Composite Action Without Shear Connectors

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In composite construction of building floor slabs, the common method for transferring shear stresses from the concrete slab to the steel beam is to use stud shear connectors. An alternate method, which is seldom used, is to completely encase the steel beam in concrete. When the steel beam is completely encased, the 1970 AISC Specification assumes the beam to be interconnected to the concrete by natural bond without additional anchorage. This assumption may be used when at least 2 in. of concrete cover is provided over the sides and bottom of the beam and at least 1½ in. of cover is provided at the top. An additional requirement is that adequate mesh or reinforcing steel be used in the encasement to prevent spalling of the concrete.

In most composite construction the neutral axis of the composite section is located either in the floor slab or at a very small distance below the bottom of the slab. Since the specifications do not permit the concrete on the tension side of the neutral axis to be used as a structural element, the concrete below the neutral axis on a fully encased beam does not theoretically increase the stiffness of the beam. The tension concrete does provide more bond area between the steel and concrete; however, the advantage of this additional bond may be offset by the additional dead weight of the concrete used to encase the beam.

To offset the disadvantage of the large amount of concrete required to encase the beam, a partially encased beam has been proposed. Details of this method of construction are shown in Fig. 1. The concrete below the bottom of the deck slab that would be in tension, and consequently not add to the stiffness of the beam, has been eliminated. In addition to reducing the dead weight and quantity of concrete required, the forming costs for this type construction are drastically reduced. Corrugated steel forms used for the bottom of the deck slab can be removed and reused, and it is not necessary to provide wire mesh and forms for the sides and bottom of the beam.

In order to develop composite action in a beam with the flange embedded in the deck slab, sufficient natural bond and friction forces must exist between the flange and concrete to transfer the horizontal shearing forces. To determine if the natural bond forces are adequate, a series of tests were conducted at The Pennsylvania State University. This paper discusses the method of testing and the results of the tests.

The steel beams used in the tests were castellated beams, since data from previous tests on composite castellated beams with shear connectors were available for comparison. Castellated steel beams are formed by cutting or punching a wide flange beam in a truncated sawtooth pattern, staggering top and bottom chords, and welding the chords together as shown in Fig. 2.

In the composite system of concrete slab and steel beam, almost all of the concrete slab will be in compression while a large percentage of steel will be in tension. This indicates that a larger cross-sectional area of steel should be used in the bottom chord and a smaller cross-sectional area of steel for the top chord, as shown in Fig. 1. This hybrid combination minimizes the weight and achieves a little larger load carrying capacity for a given quantity of steel.

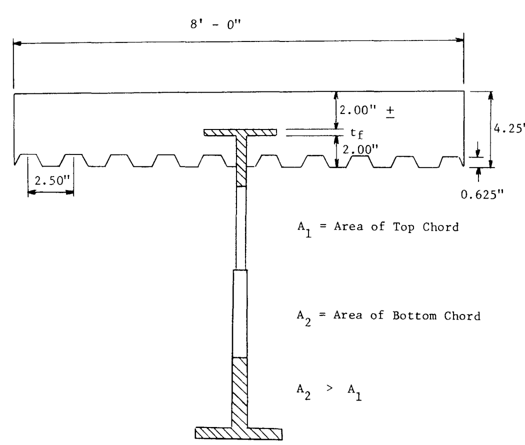

Fig. 1. Flange embedded for castellated hybrid steel beam

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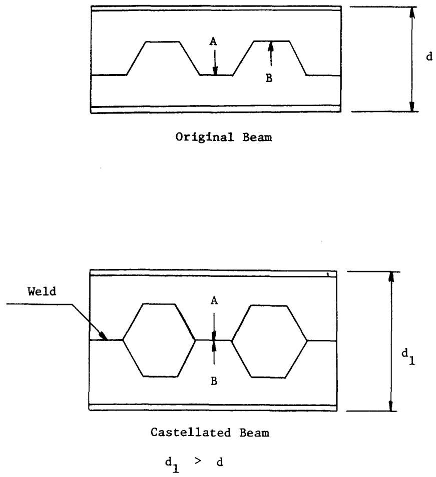

