Composite Beams with Formed Steel Deck

JOHN A. GRANT, JR., JOHN W. FISHER, AND ROGER G. SLUTTER

DURING the past forty years, formed steel deck has become the most common floor system used in high rise steel frame structures.[1] A natural consequence of this floor system was the development of composite action between the steel beam and the concrete slab by means of shear connectors welded through the deck to the beam flange.

There are two conditions to be considered. The first condition is when the corrugations of the deck run parallel to the beam. It seems reasonable to assume that this condition can be modeled as a composite beam with a haunched slab. The limited experimental work available for this case indicates that the shear connection is not significantly affected by the ribs.[2,3] A check may be warranted to insure that shearing of the concrete will not occur on a failure plane over the top of the connectors.

The second condition is when the corrugations of the deck run perpendicular to the beam. Initial studies of this latter condition were made on a proprietary basis for specific products in building applications and thus were uncoordinated. Consequently, considerable variance among controlled parameters existed, making it difficult to draw any general conclusions. In 1967 a detailed study by Robinson[4] showed that for high, narrow ribs the shear capacity of the connector is a function of the rib geometry and is substantially less than the capacity of connectors embedded in a composite beam with a solid slab. However, this study, as well as others, indicated that for decking with small corrugations the shear capacity of the connector, and thus the behavior of the composite beam, is unaffected. In 1970, Fisher[5] summarized the investigations that had been conducted to date and proposed design criteria. Fisher concluded that composite beams could be modeled as having a haunched slab, equal in thickness to the solid part of the slab above the rib, except that the shear capacity of the connector is reduced. He modeled this reduction in shear capacity by the following formula:

John A. Grant, Jr. is Research Assistant, Fritz Engineering Laboratory, Lehigh University, Bethlehem, Pa.

John W. Fisher is Professor of Civil Engineering, Fritz Engineering Laboratory, Lehigh University, Bethlehem, Pa.

Roger G. Slutter is Professor of Civil Engineering, Fritz Engineering Laboratory, Lehigh University, Bethlehem, Pa.

$$Q_{rib} = A \left(\frac{w}{h}\right) Q_{sol} \leq Q_{sol} \qquad (1)$$

where

Q_{rib} = shear strength of connection in a rib
A = numerical coefficient (0.5 for beams)
w = average rib width
h = height of rib
Q_{sol} = shear strength of a connector in a solid slab

With the many uncontrolled and ill-defined variables in these early investigations, there was a need for additional research in this area. A research program was initiated at Lehigh University in 1971 involving 17 full scale beam tests. The work reported herein includes a detailed analysis of these 17 composite beams. Additionally, this analysis is supplemented by an evaluation of 58 beam tests reported by other investigators. The Lehigh tests involved steel beams acting compositely with lightweight concrete slabs cast on formed steel deck and connected by ¾-in. stud shear connectors. The variables considered were the yield strength of steel beam, the geometry of the deck, and the degree of partial shear connection. The other 58 tests treated variables such as weight and strength of concrete, diameter and height of stud shear connectors, type of slab reinforcement, and type of loading.

This report provides an evaluation of the shear capacity of stud connectors ¾-in. and smaller in diameter embedded in composite beams with formed steel deck, as well as the flexural capacity of the composite beams themselves. Additionally, the behavior of composite beams with or without formed steel deck is evaluated for working loads. Also, a comparison is made of connector capacity and beam behavior with existing design criteria.

DESCRIPTION OF TESTS

Test Program—The experimental program conducted at Lehigh consisted of tests on 17 simple span composite beams. The program was designed in accordance with the recommendations suggested in Refs. 5 and 6. A breakdown of the experiment design by specimens is given in Table 1.

Table 1. **Experiment Design by Specimen Designation**

Series	Rib Ht. h (in.)	Rib Width $w_1 = 1.5h$	$w_2 = 2.0h$
1A	1½	Spec 1A1R	Spec 1A5R
	2	Spec 1A2	Spec 1A6R
	3	Spec 1A3R	Spec 1A7
1B	1½	—	Spec 1B2
	3	Spec 1B1	—
1C	1½	Spec 1C1	—
	2	—	Spec 1C3
	3	Spec 1C2a	Spec 1C4
	3	Spec 1C2b	—
1D	1½	Spec 1D1	—
	1½	Spec 1D2	—
	3	—	Spec 1D3
	3	—	Spec 1D4

Series A consisted of six beams. It served as the basic series in the program, with average rib width-height ratios of 1.5 and 2. The beams were designed for an 80% partial shear connection, as defined by the AISC Specification.[7] Series B consisted of two mild steel beams, as all other beams were an intermediate grade. Series C consisted of five beams with low degrees of shear connection (below 50%). Series D consisted of four beams with larger rib slopes as their major variable.

Test Specimens—The composite specimens consisted of steel beams on simple spans of 24 ft or 32 ft, acting compositely with concrete slabs cast on formed steel deck. Details and material properties of the beams are provided in Table 2(A).

The slabs of the beams were made with structural lightweight concrete conforming to the requirements of ASTM C330 (*Specification for Lightweight Aggregates for Structural Concrete*). Concrete properties were maintained as constants within fabrication tolerances. Minimal reinforcement for all of the beams consisted of 6 × 6-#10/10 welded wire fabric placed at mid-depth of the slab above the ribs. The thickness of the solid part of the slab was a constant 2½ in. for all of the beams. The slab widths were proportioned as 16 times the full thickness of the slab plus the flange width of the steel beam. All slabs were cast without shoring.

The slabs were cast on 20 gauge galvanized steel deck without embossments. The rib heights of the deck were 1½, 2 or 3 in. for average rib width-height ratios of 1.5 and 2. The slopes of the ribs were a nominal 1 to 12, except for the series D beams which had 1 to 2 (1D2 and 1D4) and 1 to 3 (1D1 and 1D3) slopes. The steel deck was fabricated in widths of 24 or 36 in., with corresponding rib modules of 6 and 12 in.

Composite action between the steel beam and the slab was provided by the placement of ¾-in. shear connectors. All studs conformed to ASTM A108 specification and were welded through the steel deck to the beam flange in a staggered pattern. The stud layout is shown in Fig. 1. All welds were tested by "sounding" the studs with a hammer.

