-up

support chord near the bearing seat, preventing a true evaluation of the ultimate moment capacity. The measured end slip was negligible for the composite joist, even at failure, but it was considerable for the conventional joist, indicating that the extended webs were capable of providing a relatively rigid connection. Near ultimate load the crack pattern on the top of the composite joist slab indicated that there was a plane of weakness along the joist, caused by the vertical leg of the top chord extending into the concrete.

The above study did show that for any given depth the composite joist could provide a greater moment capacity and stiffness with the same tension chord and web system, and with a considerable reduction in compression chord size.

In 1967, Wang and Kaley[5] reported four tests on composite open-web steel joists. Test No. 1 was performed on a non-composite joist to allow for comparisons to be made. Test No. 3 had a 2½-in. concrete slab placed on corrugated steel form, which in turn was plug-welded to the compression chord using standard washers. These washers were assumed to develop some shear connection between the joist and slab. Tests No. 2 and No. 4 were constructed according to the K-composite system. In this system interaction is achieved by providing a longitudinal shear key which is obtained by crimping the one-piece top-chord, so that the throat of the opening is narrower than the bulb below. The concrete is allowed to enter the bulb when the slab is cast. In Test No. 2 continuous metal chairs were also placed into the bulb to strengthen the shear key, while in Test No. 4 these metal chairs were omitted. Shear connection between the top chord and the slab was developed by bond and mechanical friction; at the ends of the beam the slab was haunched down to totally encase the top flange.

In the initial loading stages the deflection data indicated that for Tests No. 2 and No. 4 complete interaction was achieved, whereas in Test No. 3 incomplete interaction was developed due to the washers used to hold the form-work in place. Test No. 1 was a non-composite member and it behaved as predicted. The measured strains substantiated the results, that is, tension strains were obtained in the top chord for the composite joists (No. 2 and No. 4) while compression strains were recorded in the other two tests. The report only considered the composite action up to the beginning of first loss of composite action as indicated by the deflection curve of Test No. 4, since the bottom chord strains were below yield. As a result, no knowledge of the ultimate strength behavior of the composite joists was obtained.

The preceding two reports have indicated that it is possible to achieve composite action in open-web steel joist construction.

PURPOSE OF RESEARCH

The purpose of the research described in this report was to investigate the behavior of structural beams consisting of open-web steel joists, concrete slabs and shear-connectors. In particular, the research was initiated to determine the behavior of the stud shear-connector. To increase the probability of shear-connector failure, fewer shear-connectors were used than necessary to achieve full ultimate load.

The test series consisted of five beam test specimens (Fig. 1). The test variables were:

(a) Two types of top and bottom chords were used: one-piece chords (cold-formed shapes) or two-piece chords (two hot-rolled angles).
(b) The shear-connectors were placed either at the panel points or midway between the panel points (see Fig. 2).
(c) The number of shear-connectors was varied from three to six per shear span.
(d) The size of the tension chord was varied so as to provide different stiffness factors.
(e) The web members were increased in size above those of normal joists to eliminate web failures.

The remaining physical dimensions were, for all practical purposes, constant for all of the test specimens.

The slab dimensions were 12 in. wide by 3 in. deep and the full length of the beam in all cases. The results obtained from the steel coupon tests indicated that the yield point of the steel satisfied the requirements for H-type joists.

TEST SPECIMENS AND TESTING PROCEDURE

The specimens were obtained from several steel joist manufacturing shops, where the shear-connectors were welded to the top chord, using standard equipment and inspection procedures.

Only one type of shear-connector was used, a ⅜-in. diameter Nelson stud connector. For the two-piece compression chord the connectors were 2½ in. long, whereas for the one-piece compression chord the connector was

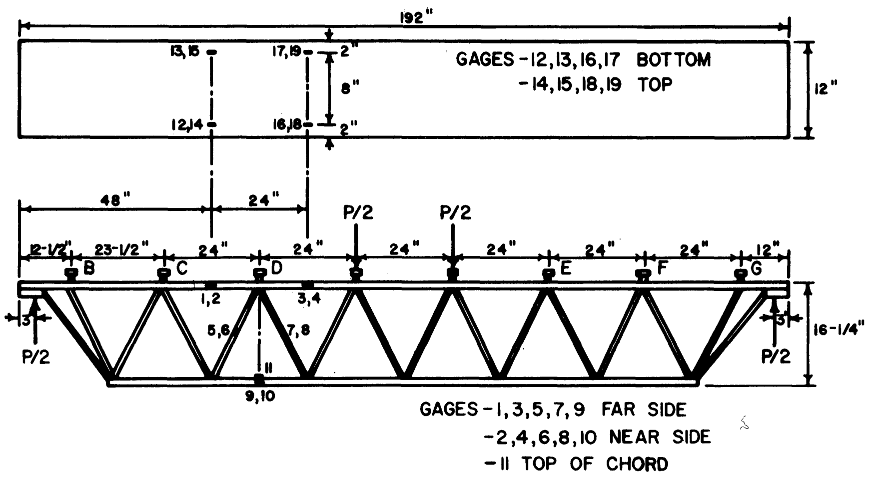


Fig. 2. Test specimen B5

attached at the bottom of the trough, necessitating a connector approximately 3 in. long so that it would extend $2\frac{1}{4}$ in. above the top surface of the top chord.