Fig. 2. Fabrication of castellated steel beam

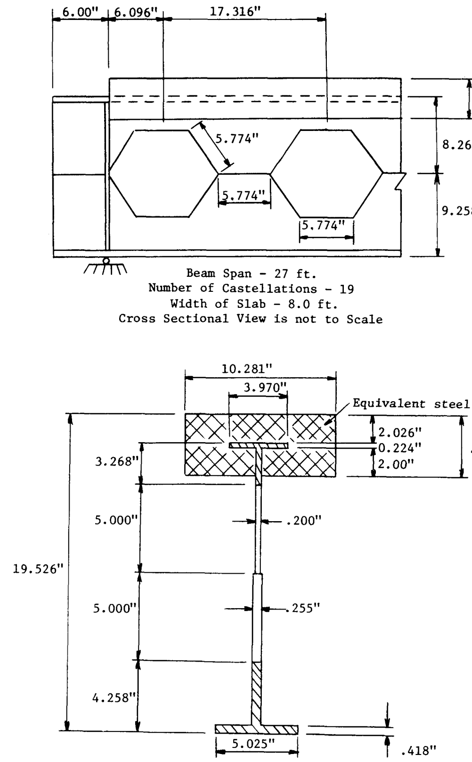

Fig. 3. Geometry of beam 18CCB20

DESCRIPTION OF TEST SPECIMENS

Test specimens were hybrid castellated steel beams designated as 18CCB20 and 20CCB25. The 18CCB20 beam was fabricated from a W12×14 (top chord) and a W14×26 (bottom chord). Beam 20CCB25 was fabricated from a W12×19 (top chord) and a W16×31 (bottom chord). Beam 18CCB20 was 28 ft long, while beam 20CCB25 was 30 ft long. Both beams had stiffeners 6 in. from the ends to prevent web crippling at the supports. The beams were made from A36 steel with a measured yield stress of 38 ksi. Dimensions of the beams are shown in Figs. 3 and 4, along with the equivalent cross-section used for design.

The normal weight concrete slabs were 8 ft wide by $4\frac{1}{4}$ in. thick, with a 28-day compressive strength of 4.0 ksi. The concrete was placed on removable corrugated metal decking, which was supported by wooden bracing. The decking was placed under the top flange a distance of 2 in., as shown in Fig. 1, and oiled slightly to reduce any adhesion between the decking and concrete. Construction chairs were wired to the top flange to hold the 6″ x 6″-8 ga. welded wire fabric $\frac{3}{4}$-in. from the top of the slab. The welded wire fabric served as both temperature steel and tensile reinforcement for the concrete. No special cleaning treatment was used on the top flange, and the concrete was placed around the flange and vibrated.

When the concrete slabs were placed, shored construction was used with the steel beams supported at their third points and the metal deck supported along the edges by wooden forms. After the compressive strength of the concrete reached 4.0 ksi, all temporary supports were removed and the instrumentation and loading apparatus were installed. Shored construction was necessary for the individual test beams, since the deck forms were fully supported along both edges.

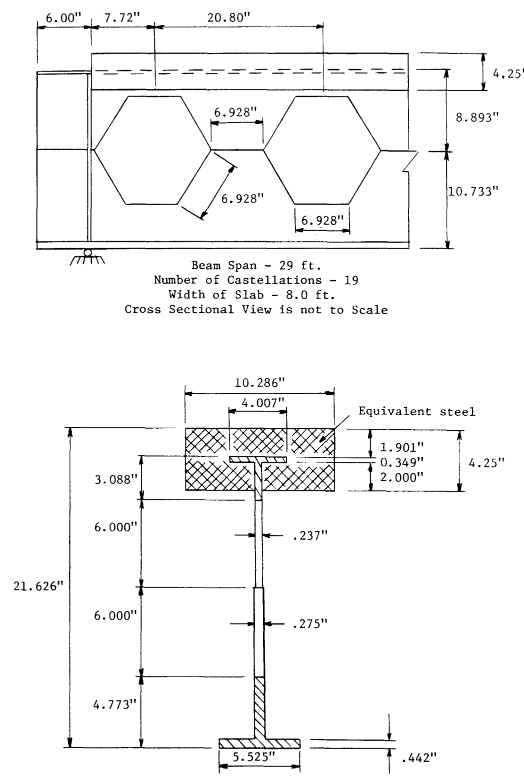

Fig. 4. Geometry of beam 20CCB25

INSTRUMENTATION

To evaluate the composite action, instrumentation was provided to measure slip between the concrete and steel, deflection of the composite beam, and strains in the beam and concrete slab.

Slip between the concrete and steel was measured with dial gages having a least reading of 0.0001 in. These gages were fastened to the beam web immediately below the deck slab, and the plunger of the gage acted on a metal clip cemented to the bottom of the deck. Ten gages were used between the center line of the span and the end support. In addition gages were placed on the top flange at each end to measure the slip at the ends of the deck slab.