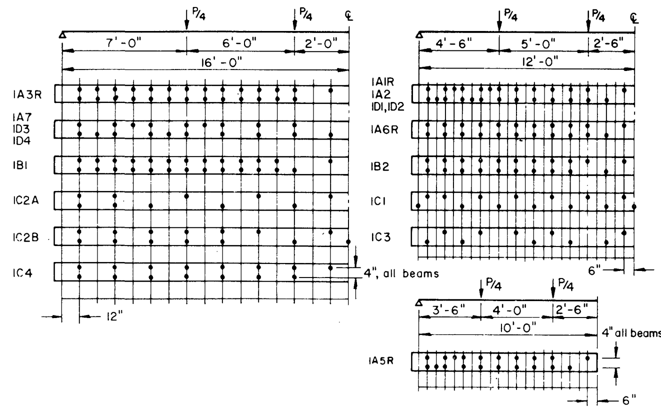

Fig. 1. Load point and stud layout

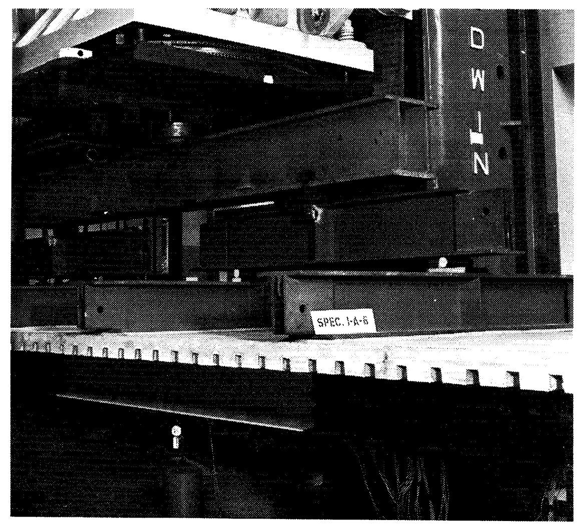

Fig. 2. Test setup

Questionable studs were given a 15 degree bend test. Faulty studs were replaced and retested. One or two studs were placed in a rib. The stud spacing was adjusted to accommodate the varying rib geometry, but never exceeded 24 in. All studs were embedded 1½ in. above the rib.

Test Procedure—Four point loading was used on all of the beams to provide shear and moment conditions comparable to uniform load conditions. The loads were about equally spaced, but varied slightly so that loads were applied over a rib and not over a void. The load points are shown in Fig. 1. Figure 2 shows a typical test setup.

The beams were loaded in increments up to their estimated working load, then cycled 10 times. After cycling, the beams were reloaded in increments to near the ultimate load. Near ultimate, load was applied to produce fixed increments of deflection. Loading was terminated once the plateau of the load-deflection curve was established and deflections became excessive.

Table 2(A). Summary of Lehigh Test Data

Beam	Steel Section	Beam Span (ft)	Slab Width (in.)	Slab[b] Depth (in.)	Rib Ht. (in.)	Avg. Rib Width (in.)	Stud Dia. (in.)	Stud Ht. (in.)	Studs[c] per Shear Span	Stud[d] Space (in.)	w_c[e] (pcf)	E_c[f] (ksi)	f'_c[g] (ksi)	f_y[h] (ksi) Flange	f_y[h] (ksi) Web
1A1R	W16 × 40	24.0	72.0	4.0	1.5	2.25	3/4	3.0	24	12.0	114.6	2030.	3.470	68.8	74.9
1A2	W16 × 40	24.0	80.0	4.5	2.0	3.0	3/4	3.5	24	12.0	117.9	2170.	3.710	57.5	61.7
1A3R	W16 × 40	32.0	96.0	5.5	3.0	4.5	3/4	4.5	26	24.0	115.0	1990.	3.250	65.6	68.2
1A5R	W16 × 45	20.0	72.0	4.0	1.5	3.0	3/4	3.0	18	12.0	117.9	2170.	3.710	66.1	68.3
1A6R	W16 × 45	24.0	80.0	4.5	2.0	4.0	3/4	3.5	20	12.0	126.0	2520.	4.930	66.2	67.5
1A7	W16 × 40	32.0	96.0	5.5	3.0	6.0	3/4	4.5	18	24.0	119.4	2290.	4.200	63.3	65.8
1B1	W16 × 58	32.0	96.0	5.5	3.0	4.5	3/4	4.5	25	24.0	122.8	1810.	3.750	32.2	34.9
1B2	W16 × 58	24.0	72.0	4.0	1.5	3.0	3/4	3.0	19	12.0	115.1	2160.	4.830	37.1	39.1
1C1	W16 × 40	24.0	72.0	4.0	1.5	2.25	3/4	3.0	11	12.0	116.6	2490.	4.350	57.7	62.4
1C2A	W16 × 40	32.0	96.0	5.5	3.0	4.5	3/4	4.5	9	24.0	113.3	2480.[a]	4.130	66.0	67.2
1C2B	W16 × 40	32.0	96.0	5.5	3.0	4.5	3/4	4.5	12	24.0	113.3	2510.[a]	3.990	66.2	69.1
1C3	W16 × 40	24.0	80.0	4.5	2.0	4.0	3/4	3.5	8	12.0	117.8	2500.	4.840	69.2	74.7
1C4	W16 × 45	32.0	96.0	5.5	3.0	6.0	3/4	4.5	14	24.0	118.7	2090.	3.250	64.9	68.2
1D1	W16 × 40	24.0	72.0	4.0	1.5	2.25	3/4	3.0	24	12.0	114.8	2030.	3.470	69.0	73.8
1D2	W16 × 40	24.0	72.0	4.0	1.5	2.25	3/4	3.0	24	12.0	115.1	2170.	4.610	55.4	60.6
1D3	W16 × 45	32.0	96.0	5.5	3.0	6.0	3/4	4.5	18	24.0	122.4	2200.	3.960	64.9	67.9
1D4	W16 × 40	32.0	96.0	5.5	3.0	6.0	3/4	4.5	18	24.0	124.8	2580.	4.850	64.3	68.7

[a] Computed by ACI formula
[b] Total thickness including height of rib
[c] Number of studs between point of maximum moment acting on the composite section and the nearest point of zero moment
[d] Maximum spacing between two connector groups in a shear span
[e] Concrete density (dry weight)
[f] Modulus of elasticity of concrete, measured by cylinder tests
[g] Compressive strength of concrete at test
[h] Static yield point of steel beam