In the laboratory the bare joist was placed in the formwork, and the top chord was greased to prevent bond from developing. A wire mesh 6-6-10 was placed in the form, and the concrete, made from Type III high early cement, was cast. From each batch of concrete at least three cylinders were also poured. The concrete was wet cured for seven days and then the forms were stripped. The complete composite specimen was then moved to a Universal Testing machine (Fig. 1), where it was instrumented and tested.

The instrumentation of the specimen consisted of electrical SR-4 strain gages on the steel and concrete, a deflection gage at the mid-point and slip dials at each shear-connector location. The locations of the SR-4 strain gages are shown in Fig. 2. The slip readings were achieved by attaching a small steel clip to the top chord of the joist, to which an extension dial could be mounted. The runner from the dial was placed against an angle glued to the concrete adjacent to the shear-connector (Fig. 3); the dial readings then gave directly the relative movement of the concrete slab with regard to the shear-connector.

The specimens were basically of the 16H4 type, however, the tension chord was increased in size to provide for increased moment capacity. The web members were also considerably larger than the normal web sizes to prevent web buckling. A representative cross section is shown in Fig. 4 with geometric properties indicated on the cross section and tabulated in Table 1. In addition, Table 1 provides the total length and shear span of each test member, the moment of inertia of the bare joist and the composite member, the values of the cross-sectional constants C_a and C_b, and, finally, the yield strength of the steel and the compressive strength of the concrete.

The testing of each specimen was done in increments of load. At each load increment all strain, deflection, and slip readings were taken and the concrete was examined for cracks and separation from the joist.

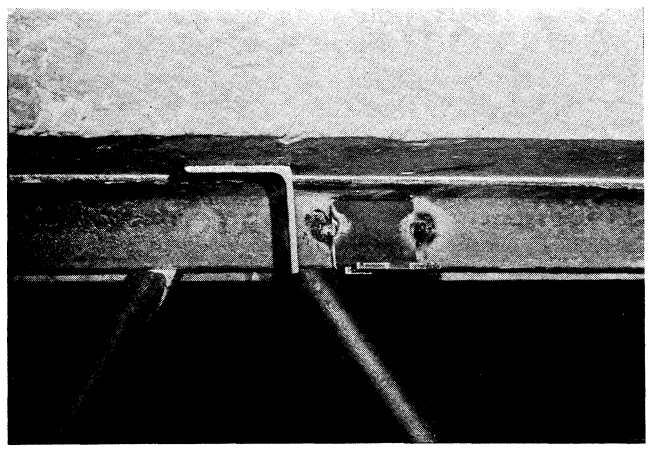


Fig. 3. Slip measuring device

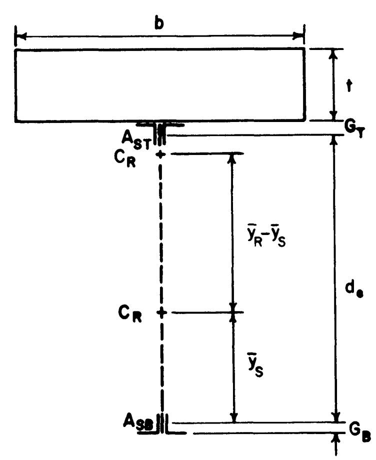


Fig. 4. Typical cross-sectional properties

Table 1. Beam Cross-Section Properties

Joist Type	B1 Two Piece	B2 Two Piece	B4 One Piece	B5 One Piece	B6 Two Piece
			Connectors		
	3	4	3	3	6
L (in.)	186	186	186	186	186
a (in.)	81	81	81	81	81
d_e (in.)	15.16	15.16	15.40	15.40	15.16
b (in.)	12	12	12	12	12
t (in.)	3	3	3	3	3
A_{st} (in.^{2})	0.79	0.79	0.72	0.72	0.79
A_{sb} (in.^{2})	1.09	1.09	1.07	1.07	1.09
A_w (in.^{2})	0.54	0.54	0.52	0.52	0.54
y_s (in.)	6.38	6.38	6.22	6.22	6.38
I_s (in.^{4})	105.1	105.1	102.1	102.1	105.1
y_r (in.)	13.67	13.67	13.90	13.90	13.67
I_r (in.^{4})	254.5	254.5	257.7	257.7	254.5
C_a (lbs^{-1})	$6.35(10)^{-8}$	$6.35(10)^{-8}$	$6.84(10)^{-8}$	$6.84(10)^{-8}$	$6.35(10)^{-8}$
C_b (in./lbs)	$3.41(10)^{-9}$	$3.41(10)^{-9}$	$3.64(10)^{-9}$	$3.64(10)^{-9}$	$3.41(10)^{-9}$
F_y (ksi)	50	50	50	50	50
f_c' (psi)	3620	3860	3120	3220	2050