A combination of transducers and dial gages were used to measure the vertical displacement. Deflections were measured to 0.0001 in. at nine locations between the center line of the span and one end support.

Linear and delta rosette electrical resistance metal-film strain gages were placed around two castellations on each beam. On both beams the castellations selected were at the center line of the span where high moment and low shear exists, and near the support where low moment and high shear occurs. Twenty-one linear gages and 13 rosettes were used on each beam.

Strains in the top of the concrete deck slab were measured with SR-4 wire strain gages. Six of these gages were located on the center line of the span, and two were located 12 in. from the center line of the span.

LOADING CONDITIONS

The beams were placed in the loading frame with the bottom flange supported by rollers. Each end was braced with box-type bracing to prevent lateral movement during slab construction and loading. An approximate uniform load was applied by hydraulic load cylinders. Loads from four load cylinders were transmitted to eight point loads, as shown in Fig. 5.

TEST PROCEDURE

Static live loads were applied to both beams in a similar fashion. Initial readings were taken at zero load on all instruments which were to measure slip, deflections, and strains. For beam 18CCB20, load increments of 5 kips were applied until the working live load of 15 kips or a uniform live load of 0.556 kip/ft was reached. Readings were taken after each increment of load was applied and after all loads were removed. The data were reduced and studied, then the procedure was repeated. When sufficient data were obtained at the working loads, the loads were increased at various increments until failure.

For beam 20CCB25, load increments of 5 kips were applied up to 20 kips, with a final increment of 2 kips, until the working live load of 22 kips or a uniform live load of 0.760 kip/ft was reached. After sufficient data were obtained at working loads, a repeated load was applied. The repeated live load varied from 11.2 kips to 28 kips. This is equivalent to a uniform live load of 0.386 kip/ft to 0.965 kip/ft and produced live load stresses of 6.5 ksi to 16.9 ksi at the bottom flange near the center line of the span. Adding dead load stress of approximately 8.5 ksi gives a total of 15.0 ksi to 25.4 ksi.

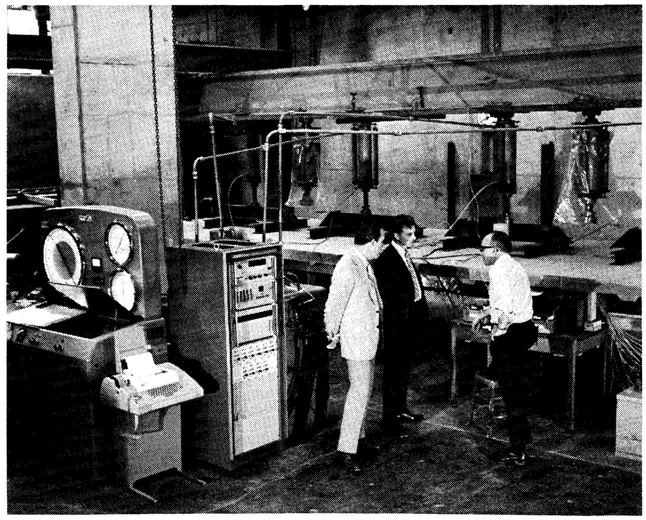

Fig. 5. General view showing loading cylinders and instrumentation

SLIP MEASUREMENT RESULTS

Maximum slip at the working live load of 15 kips was 0.0018 in. for beam 18CCB20 and was measured at the ends on the top of the beam. Maximum slip at the working live load of 22 kips was 0.0031 in. for the 20CCB25 beam and was measured at the ends on the top of the beam. Slip measurements for both beams decreased along the span and measured zero or nearly zero in many places, as shown in Figs. 6 and 7.