Table 2(B). Summary of Other Test Data

Beam	Steel Section	Beam Span (ft)	Slab Width (in.)	Slab[b] Depth (in.)	Rib Ht. (in.)	Avg. Rib Width (in.)	Stud Dia. (in.)	Stud Ht. (in.)	Studs[c] per Shear Span (1)	(2)	(3)	(T)	Stud[d] Space (in.)	w_c[e] (pcf)	E_c[f] (ksi)	f'_c[g] (ksi)	f_y[h] (ksi) Flange	f_y[h] (ksi) Web
71-17(A1)	W12 × 19	21.0	68.0	4.00	1.50	2.25	3/4	3.0	6	0	0	6	24.0	145.0[a]	3780.	4.290	41.6	46.7
71-17(A2)	W12 × 19	21.0	68.0	4.00	1.50	2.25	3/4	3.0	0	6	0	12	24.0	145.0[a]	4340.	5.670	41.6	46.7
71-17(A3)	W12 × 19	21.0	68.0	4.00	1.50	2.25	3/4	3.0	9	0	0	9	24.0	145.0[a]	4340.	5.670	40.7	46.3
71-17(A4)	W12 × 19	21.0	68.0	4.00	1.50	2.25	3/4	3.0	0	6	0	12	36.0	145.0[a]	3600.	3.890	40.7	46.3
71-17(A5)	W12 × 19	21.0	68.0	5.00	1.50	2.25	3/4	4.0	0	6	0	12	36.0	145.0[a]	3600.	3.890	40.7	46.3
71-17(B1)	W12 × 19	21.0	68.0	4.00	1.50	2.25	3/4	3.0	21	0	0	21	6.0	145.0[a]	3940.	4.650	41.2	46.4
71-17(B2)	W12 × 19	21.0	68.0	4.00	1.50	2.25	3/4	3.0	11	0	0	11	12.0	145.0[a]	3395.	3.457	41.2	46.4
71-17(B3)	W12 × 19	21.0	68.0	4.00	1.50	2.25	3/4	3.0	16	0	0	16	12.0	145.0[a]	3370.	3.410	41.2	46.4
71-17(B4)	W12 × 19	21.0	68.0	4.00	1.50	2.25	3/4	3.0	21	0	0	21	6.0	145.0[a]	3430.	3.530	41.2	46.4
70-4	W16 × 26	30.0	70.0	4.00	1.50	2.13	3/4	3.0	14	0	0	14[a]	12.0	113.3[i]	2243.	3.175	39.5[j]	42.3[j]
70-3(A)	W12 × 27	15.0	48.0	5.50	1.50	1.94	3/4	4.5	9	3	0	15	6.0	118.0[i]	2933.	4.808	35.3[k]	39.5[k]
70-3(B)	W12 × 27	15.0	48.0	5.50	1.50	1.94	3/4	4.5	9	3	0	15	6.0	118.0[i]	2933.	4.808	35.3[k]	39.5[k]
66-11(42)	W8 × 15	20.0	44.0	4.00	1.50	2.25	3/4	3.0	4	0	0	4	24.0	145.0[a]	2850.	3.325	49.6[n,o]	
66-11(56)	W8 × 15	20.0	44.0	4.00	1.50	2.25	3/4	3.0	5	0	0	5	12.0	145.0	3219.	3.265	46.2[n,o]	
66-11(B)	W12 × 24	20.0	44.0	4.00	1.50	2.25	3/4	3.0	9	0	0	9	12.0	144.6	2825.	3.470	49.1[n,o]	
66-11(W)	W14 × 30	20.0	47.0	4.00	1.50	2.25	3/4	3.0	13	0	0	13	12.0	145.2	3270.	4.030	39.3[n,o]	

(Continued next page)

Table 2(B). Summary of Other Test Data (continued)

Beam	Steel Section	Beam Span (ft)	Slab Width (in.)	Slab[b] Depth (in.)	Rib Ht. (in.)	Avg. Rib Width (in.)	Stud Dia. (in.)	Stud Ht. (in.)	Studs[c] per Shear Span				Stud[d] Space (in.)	w_c[e] (pcf)	E_c[f] (ksi)	f'_c[g] (ksi)	f_y[h] (ksi)	
									(1)	(2)	(3)	(T)					Flange	Web
64-15(H1)	W12 × 27	15.0	48.0	4.00	1.50	4.50	3/4	3.0	12	0	0	12	12.0	115.0[a]	2519.	3.860	37.4[k]	45.6[k]
64-15(E1)	W12 × 27	15.0	48.0	4.00	1.50	4.50	3/4	3.0	5	4	0	13	12.0	115.0[a]	2566.	4.010	34.4[k]	39.3[k]
64-15(E2)	W12 × 27	15.0	48.0	4.00	1.50	4.50	3/4	3.0	0	8	0	16	24.0	114.7	2872.	5.020	34.1[k]	40.5[k]
67-38	W14 × 30	24.0	58.0	4.75	1.75	6.00	7/8	3.5	0	5	0	10[a]	30.0	114.0[a]	2230.[a]	3.100	37.8[j]	41.9[j]
71(EPIC)	W12 × 27	15.0	60.0	5.25	2.00	5.00	3/4	4.0	11	0	0	11	6.0	109.8	2116.	3.090	35.8[k]	—
72-12(75)	W12 × 58	25.0	72.0	6.23	3.00	7.25	3/4	4.5	0	7	0	14	16.0	116.0[i]	2628.	4.063	55.5[j]	61.8[j]
73(RF)	W14 × 30	25.0	96.0	6.25	3.00	4.00[l] 6.00	3/4	5.5	0	8	0	16	24.0	109.2	2620.	4.740	37.4[k]	39.2[k]
67-11(B1)	W12 × 27	15.0	48.0	5.50	3.00	4.06	3/4	5.0	0	4	0	8	26.0	116.0	2170.[m]	4.400	36.7[k]	41.4[k]
67-11(B2)	W12 × 27	15.0	48.0	5.50	3.00	4.06	3/4	5.0	0	4	0	8	26.0	116.0	2170.[m]	4.900	36.7[k]	41.4[k]
68-4(1)	W14 × 30	20.0	47.0	4.00	1.50	2.25	3/4	3.0	13	0	0	13	12.0	116.9	2590.[m]	4.300	36.2[k]	40.3[k]
68-5(2)	B16 × 26	30.0	45.5	4.00	1.50	2.25	3/4	3.0	14	0	0	14	24.0[a]	111.0	2483.[m]	3.400	41.2[k]	50.9[k]
69-1(3)	W14 × 30	20.0	47.0	4.00	1.50	2.25	3/4	3.0	12	0	0	12	12.0	115.3	2230.[m]	4.600	36.4[k]	38.7[k]
69-12(4)	W18 × 45	36.5	47.5	4.00	1.50	2.25	3/4	3.0	18	0	0	18	12.0	111.0	2756.[m]	5.100	37.3[j]	44.2[j]
70-31(A)	W14 × 30	19.0	48.0	4.00	1.50	2.25	3/4	3.0	8	5	0	18	12.0	116.0	2370.[m]	3.300	36.5[k]	36.5[k]
70-31(D)	W14 × 30	19.0	48.0	4.00	1.50	2.25	1/2	3.0	5	9	0	23	16.0	116.0	2370.[m]	3.300	36.5[k]	36.5[k]
70-31(C)	W18 × 60	35.5	72.0	6.00	3.00	5.63	3/4	5.0	16	1	0	18	12.0	116.0	2370.[m]	3.300	32.4[k]	37.5[k]
69-2(HR)	W10 × 21	21.0	50.0	6.20	3.00	2.63	3/4	5.0	6	0	0	6	12.0	145.0	3490.[m]	4.800	37.5[k]	38.8[k]
67-36(CU3)	W12 × 27	24.0	62.5	3.50	1.50	2.25	5/8	2.5	12	3	0	18	12.0	145.0	3250.	3.200	36.4[o]	36.4[o]
70-5(C2)	B14 × 22	24.0	60.0	6.00	2.00	3.00	3/4	4.0	11	0	0	11	6.0	145.0	3640.	4.000	36.0[a]	36.0[a]
65-19(BS12)	W12 × 27	15.0	48.0	4.00	1.31	2.25	3/4	3.0	14	0	0	14	9.0	145.0	3640.	4.000	36.0[k]	41.3[k]
65-19(BS11)	W12 × 27	15.0	48.0	4.00	.88	1.75	3/4	3.0	14	0	0	14	6.75	145.0	3640.	4.000	35.6[k]	42.0[k]
67-36(CU2)	W12 × 27	24.0	62.5	3.50	1.50	3.63	5/8	2.5	18	0	0	18	12.0	145.0	3730.	4.200	40.5[o]	40.5[o]
67-36(CU1)	W12 × 27	24.0	62.5	3.50	1.50	5.00	5/8	2.5	10	4	0	18	15.0	145.0	3770.	4.300	36.1[o]	36.1[o]
TEX-1	W16 × 50	32.0	96.0	6.25	3.00	6.00	3/4	4.5	0	13	0	26	12.0	109.0	2057.	3.000	38.4[k]	36.5[k]
TEX-2	W16 × 50	32.0	96.0	6.25	3.00	6.00	3/4	5.5	0	13	0	26	12.0	108.0	2283.	3.800	35.7[k]	45.2[k]
TEX-3	W16 × 50	32.0	96.0	6.25	3.00	6.00	3/4	6.0	0	13	0	26	12.0	106.0	2220.	3.800	35.8[k]	36.7[k]
TEX-4	W16 × 50	32.0	96.0	6.25	3.00	6.00	3/4	5.0	0	13	0	26	12.0	102.4	2023.	3.500	36.4[k]	36.4[k]
TEX-5	W16 × 50	32.0	96.0	6.25	3.00	6.00	3/4	5.0	0	13	0	26	12.0	102.4	2023.	3.500	38.9[k]	36.6[k]
TEX-6	W16 × 50	32.0	96.0	6.25	3.00	6.00	3/4	4.5	0	13	0	26	12.0	102.1	2042.	3.600	38.0[k]	37.4[k]
TEX-7	W16 × 50	32.0	96.0	6.25	3.00	6.00	3/4	5.0	0	13	0	26	12.0	106.6	2268.	3.900	39.7[k]	39.0[k]
TEX-8	W16 × 50	32.0	96.0	6.25	3.00	6.00	3/4	5.5	0	13	0	26	12.0	105.0	2259.	4.050	39.8[k]	39.0[k]
HHR-1-76	W16 × 45	32.0	95.0	5.50	3.00	6.00	3/4	5.0	0	6	0	12	24.0	144.0	3232.	4.260[m]	37.1[k]	42.2[k]
IR-1-76	W16 × 45	32.0	95.0	5.50	3.00	7.25	3/4	5.0	0	6	0	12	32.0	144.0	3112.	4.180[m]	37.0[k]	39.6[k]
HHR-2-76	W16 × 45	32.0	95.0	5.50	3.00	6.00	3/4	5.0	0	6	0	12	24.0	145.0	3540.	4.590[m]	36.8[k]	41.3[k]
IR-2-76	W16 × 45	32.0	95.0	5.50	3.00	7.25	3/4	5.0	0	6	0	12	32.0	144.0	3610.	4.730[m]	37.3[k]	39.9[k]
RF-1-76	W16 × 45	32.0	95.0	5.50	3.00	6.00	3/4	5.0	0	6	0	12	24.0	145.0	3065.	4.400[m]	36.8[k]	42.2[k]
RF-2-76	W16 × 45	32.0	95.0	5.50	3.00	6.00	3/4	5.0	0	6	0	12	24.0	143.0	3142.	4.310[m]	36.9[k]	40.9[k]
72-12(80)	W12 × 65	25.0	72.0	7.00	3.00	7.25	3/4	4.5	0	0		21	16.0	115.6[i]	2548.	3.860	33.5[j]	37.5[j]
75-16	W16 × 40	30.0	87.0	5.00	3.00	6.00	3/4	4.5	12	0	0	12	12.0	145.0	2375.	3.290[m]	41.7[n]	48.8[n]
175-75	W24 × 55	34.9	103.0	6.25	3.00	7.25	3/4	5.0	4	7	0	18	16.0	116.0[i]	2622.	4.040	38.9[j]	38.1[j]
174-75	W24 × 61	34.9	103.0	9.00	3.00	7.25	3/4	7.0	0	5	6	28	16.0	144.0[i]	3716.	4.246	35.8[j]	36.7[j]
16-76	W21 × 44	40.0	94.5	5.50	3.00	6.75[l] 7.25	3/4	4.9	0	10	0	20	30.0	145.0	3394.	3.470	40.6[j]	42.4[j]