THEORETICAL ANALYSIS OF INCOMPLETE INTERACTION

In a fully composite member the strain is assumed to have a linear variation throughout the depth of the member. This assumption permits the calculation of stress and strain by the linear elastic theory. An examination of the results of the slip measuring dials in these tests showed that the specimens did not behave in a fully composite manner. Therefore, if there is some relative movement between two adjacent surfaces, as is the case for incomplete interaction, the simple elastic theory formulas must be replaced with ones that take into account this strain discontinuity. Newmark, Siess and Viest[6] derived incomplete interaction formulas and compared them with experimental results.

In deriving the incomplete interaction formulas, the following assumptions were made:

(1) Elastic beam theory (as contrasted to truss theory).
(2) Usefulness is terminated at first yield.
(3) Strain distribution across the section is linear.
(4) Curvature of slab and joist is equal.
(5) Bending in top chord angles is negligible.
(6) Shear connection between joist and slab is continuous.
(7) Change in slip is proportional to change in connecting shear force.
(8) Specimens behave elastically between load increments.

Using incremental equilibrium equations substituted into a strain compatibility equation, including slip, a second order differential equation results. The solution for the equation is given by Newmark, Siess and Viest[6] and by the authors.[7]

The relationship between the shear-connector stiffness K and the slip increment $\delta\Delta_s$ is as follows:

$$K = \frac{C_b\, \delta P\, G(K)}{C_a\, \delta\Delta_s} \tag{1}$$

where

$$G(K) = 1 - \cosh(vx)\left[\frac{\cosh(vL/2 - va)}{\cosh(vL/2)}\right] \tag{2}$$

$$v = \sqrt{C_a K} \tag{3}$$

$$C_a = \frac{1}{A_c E_c} + \frac{(t/2 + G_t)[\beta d_e + (t/2 + G_t)(\beta + 1)]}{I_c E_c\,(\alpha + \beta + 1)} + \frac{[d_e + \alpha(d_e + t/2 + G_t)]}{A_{st} E_s d_e\,(\alpha + \beta + 1)} \tag{4}$$

$$C_b = \frac{(t/2 + G_t)(\beta + 1)}{I_c E_c\,(\alpha + \beta + 1)} + \frac{\alpha}{A_{st} E_s d_e\,(\alpha + \beta + 1)} \tag{5}$$

$$\alpha = n A_{sb} d_e^2 / I_c \tag{6}$$

$$\beta = A_{sb}/A_{st} \tag{7}$$

In these equations L and a are the length and shear span, respectively, as shown in Fig. 2 and tabulated in Table 1. The terms C_a and C_b are geometric constants of the composite joist shown in Fig. 4 and tabulated in Table 1. The term δP is the incremental load value and $\delta\Delta_s$ is the resulting incremental slip.

In each experiment the values of $\delta\Delta_s$ were measured at the shear-connector locations $\mathbf{x}$ and for load increments δP. With this experimental data it is possible to solve Eq. (1) for K.

In the tests the shear connection was not continuous, but was composed of discrete shear-connectors located at intervals along the shear span. If the shear-connectors are at a reasonably uniform spacing λ and if the strain in the concrete is small compared to the slip, then one can define the shear-connector force from Eq. (1) as:

$$\delta F_c = \lambda K \delta\Delta_s \tag{8}$$

DISCUSSION OF TEST RESULTS

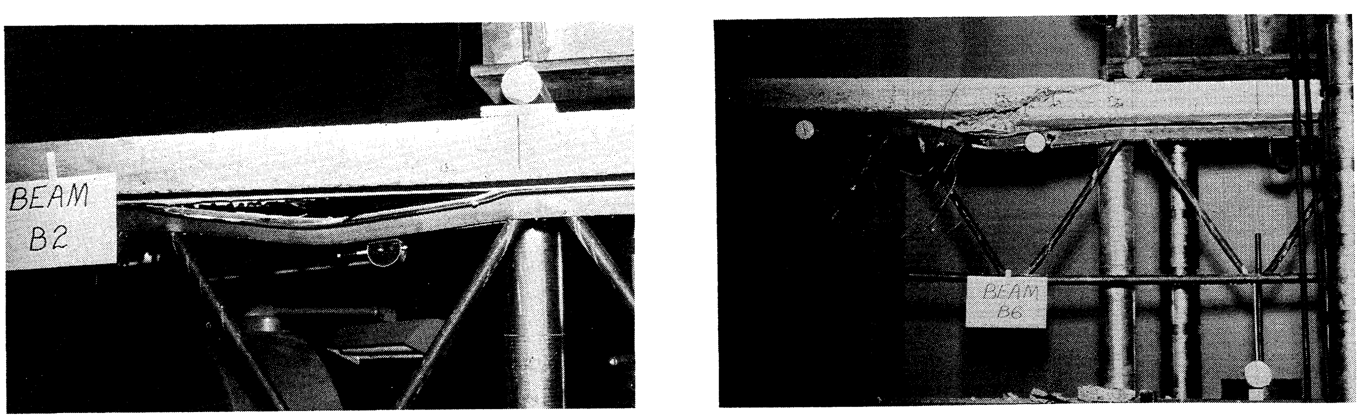
Typical failures of the beams tested are shown in Fig. 5. A more detailed explanation of each test follows.