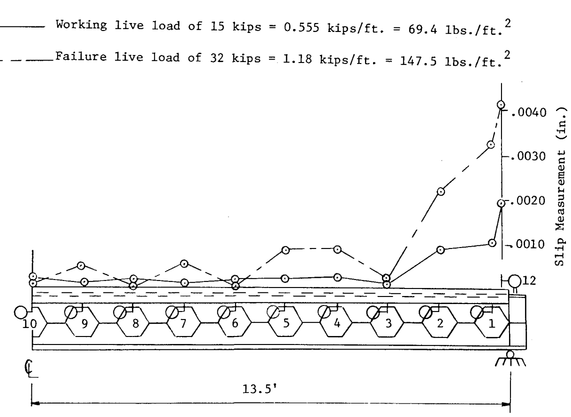

Fig. 6. Slip at working load and failure load for beam 18CCB20

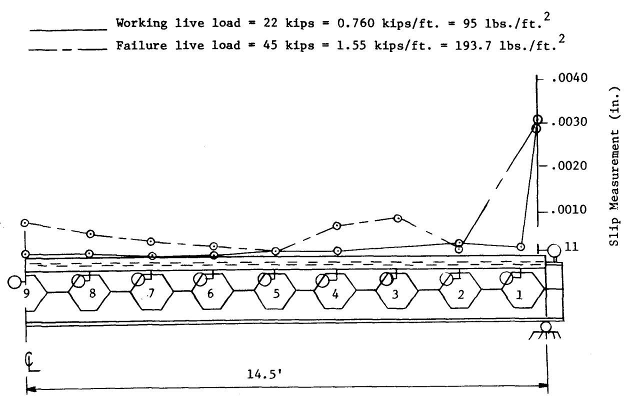

Fig. 7. Slip at working load and failure load for beam 20CCB25

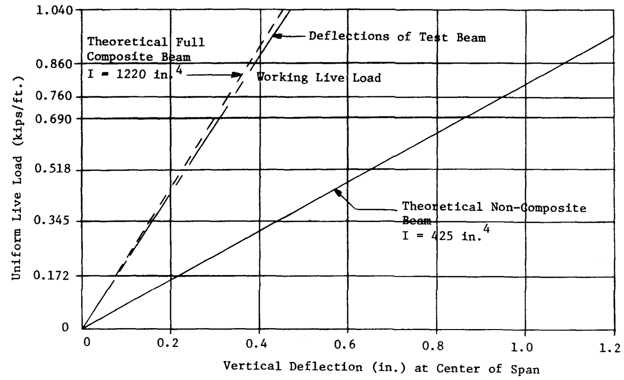

Fig. 9. Experimental deflections vs. theoretical deflections for beam 20CCB25

With these values for slip it is difficult to determine to what extent the adhesion has or has not failed. An examination of flexural stresses directly above each castellation shows some adhesion failure at the ends for beam 18CCB20, but not for beam 20CCB25, even though the amount of slip was greater. Slip for both beams was small enough along the span so that, at failure, yielding of steel occurred without adhesion failure.

In a typical floor system the slab is not simply supported but continuous over several spans. With a continuous slab, the slip at the ends of the beams may be greatly reduced, giving even greater composite action.

LOAD-DEFLECTION CHARACTERISTICS

The comparison of experimental deflections to theoretical deflections for a full composite beam and a non-composite beam is illustrated in Figs. 8 and 9. The theoretical deflections are computed from the equation $\Delta = 5wl^4/384EI$. From this comparison, beam 18CCB20 achieved 95 percent of full composite action and beam 20CCB25 achieved 99 percent.

A comparison of beam 18CCB20 with flange embedded can be made to beam 18CCB20 as tested by Tenbus,[7] where shear connectors were used and full composite action is generally accepted. Maximum deflection at center line for the flange embedded beam at a working live load of 15 kips is 0.290 in. while the beam with shear connectors had a deflection of 0.335 in. at a working live load of 20 kips. Although both steel beams were identical, the top flange of the composite sections were different, and the total depth of both composite sections were different. The embedded flange beam had a theoretical moment of inertia 18.5 percent less than the beam tested by Tenbus.

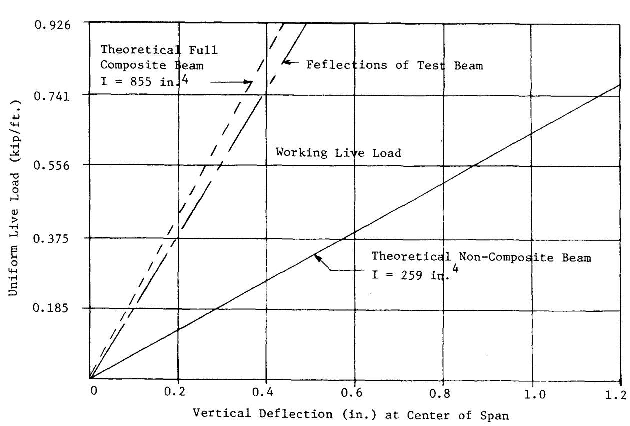

Fig. 8. Experimental deflections vs. theoretical deflections for beam 18CCB20

STRESS DISTRIBUTIONS

Comparison of stress distributions shows similar patterns for the 18CCB20 test beam with the flange embedded and the 18CCB20 beam tested by Tenbus with shear connectors. The beam with shear connectors had the slab resting on the top flange rather than embedded. Since the top flange and slab conditions are different, only the stress patterns for the lower chord are shown in Fig. 10. The stress patterns are in close agreement, even though the values are different due to the different loads and moments of inertia. This indicates that the flange embedded beam achieves nearly full composite action.