[a] Estimated
[b] Total thickness including height of rib
[c] Number of studs between point of maximum moment acting on the composite section and the nearest point of zero moment: (1) No. of ribs with 1 stud; (2) No. of ribs with 2 studs; (3) No. of ribs with 3 studs; (T) Total no. of studs
[d] Maximum spacing between two connector groups in a shear span
[e] Concrete density (dry weight)
[f] Modulus of elasticity of concrete, computed by ACI formula
[g] Compressive strength of concrete at test
[h] Yield point of steel beam
[i] Wet weight
[j] 0.2% offset yield stress
[k] Static yield stress
[l] Two types of deck used
[m] Measured by cylinder tests
[n] Dynamic yield stress
[o] Averaged, flange and web

Instrumentation—The test beams were instrumented to measure deflection, slip, and strain, as shown in Fig. 3. A 0.001-in. dial gage was used to measure the deflection at the midspan and the slip between the steel beam and the slab at each end of the beam. Slips were measured at selected points along the span with electrical slip gages. The relative horizontal movements of rods embedded in the slab on either side of a rib were taken to eliminate the effect of rib rotation. These measurements were then averaged to obtain the slip at the rib. Six electrical strain gages were placed on a section in the same plane as the rods to measure the strains in the steel beam.

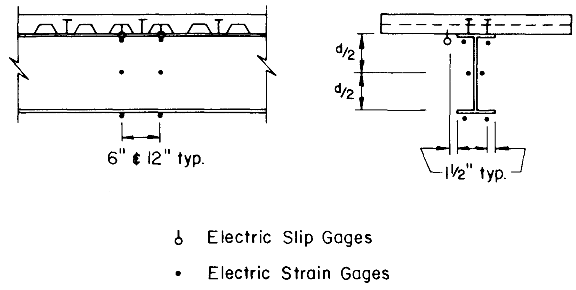

Fig. 3. Instrumentation

THEORETICAL CONSIDERATIONS

Elastic Behavior—The theoretical basis of elastic behavior for composite beams is essentially that outlined in Ref. 8, with modifications as adapted in Ref. 9 for beams with formed steel deck.

Equal curvatures are assumed for the slab and the steel beam. The horizontal forces, F, transferred between the slab and the beam by the connectors, are assumed to act at the centroids of the solid part of the slab and the beam cross sections as shown in Fig. 4. The total internal moment is equal to the sum of the individual moments in the slab and beam, M_c and M_s, respectively, plus the additional couple produced by the horizontal forces, F. Thus,

$$M = M_c + M_s + Fz \tag{2}$$

where z is the distance between the centroids of the slab and beam cross sections.

For the case where there is no slip between the slab and the beam, F is a maximum. If the connectors have some degree of flexibility, they deform and permit slip between the slab and the beam, which decreases F. When there are no connectors, F approaches zero. There is always some small force transfer due to friction between the slab and the beam.