Specimen B1—This specimen had a two piece compression chord with three shear-connectors in each shear span. The shear-connectors were located at the panel points. Readings were taken and observations made at each application of a load increment. The first cracks in the concrete slab occurred under the load points at a fairly high load. Increasing the load slightly caused a considerable separation of the slab from the compression chord in the vicinity of the second shear-connector from the left end. With additional load a loud report was heard and the left compression chord buckled laterally about 1-in. The three shear-connectors in the left shear span failed at the same time. Failure was caused by the buckling of the compression chord after the shear-connectors had failed.

Specimen B2—This specimen was similar to B1 except that it had four shear-connectors in each shear span, located midway between the panel points. The cracks in the concrete were first observed in both shear spans under the load points and at each shear-connector closest to the load point. With the application of the next load increment a loud report was heard. Examination of the beam indicated considerable separation at the shear-connector closest to the load point in the right span. On attempting to increase the load the compression chord buckled vertically (Fig. 5a). The failure was due to the fracture of the shear-connector followed by the compression chord buckling.

Specimen B4—This specimen had a one-piece compression chord with three shear-connectors mid-way between the panel points in each shear span. The first cracks appeared in the concrete at the shear-connector closest to the end reaction in each shear span after only a small amount of load was applied. Additional load resulted in the formation of cracks at the second shear-connector from the end reaction and at the load points. With more load increments it became difficult to hold the load steady and then the first compression web member in the left span buckled, preventing a total evaluation of the shear-connectors.

Specimen B5—This specimen was similar to B4. It had three shear-connectors located mid-way between the panel points in each shear span. As with B4 the first cracks in the concrete occurred near the shear-connector closest to the reaction points in each shear span. Additional loading resulted in cracks in the concrete under the load points. At this time it became difficult to hold the load steady and then as the load was increased the weld connection between the web member and the tension chord fractured. This premature failure prevented a total evaluation of the shear-connectors.