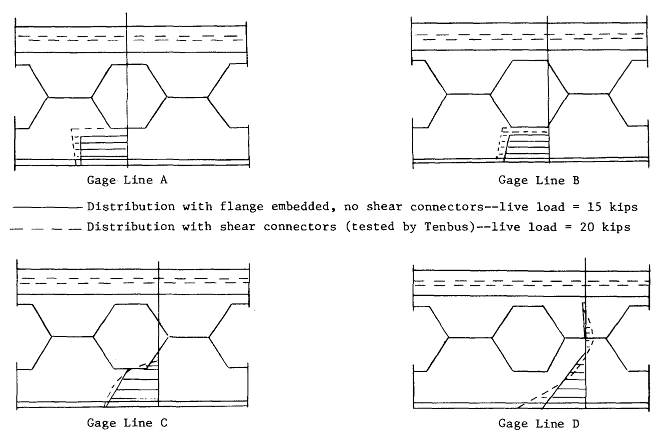

Fig. 10. Comparison of flexural stress distributions for beam 18CCB20 at center line

HORIZONTAL SHEAR STRESS

Since no shear connectors are present to resist the horizontal shear stress, the adhesion and friction between concrete and steel must provide enough resistance to develop the horizontal shear stress. The horizontal shear stress is calculated from the equation:

$$v = \frac{VQ}{I}$$

where:

- v = Horizontal shear, kips/in.
- V = External shear, kips
- Q = Statical moment of the area above or below the neutral axis taken about the neutral axis, $in.^3$
- I = Moment of inertia of composite section, $in.^4$

This will develop the horizontal shear in kips/in. and to convert to psi it is necessary to divide by the contact area between concrete and steel. This area is the perimeter of the top flange and stem which is embedded in concrete. A realistic approach to horizontal shearing stress is to take only one-half of the surface area under the flange when determining contact area between steel and concrete. Since air pockets form under the bottom of the top flange during vibration of the wet concrete, the reduced area gives a more realistic assumption. An attempt to eliminate the air bubbles by placing concrete to the bottom of the top flange, instead of completely covering the flange and then vibrating, did reduce some of the air pockets. However, complete elimination of air pockets is not possible in normal construction practice. Using the reduced area method, the equation for horizontal shear stress becomes:

$$v = \frac{VQ}{IP_{cs}}$$

where P_{cs} = reduced contact area of steel and concrete, in.

From this equation, the reduced area increases the horizontal shear stress in beam 18CCB20 to 82.8 psi and 88.0 psi for beam 20CCB25 at working loads.

Relating the computed shearing stress to available code values may help to further understand the calculated stresses. The American Concrete Institute building code of 1963 recommends a bond stress of not more than 160 psi for plain bars (Working Stress Design). Values obtained using the reduced area method reach only 55 percent of this code value.

REPEATED LOADS

Beam 20CCB25 was also tested with repeated loads to investigate a failure of adhesion after many load cycles. The maximum stress under repeated loading was 25.4 ksi, which is greater than the design stress of 22 ksi. Despite the overloading, there was no adhesion failure after 750,000 load cycles.

CONCLUSIONS

The following conclusions are based on the test results for the two castellated beams described above. Additional tests may be in order for shallower beams and beams that have different encased perimeter lengths.

1. Sufficient friction and natural adhesion can be developed with a steel beam partially encased in concrete to develop the horizontal shear stresses required to achieve composite action.

2. The composite action achieved in a partially encased beam has sufficient strength to develop yielding in the steel beam before the composite action fails.

3. Since air bubbles cannot be eliminated under the bottom of the top flange of the steel beam, the design method should use only one-half of the contact area under the top flange when determining horizontal shear stress between the steel and concrete. The following formula may be used for horizontal shear stress:

$$v = \frac{VQ}{IP_{cs}}$$

4. Repeated loads did not destroy the adhesion between concrete and steel after 750,000 cycles, even though the stresses from the repeated loads exceeded the design working stresses.

5. Slight adhesion failure did occur at the ends of the test beams. However, this did not affect the failure strength of the composite beam. This adhesion failure probably will not occur when the slab is continuous over several spans.

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