When there is no shear connection, Eq. (2) reduces to

$$M = M_c + M_s \tag{3}$$

Assuming equal curvatures

$$M_c = M_s\,(I_c/nI_s) \tag{4}$$

where

n = modular ratio, E_s/E_c

E_s, E_c = moduli of elasticity of the beam and slab, respectively

I_s, I_c = moments of inertia of the steel beam and the solid part of the slab, respectively

The value of (I_c/nI_s) is generally less than 5% for composite beams. Thus the total internal moment for zero degrees of partial shear connection is essentially that of the steel beam alone.

From Eq. (3) and the assumption that curvatures, ϕ, are equal in the slab and the steel beam, it may be concluded that the stiffness of a composite beam at zero degrees of partial shear connection is essentially that of the steel beam alone. This conclusion is in agreement with the effective moment of inertia equation provided in Supplement No. 3 to the AISC Specification.[10]

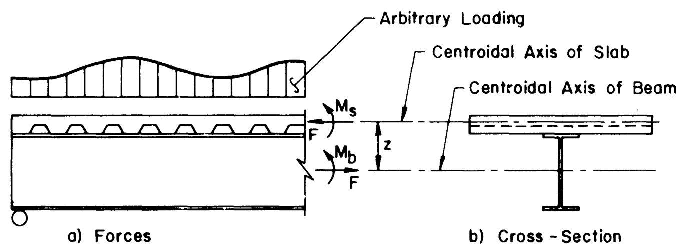

Fig. 4. Internal forces in a composite beam

Since the slab moment is so small, it is obvious that, for the limiting case of zero partial shear connection, the steel beam is carrying the load which is also compatible with the design provisions of the AISC Specification.[7]

When F is not zero, it becomes a key parameter governing the behavior of the connectors as well as the composite beam. Connector forces, beam deflection, and the stresses in the slab and steel beam are all functions of F. Consequently, if a relationship could be established between F and degree of partial shear connection, then the "properties" used to evaluate working load behavior, such as an effective section modulus or moment of inertia, could be related to the degree of partial shear connection as well.

If only elastic behavior is considered, values of F for various external loading and support conditions can be found from a solution of a governing differential equation. The solution for F is a function of a dimensionless interaction coefficient, $1/C$, and the loading condition. The interaction coefficient, $1/C$, is a function of the geometrical and material properties of the composite cross section, the span, L, the spacing of the connectors, s, and the connector stiffness modulus, k. Average values of $1/C$ can be established for a given loading condition for determining strains and deflections in a composite beam.[8]

At ultimate load, the force in the slab is the sum of the shear capacities of the connectors in the shear span of the composite beam, but is never larger than the maximum force that the slab or the steel beam can sustain.[6] The 1969 AISC Specification[7] relates this force to a working design parameter, $V'h/Vh$, which is the degree of partial shear connection. Vh is equal to one-half the total force possible in either the slab or the steel beam, whichever is smaller. $V'h$ is equal to the number of connectors in the shear span times the allowable load per connector. But this allowable load is approximately half the ultimate load.[11] Thus the ultimate force in the slab is equal to $2 \times V'h$ up to a limit of $2 \times Vh$.

The slab force, F, can be related to the degree of shear connection, $V'h/Vh$, through the interaction coefficient, $1/C$. The partial shear connection, $V'h$, and the allowable load per connector determine an average spacing, s, and an average connector stiffness modulus, k. Together with the geometrical and material properties of the composite cross section and the span, s and k determine an average value of $1/C$ for the beam. Since F is a function of $1/C$, it can be related to $V'h/Vh$.

Since the strains and deflections of the composite beam are functions of F, an effective section modulus or an effective moment of inertia can be related to $V'h/Vh$ for a given loading condition. For different loading conditions, F and $V'h$ are affected similarly. Hence, a general relationship between an effective section modulus or moment of inertia and $V'h/Vh$ can be established. The advantage of such a relationship is that stresses and deflections can be predicted in terms of the design parameter $V'h/Vh$ and thus be based on the capacity of the slab and beam, rather than the localized overloading of a connector.

Ultimate Strength—The flexural capacity of the test beams reported herein was determined essentially from the model suggested by Slutter and Driscoll[6] for composite beams with flat soffit slabs. However, the slab force was assumed to act at the centroid of the solid portion of the slab above the top of the ribs, and not at the center of the concrete stress block.

In many instances, the location of the slab force made little difference in the computed flexural capacity. For beams designed fully composite with the concrete slab governing the shear connection, the center of the stress block coincides with the centroid of the solid portion of the slab. However, for beams with low degrees of partial shear connection and/or high ribs, the location of the stress block has a significant influence on capacity.

For composite beams, with or without formed steel deck, there is loss of interaction, or slip, between the slab and the steel beam before developing the flexural capacity. This slip has little effect on the shear capacity of the connectors. However, it does affect the location of the slab force. Without any connection at all, the very small compressive stress resultant would lie somewhere in the upper half of the full slab depth. However, with the bottom of the slab constrained by the presence of shear connectors, the location of the stress resultant in the slab drops. The assumption that the stress resultant acts at the centroid of the solid portion of the slab seems to more adequately account for all cases involving composite beams with formed steel deck.

Robinson[9] has compared this difference in the assumed location of the slab force for a beam with 3-in. ribs and about 30% partial shear connection. He found that applying the method in Ref. 6 directly provided an estimated capacity 3% higher than the test data, and that assuming the slab force to act at the center of the solid slab above the ribs underestimated the capacity. However, he did not include the force on the shear connector directly under the load point, which falls at the edge of the shear span. Had this connector been included, the beam capacity would be overestimated by 9%. With the slab force acting at the center of the solid slab above the rib, the capacity would be overestimated by 1%. Strain measurements taken at the load points in this beam at ultimate load are shown in Fig. 18 of Ref. 9 and confirm the location of the stress resultant in the slab as near the mid-depth of the solid portion of the slab. A similar conclusion was drawn from the Lehigh test beams.

BEHAVIOR OF COMPOSITE BEAMS WITH METAL DECK

Ductility—Typical load-deflection curves for several of the tests are shown in Fig. 5. Also shown on the plots are two idealized elastic-plastic load-deflection curves. The elastic portion of the upper idealized curve assumes complete interaction between the slab and the beam. The plastic plateau of that curve is the ultimate load for a partial shear connection with a reduced connector capacity defined in Eq. (1). The lower idealized curve is adjusted to account for an effective moment of inertia in the elastic range, in accordance with Eq. (8) (see also Ref. 10). The plastic plateau for that curve reflects a modified connector capacity, as will be discussed later.

A significant aspect of these beams is their ductility. This ductility is demonstrated by the large deflections shown in the load-deflection plots in Fig. 5, even for beams with low degrees of partial shear connection. This ductility is also shown in Fig. 6, a photograph of a typical beam after testing. All of the test beams sustained maximum deflections between 8 and 22 in. These deflections correspond to more than 10 times the deflection at working load in all but two cases. Maximum vs. working load deflection ratios are shown in Table 3(A). Such large deflections were permitted by the formation of a plastic hinge near the midspan in all of the beams. Figure 7 shows a plastic hinge for one of the test beams. The formation of these plastic hinges, which produced the desired ductility, could only have been possible with a ductile shear connection which permitted redistribution of the slab force along the span of the beam.