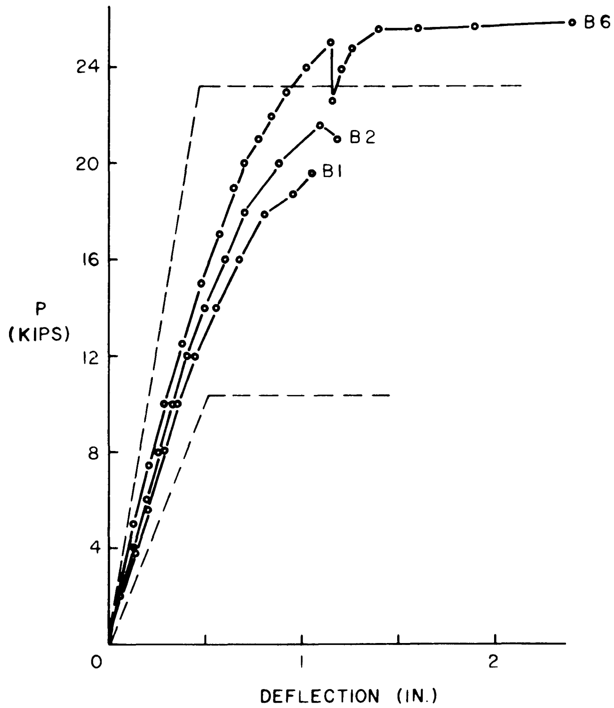
a. Specimen B2

b. Specimen B6

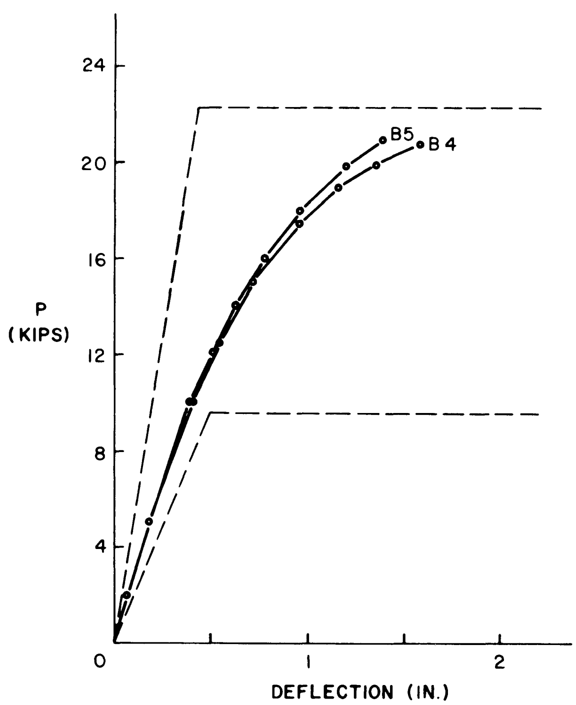
Fig. 5. Typical joist failures

Specimen B6—Similar to B1 and B2 except that it had six shear-connectors in each shear span. The six connectors in each span were arranged in three groups of two connectors each and were located midway between the panel points. After several load increments a crack appeared at the pair of shear-connectors closest to the right reaction. After additional loading, a crack appeared near the left load point. The next loading resulted in the fracture of a weld between the web and the tension chord. The beam was unloaded and the weld repaired.

The beam was reloaded to the previous value and the test continued. Shortly thereafter it appeared as if the tension chord had yielded as large deflections with small load increases were obtained. As more load was applied it was noticed that the concrete slab was being bent in reverse curvature as tension cracks were appearing on the top of the slab near each reaction point. As more deflection occurred, the slab split just to the left of the left load point (Fig. 5b). The failure was due to the yielding of the tensioned chord followed by the splitting of the concrete.



a. Specimens B1, B2 and B6



b. Specimens B4 and B5

Fig. 6. Load-deflection diagrams

Discussion—In conventional steel joist construction the compression chord has a larger area than the tension chord. This is to insure that buckling of the compression chord does not occur before yielding of the tension chord.

The effect of the increased tension chord size in a composite system can be seen in the load deflection diagrams of Fig. 6. The ordinate of the figure is the total load applied to the specimen by the testing machine while the abscissa is the center line deflection of the tension chord. The test results are superimposed on the theoretical upper and lower bounds, indicated as dashed lines. The upper bound is based on the yielding of the tension chord of the composite joist, whereas the lower bound is based on the buckling of the compression chord of the bare joist.

The three beams represented in Fig. 6a were identical except for the number and location of the shear-connectors, as previously mentioned. In the case of specimens B1 and B2, the degree of composite action was almost sufficient to develop the full capacity of the system; however, enough slip occurred to finally allow the compression chord to buckle. For B6 the full moment capacity was achieved, even though there was slip and yielding of the tension chord prior to the splitting of the concrete.

Results similar to the above were obtained for B4 and B5, as shown in Fig. 6b. These two specimens were identical to each other and only differed from the previous group in that its chords were one-piece members as compared to two-piece chords. In both cases only three stud shear-connectors per shear span were used and the maximum theoretical moment was nearly achieved even though considerable slip had occurred. In both cases the failure was of a secondary nature.

It is clearly evident that complete interaction was not achieved with the number of stud shear-connectors used in these tests. This loss of composite action is typically represented in Fig. 7 for specimen B2, where the horizontal slip between the concrete slab and the top chord of the

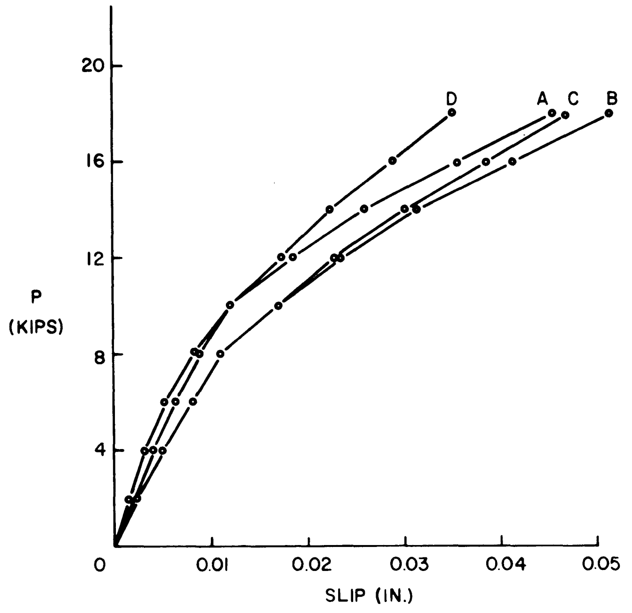


Fig. 7. Load-slip diagram for specimen B2

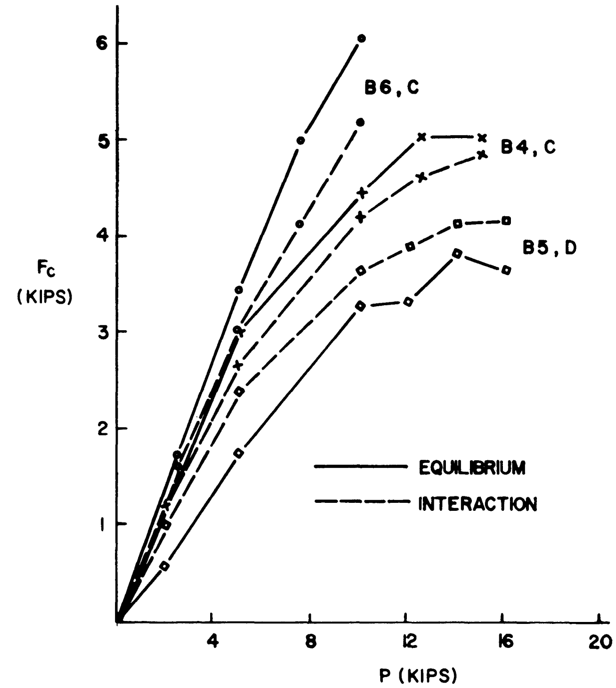


Fig. 8. Shear-connector forces

joist is plotted against the total load applied to the specimen. The slip began immediately with the application of the first load and at first appears to have a somewhat linear relation ship with the applied load. However, in general the linear relationship quickly becomes nonlinear as the amount of slip increased rapidly with small increases in load. The slip generally was larger towards the end reactions, indicating that the end shear-connectors are subjected to more shear force than those nearer the load points.

In developing the interaction equation of the previous section it was necessary to make use of the slip and the corresponding applied load from the test results to permit an evaluation of the shear-connector forces. In each of the tests electrical SR-4 strain gages were placed around one upper panel point to provide a means to measure the forces at the panel point. In the specimens where a stud shear-connector was located at this panel point, the unbalanced force was the shear-connector force. The results of this equilibrium condition are compared with the interaction results for several cases in Fig. 8, where the total load applied to the specimen is plotted against the shear-connector force. The correlation between the two methods is especially good at the lower load levels, indicating that the shear-connector forces obtained by this method are relatively accurate. At higher loads and for some of the other tests the correlation is not as conclusive, due to the bending of the compression chord. A summary of all the shear-connector forces using the interaction equation is given in Table 2 for a slip value of 0.003 in., which is the current allowable slip in the AASHO specifications, and the value of the maximum shear force obtained for each stud connector (Table 3). A force-slip curve was computed for each shear-connector, using Eqs. (1) and (8). The measured incremental slip readings and load readings were utilized in solving these equations. The computations were made on an electronic computer.

The distribution of shear-connector forces along a beam, along with load-by-load development, is given in Fig. 9 for representative specimen B2. In the lower portion of the figure the slip is plotted against the shear-connector force, while in the upper portion the maximum

Table 2. Shear Force in ⅜ in. Stud Shear-Connectors at 0.003 in. Slip (kips)

Specimen	A	B	C	D	E	F	G	H	Average
B1	2.11	1.76	2.18	1.80	2.28	1.64	—	—	2.0
B2	1.91	1.46	1.24	0.86	0.72	1.56	1.78	1.37	1.4
B4	—	0.92	2.03	1.76	1.64	1.93	0.56	—	1.5
B5	—	1.70	1.99	1.98	1.77	1.79	1.43	—	1.8
B6*	—	3.60	3.33	2.79	2.77	2.88	2.99	—	1.5
									1.6 Average

Table 3. Maximum ⅜-in. Stud Shear-Connector Strength (kips)

Specimen	A	B	C	D	E	F	G	H
B1	7.16	5.58	4.70	5.02	6.45	6.09	—	—
B2	6.68	7.87	6.17	3.01	2.75	5.84	7.43	6.15
B4	—	3.38	4.87	3.05	2.74	4.92	2.48	—
B5	—	3.32	5.28	4.32	4.20	5.51	3.68	—
B6*	—	11.84	9.89	5.73	5.43	8.31	7.90	—

* There were two connectors at each location

shear-connector is placed at its relative position in the specimen. This figure gives the same indication as did Fig. 7 that at low loads there is a linear relationship between the slip and the shear-connector force. This relationship quickly becomes non-linear as large increases in slip produce only small changes in shear-connector forces. The shear-connector forces are larger towards the end reaction points with the exception of the last and second-last connector. This difference probably occurs because the concrete usually cracked at the shear-connector nearest the end reaction at a fairly low load, invalidating some of the assumptions on which the analysis was based.

Using the electrical SR-4 strain gage readings obtained for the web members, it was possible to determine, for one panel, the amount of vertical shear that was carried by those web members. The results are plotted in Fig. 10 for specimen B5 where the applied vertical shear is plotted against the strain in web members. The results are superimposed on a theoretical line indicating the strain that would be in the members if they carried all the vertical shear. Within the range of experimental accuracy it appears that the webs carry nearly all the vertical shear.