Shear connectors were instrumented at selected points along each of the beams. Data from the beams have been analyzed and confirm the ductility of the shear connection.

Table 3(A). Summary of Lehigh Test Results

Beam	M_d [a]	P_{max} [b]	M_{max} [c]	Δ_{max} [d] / $\Delta W.$	$V'h$ [e,f]	$V'h/Vh$ [e,f]	M_u [f,g]	M_{max}/M_u	$V'h$ [e,g]	$V'h/Vh$ [e,h]	M_u [g,h]	M_{max}/M_u
1A1R	16.6	169.4	609.5	8.6	437.4	0.824	659.1	0.93	445.9	0.840	661.4	0.92
1A2	21.0	142.4	519.9	10.3	344.7	0.546	551.8	0.94	369.3	0.585	559.3	0.93
1A3R	42.8	107.7	581.3	13.1	230.3	0.347	591.7	0.98	230.9	0.348	591.9	0.98
1A5R	12.8	270.5	756.7	10.0	383.0	0.675	679.3	1.11	383.0	0.675	679.3	1.11
1A6R	23.8	192.4	697.2	12.6	445.4	0.531	709.2	0.98	446.2	0.532	709.4	0.98
1A7	49.3	93.2	515.3	11.0	244.2	0.314	585.6	0.88	267.5	0.344	597.8	0.86
1B1	47.4	86.4	479.5	17.0	221.7	0.402	434.3	1.10	226.1	0.410	435.9	1.10
1B2	18.9	138.4	503.3	17.7	436.6	0.685	518.7	0.97	436.6	0.685	518.7	0.97
1C1	16.8	140.6	508.9	17.6	234.7	0.353	509.4	1.00	260.6	0.392	518.7	0.98
1C2A	42.2	96.4	524.2	10.7	94.4	0.117	493.9	1.06	116.9	0.145	512.2	1.02
1C2B	22.1	99.0	537.2	14.3	125.2	0.154	527.7	1.02	134.1	0.165	534.7	1.01
1C3	49.7	158.6	577.2	14.5	176.6	0.215	569.5	1.01	195.5	0.238	581.4	0.99
1C4	16.6	114.2	620.7	14.1	168.9	0.255	602.7	1.03	168.9	0.255	602.9	1.03
1D1	50.9	166.6	599.7	7.8	437.4	0.824	656.2	0.91	445.9	0.840	658.5	0.91
1D2	51.2	142.4	515.0	13.9	490.5	0.716	566.9	0.91	500.8	0.731	569.3	0.91
1D3	50.9	114.6	623.9	11.2	235.7	0.292	646.8	0.97	257.5	0.319	660.3	0.95
1D4	51.2	95.6	529.2	11.2	268.7	0.336	612.6	0.86	294.3	0.368	624.8	0.85

[a] Dead load moment, kip-ft

[b] Maximum measured test load including weight of load beams, kips

[c] Computed flexural capacity, kip-ft

[d] Ratio of measured deflection at termination of test to measured deflection at working load. (Working load moment determined by S_{eff} as defined in Ref. 7).

[e] $V'h = \Sigma Q_u$; Q_u = capacity of shear connection, kips; $Vh = A_s f_y$ or $0.85 f'_c b (t - h)$ (Ref. 7)

[f] $Q_u = 0.6\left(\frac{H-h}{h}\right)\left(\frac{w}{h}\right) Q_u^s \leq Q_u^s$

where

H = height of stud

h = height of rib

w = average width of rib

$Q_u^s = 1.106 A_s f'_c{}^{0.3} E_c{}^{0.44}$ = capacity of shear connection in solid slab, kips (Ref. 11)

[g] Predicted flexural capacity, kip-ft

[h] $Q_u = \frac{0.85}{\sqrt{N}}\left(\frac{H-h}{h}\right)\left(\frac{w}{h}\right) Q_u^s \leq Q_u^s$

where N = number of studs in a rib (other parameters same as above)

Table 3(B). Summary of Other Test Results

Beam	M_{max} [c]	$V'h/Vh$ [e,f]	M_u [f,g]	M_{max}/M_u	$V'h/Vh$ [e,h]	M_u [g,h]	M_{max}/M_u	Ref.
71-17 (A1)	144.3	0.621	159.1	0.91	0.690	163.3	0.88	12
71-17 (A2)	165.2	1.000	181.6	0.91	1.000	181.6	0.91	12
71-17 (A3)	168.5	0.957	176.5	0.96	1.000	178.9	0.94	12
71-17 (A4)	167.5	1.000	178.9	0.94	1.000	178.9	0.94	12
71-17 (A5)	191.5	1.000	189.1	1.01	1.000	189.1	1.01	12
71-17 (B1)	195.8	1.000	180.2	1.09	1.000	180.2	1.09	12
71-17 (B2)	156.0	1.000	180.2	0.87	1.000	180.2	0.87	12
71-17 (B3)	181.8	1.000	180.2	1.01	1.000	180.2	1.01	12
71-17 (B4)	199.3	1.000	180.2	1.11	1.000	180.2	1.11	12
70-4	266.4	0.782	260.2	1.02	0.920	270.5	0.99	18
70-3 (A)	223.5	1.000	230.8	0.97	1.000	230.8	0.97	19
70-3 (B)	233.9	1.000	230.8	1.01	1.000	230.8	1.01	19
66-11 (42)	102.5	0.381	91.3	1.12	0.423	93.9	1.09	20
66-11 (56)	108.7	0.535	93.4	1.16	0.595	96.4	1.13	20
66-11 (B)	127.8	0.938	143.8	0.89	1.000	146.7	0.87	20
66-11 (W)	281.8	0.882	270.1	1.04	0.980	278.0	1.01	20

(Continued next page)

Table 3(B). Summary of Other Test Results (continued)