The strength of the concrete was assumed to be a variable in the program; however, it appears not to have a significant effect, as shown by specimen B6, which had a very weak concrete strength compared to B1 or B2 and yet was able to develop its full moment capacity.

The effect of the type of compression chord appears more as an effect of the length of the attached shear-connector than of the geometric shape of the compression chord. The one-piece compression chords of B4 and B5 had the shear-connector located in the trough whereas B1, B2 and B6 had the shear-connectors located on the horizontal leg of the angle. The increased shear-connector length resulted in increased flexibility, more slip, and as a result, more deflection.

The location of the shear-connector was noticeable in the amount of vertical separation that occurred between the concrete slab and the compression chord of the joist. In the case of the two-piece compression chords of B1, B2 and B6, the shear-connectors located midway between the panel points of B2 and B6 appeared to prevent the compression chord from buckling. In B2, with only one connector at each location, it was quite evident by the separation as to which of the two angles had the shear-connector attached to it and which did not. Speci-

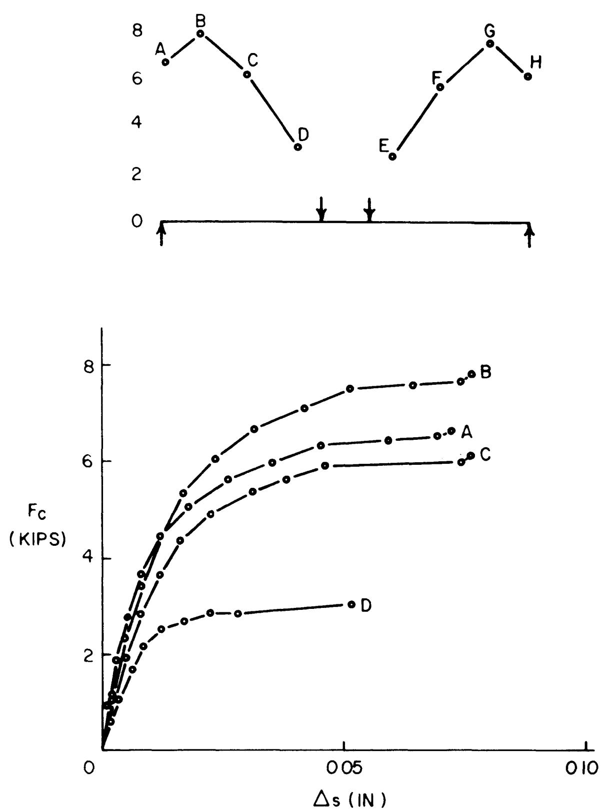


Fig. 9. Distribution of shear connection for specimen B2

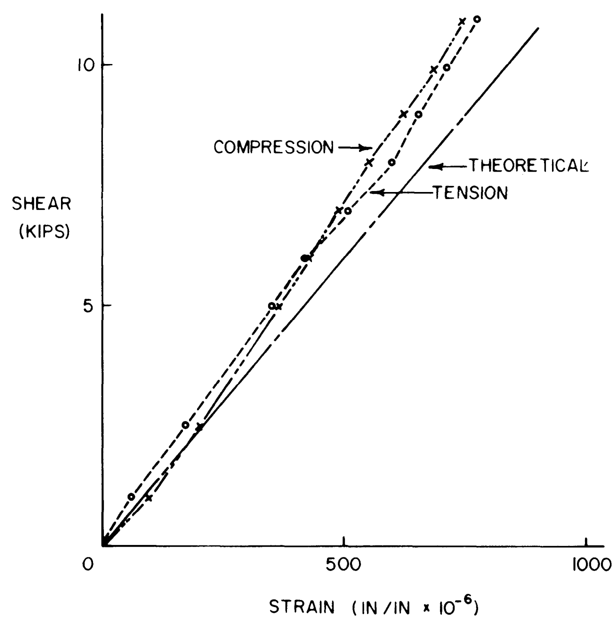


Fig. 10. Web strains in specimen B5

men B6, with a shear-connector attached on each angle between the panel points, had very little separation compared to B1, which had the shear-connectors at the panel points and exhibited considerable separation of the compression chord from the concrete. The contrast was not as good for the one piece compression chords of B4 and B5, as the in-plane symmetry helped keep the compression chord in position. The separation was however somewhat larger for B5, with the shear-connectors at the panel points, than for B4, with the shear-connectors midway between the panel points.

SUMMARY AND CONCLUSIONS

The object of this research project was to determine the load-deformation behavior of shear-connectors and to evaluate the degree of composite action that could be obtained for a composite joist composed of an open-web steel joist, a concrete slab, and mechanical shear-connectors holding the two together. A total of five simple beam specimens were tested.