Beam	M_{max}[c]	$\frac{V'h}{Vh}$[e,f]	M_u[f,g]	$\frac{M_{max}}{M_u}$	$\frac{V'h}{Vh}$[e,h]	M_u[g,h]	$\frac{M_{max}}{M_u}$	Ref.
64-15 (H1)	243.9	0.862	222.5	1.10	0.862	222.5	1.10	21
64-15 (E1)	238.1	1.000	208.7	1.14	1.000	208.7	1.14	21
64-15 (E2)	243.4	1.000	209.9	1.16	1.000	209.9	1.16	21
67-38	303.0	0.796	276.2	1.10	0.796	276.2	1.10	22
71 (EPIC)	241.3	0.770	207.6	1.16	0.770	207.6	1.16	23
72-12 (75)	528.2	0.301	568.7	0.93	0.301	568.9	0.93	24
73 (RF)	306.2	0.985	322.4	0.95	0.986	322.6	0.95	25
67-11 (B1)	186.1	0.318	182.8	1.02	0.319	182.9	1.02	26
67-11 (B2)	181.9	0.329	184.1	0.99	0.329	184.1	0.99	26
68-4 (1)	242.3	0.840	257.9	0.94	0.934	265.0	0.91	27
68-5 (2)	283.7	0.843	293.5	0.97	0.937	300.9	0.94	28
69-1	252.0	0.752	247.4	1.02	0.836	253.8	0.99	29
69-12 (4)	441.2	0.818	491.0	0.90	0.909	502.6	0.88	30
70-31 (A)	263.8	1.000	260.0	1.02	1.000	260.0	1.02	31
70-31 (D)	252.8	0.610	229.8	1.10	0.625	231.1	1.09	31
70-31 (C)	264.3	0.476	563.4	1.11	0.617	599.4	1.04	31
69-2 (HR)	129.7	0.253	115.5	1.12	0.359	127.3	1.02	9
67-36 (CU3)	199.1	0.630	181.2	1.10	0.805	192.5	1.03	32
70-5 (C2)	238.3	1.000	211.4	1.13	1.000	211.4	1.13	33
65-19 (BS12)	220.9	1.000	216.4	1.02	1.000	216.4	1.02	34
65-19 (BS11)	228.8	1.000	210.9	1.09	1.000	210.9	1.09	34
67-36 (CU2)	233.9	1.000	227.5	1.03	1.000	227.5	1.03	32
67-36 (CU1)	250.7	1.000	202.8	1.24	1.000	202.8	1.24	32
TEX-1	499.5	0.550	487.2	1.03	0.551	487.4	1.03	36
TEX-2	663.2	0.982	612.3	1.08	0.982	612.3	1.08	36
TEX-3	634.2	1.000	564.7	1.12	1.000	564.7	1.12	36
TEX-4	554.8	0.788	523.8	1.06	0.789	524.1	1.06	36
TEX-5	582.8	0.755	539.2	1.08	0.756	539.5	1.08	36
TEX-6	534.7	0.577	495.0	1.08	0.578	495.3	1.08	36
TEX-7	604.6	0.790	567.8	1.06	0.792	568.1	1.06	36
TEX-8	639.9	0.996	615.8	1.04	0.996	615.8	1.04	36
HHR-1-76	437.1	0.481	438.2	1.00	0.482	438.4	1.00	38
IR-1-76	475.9	0.579	448.5	1.06	0.580	448.7	1.06	38
HHR-2-76	435.4	0.517	440.8	0.99	0.518	441.0	0.99	38
IR-2-76	471.0	0.637	463.4	1.02	0.639	463.6	1.02	38
RF-1-76	447.2	0.472	441.0	1.01	0.473	441.2	1.01	38
RF-2-76	450.6	0.479	436.0	1.03	0.480	436.1	1.03	38
72-12 (80)	423.4	0.534	474.2	0.89	0.437	445.3	0.95	24
75-16	404.0	0.316	390.9	1.03	0.448	417.3	0.97	39
175-75	790.6	0.650	782.8	1.01	0.656	784.4	1.01	40
174-75	1084.0	1.000	967.7	1.12	1.000	967.7	1.12	40
16-76	687.3	0.824	622.5	1.10	0.825	622.8	1.10	41

[c] Computed flexural capacity, kip-ft

[e] $V'h = \Sigma Q_u$; Q_u = capacity of shear connection, kips

$Vh = A_s f_y$ or $0.85 f'_c b(t - h)$ (Ref. 7)

[f] $Q_u = 0.6 \left(\frac{H-h}{h}\right)\left(\frac{w}{h}\right) Q_u^s \leq Q_u^s$

where

H = height of stud

h = height of rib

w = average width of rib

$Q_u^s = 1.106 A_s f'_c{}^{0.3} E_c{}^{0.44}$ = capacity of shear connection in solid slab, kips (Ref. 11)

[g] Predicted flexural capacity, kip-ft

[h] $Q_u = \frac{0.85}{\sqrt{N}}\left(\frac{H-h}{h}\right)\left(\frac{w}{h}\right) Q_u^s \leq Q_u^s$

where N = number of studs in a rib (other parameters same as above)