The effect of the length of the stud shear-connector appears to be a slight reduction in stiffness, although this may be a result of the attachment in the trough of the one-piece compression chord. The location of the stud shear-connector, at either the panel point or in between the panel points, does not seem to give any appreciable difference in behavior except for the tendency to hold the joist compression chord against the concrete slab better when the connectors are located between the panel points. This may become more important if the relative size of the compression chord is reduced.

A solution for a differential equation for the slip was used, along with experimental results to evaluate the shear-connector forces, and was compared with results obtained from electrical strain gage measurements taken during the testing. The two methods agreed quite well until the deformations became quite large.

The theoretically computed shear-connector capacities are given in Table 2 for a fictitious limiting slip of 0.003 in. for each shear-connector. At this slip it can still be reasonably assumed that the slab and the joists act for all practical purposes in a fully composite manner. The average shear-connector capacity at a slip of 0.003 in. is 1.6 kip per shear-connector for the beam tests.

The shear-connector capacities at maximum load are listed in Table 3. Each connector did not carry the same load and therefore the underlined largest values are the appropriate answers. The shear-connectors failed in specimens B1 and B2. The average maximum shear-connector force from these tests is 6.7 kips per connector. The other specimens failed from other causes and their average maximum shear-connector force is 5.2 kips per connector, lower than the previous value. The former value is the appropriate value to use.

These tests indicated two limiting capacities for $\frac{3}{8}$-in. stud shear-connectors: 1.6 kips per connector for a limiting slip of 0.003 in., and 6.7 kips per connector for the maximum capacity. Depending on the type of design, the value of 1.6 kips per connector can be used directly at working loads, or the value of 6.7 divided by an appropriate factor of safety can be used as an alternate design value. In view of the scatter of data it would seem advisable to use a factor of safety no less than 2.0; perhaps 3.0 would be preferable.

The scatter of data is not surprising in view of similar scatter in other composite beam studies (Ref. 2, for example). This scatter is due to the following factors: (1) The method of analysis assumes that the behavior of the connector is linear between load increments. It uses experimentally obtained data, and it assumes continuous shear connection. (2) The shear-connector is embedded in concrete and the properties of this material may be quite different in the various small locations at which the shear-connectors are placed. (3) The diameter of the stud is quite large compared to the thin plate to which it is welded. (4) The deformations of the compression chord can influence the behavior of the connectors.

The large scatter of the results from these causes overshadowed the possible effects of the other variables, such as the concrete strength, the location of the shear-connectors, and the type of top chord. These latter variables did not seem to have as large an effect as the variations due to the uncontrollable causes.

The results of this study apply to the behavior of composite joists with $\frac{3}{8}$-in. round stud shear-connectors. Any other type of shear connection should be subjected to a similar study to determine shear-connector capacities.

The study has shown that the web system carries nearly all of the vertical shear, and therefore only the web system should be counted on to support the total shear in design.

Before a design recommendation for composite joists with $\frac{3}{8}$-in. stud shear-connectors can be advanced, it would be very desirable to test several joists with the required number of connectors necessary to achieve full composite action.

ACKNOWLEDGEMENTS

This study of this report was made possible through a grant by the Steel Joist Institute and it was performed at the Structures Laboratory of Washington University. The research work was guided by the Research Committee of the Steel Joist Institute. A special panel of this Committee, consisting of Messrs. Bradbury, Peck and Bay was intimately involved in this work, and their help is here gratefully acknowledged.

NOMENCLATURE

A_c = Cross-sectional area of concrete slab
A_{sb} = Cross-sectional area of joist bottom chord
A_{st} = Cross-sectional area of joist top chord
A_w = Cross-sectional area of joist web
a = Length of shear span
b = Concrete slab width
C_a = Composite joist cross-sectional constant
C_b = Composite joist cross-sectional constant
d_e = Distance between top and bottom chord centroids
E_c = Modulus of elasticity for concrete
E_s = Modulus of elasticity for steel
F_c = Shear-connector force
F_y = Yield point of steel
f_c' = Ultimate concrete strength
$G(K)$ = Function for stiffness of shear-connector
G_t = Distance from top of bare joist to centroid of top chord
I_c = Moment of inertia of concrete slab
I_s = Moment of inertia of bare joist
I_r = Moment of inertia of composite joist
K = Spring constant of continuous shear connection
L = Total beam length from supports
n = Modular ratio of concrete and steel
P = Total applied load
t = Concrete slab thickness
$\mathbf{x}$ = Variable distance from support
y_s = Distance from centroid of bottom chord to neutral axis of bare joist
y_r = Distance from centroid of bottom chord to neutral axis of composite joist
α = Composite joist cross-sectional constant
β = Bare joist cross-sectional constant
ν = Composite joist cross-sectional and stiffness variable
Δ_s = Slip between concrete slab and shear-connector
δ = Indicate incremental action
λ = Shear-connector spacing

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