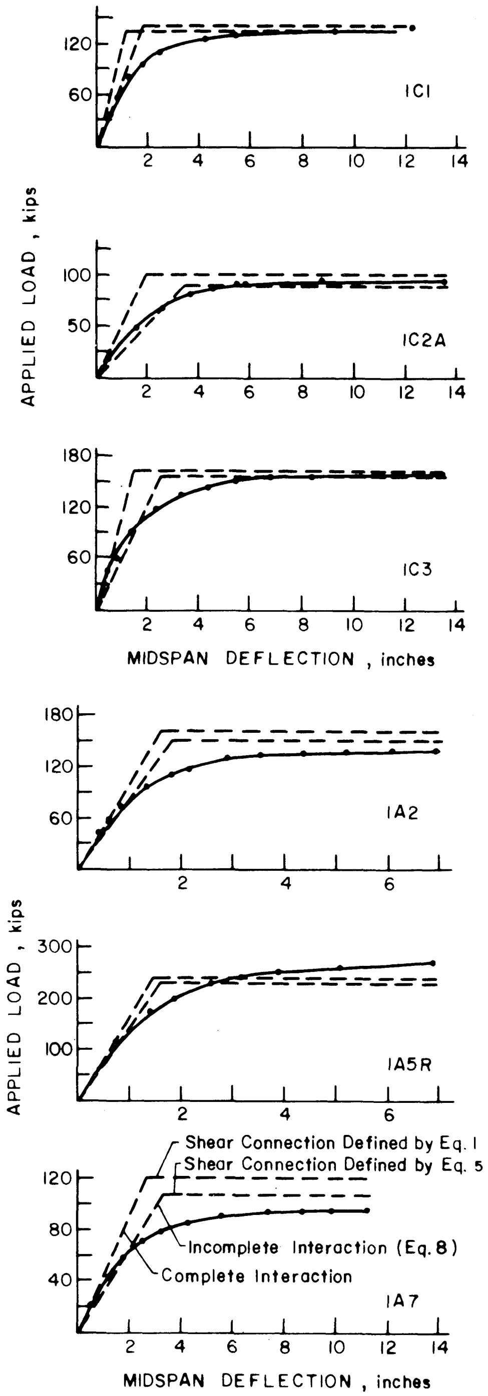

Fig. 5. Load-deflection curves

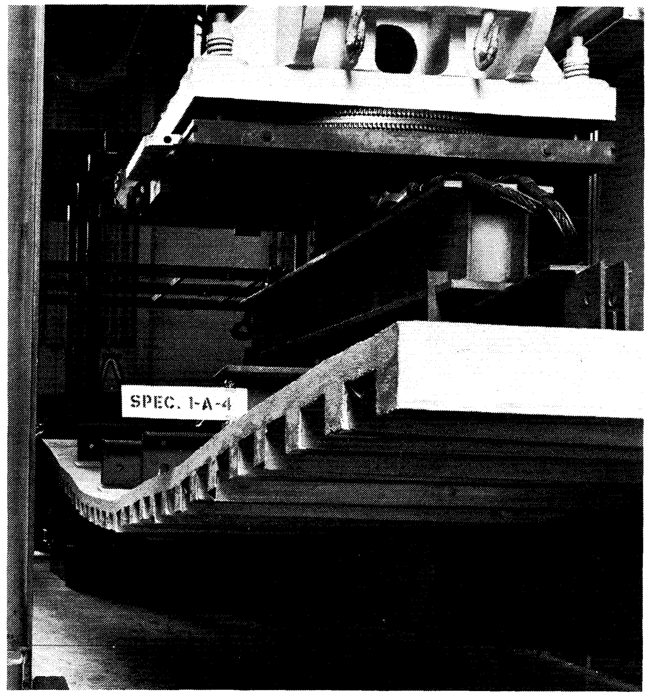

Fig. 6. Beam deflection at end of test

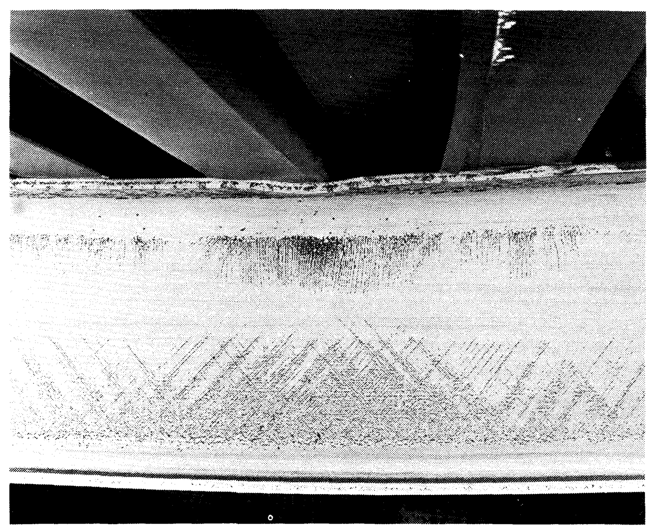

Fig. 7. Plastic hinge

Figure 8 shows some typical connector force-slip curves of one of the beams. The connector force was determined from equilibrium of forces on either side of the connector as derived from the strain measurements on the steel beam. The connector force is plotted against the corresponding average slip at the connector. As the curves indicate, the ultimate strength of individual connector groups varied considerably. However, all exhibited ductile behavior which permitted the redistribution of forces through the slab and thus a ductile composite beam. This redistribution of forces permits the prediction of an average connector capacity for the beam, such as suggested in Ref. 5.

The reason the shear connectors did not fail prematurely can be attributed to the relatively wide slabs used in the Lehigh test beams. With wide slabs, the failure surface initiates as a shear cone around the connector and pene-

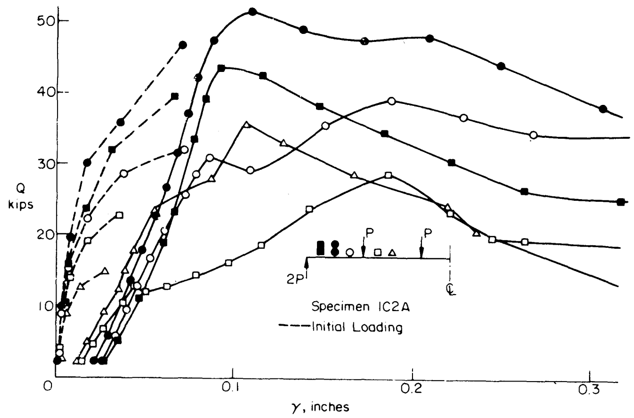

Fig. 8. Connector force vs. slip

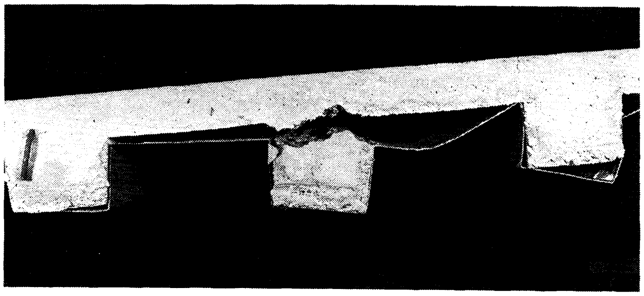

Fig. 9. Horizontal rib cracking

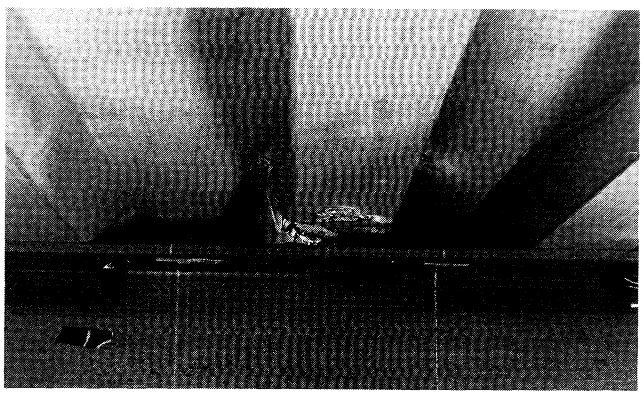

Fig. 10. Punch-through failure mode

trates through the slab to the top of the rib. From there, the failure surface propagates through the rib along a path of least resistance to a natural termination. With narrow slabs, the failure surface is smaller (a horizontal plane at the top of the rib), resulting in a weaker connection. With the wide slabs, there were very few incidents of horizontal rib cracking as shown in Fig. 9. The predominant connector failure mode was a punch-through failure, as shown in Fig.

Fig. 11. Punch-through failure, deck removed

10 for one of the beams after completion of testing. A view of this type of failure is again shown in Fig. 11 with the deck removed. Figure 11 shows the full development of the failure surface, which would have been prematurely truncated with a narrow slab. In these tests the slab widths were taken as 16 times the full thickness of the slab, including rib height, plus the width of the steel beam flange. Previous investigators[5,12] have suggested using this slab width for beam tests and for design, because it provides an upper limit connector ductility and capacity and more closely simulates the slab-beam interaction in an actual structure. Strain measurements across the slab width have indicated that shear lag is no more severe in a ribbed slab than in a solid slab.[5,13]

Flexural and Connector Capacity—The ultimate moment that each of the test beams sustained is listed in Table 3(A). Unfortunately, the connector model suggested in Ref. 5 [see Eq. (1)] for determining the flexural capacity of the composite beams proved unsatisfactory. Figure 12 shows the variation between test moment and theoretical moment, using this model for all 17 beams. The test moment is nondimensionalized by the predicted moment and plotted against the degree of partial shear connection. Despite the obvious fact that several of the beams fall below their predicted capacity, the plot also shows that several beams with very low degrees of partial shear connection can obtain their predicted capacity. This observation has been made by Robinson[12] as well. Similarly, rib slope and yield strength of the steel beam did not appreciably affect the beam capacity, as can be seen in Figs. 12 and 16. Alternatively, the apparent ineffectiveness of the rib slope can be seen from a comparison of beam 1AR with 1D1 and 1A7 with 1D4. Except for the rib slope of the steel deck, these beams are similar and produced similar ratios of actual to predicted beam capacities. It is apparent that the connector model must consider other variables.

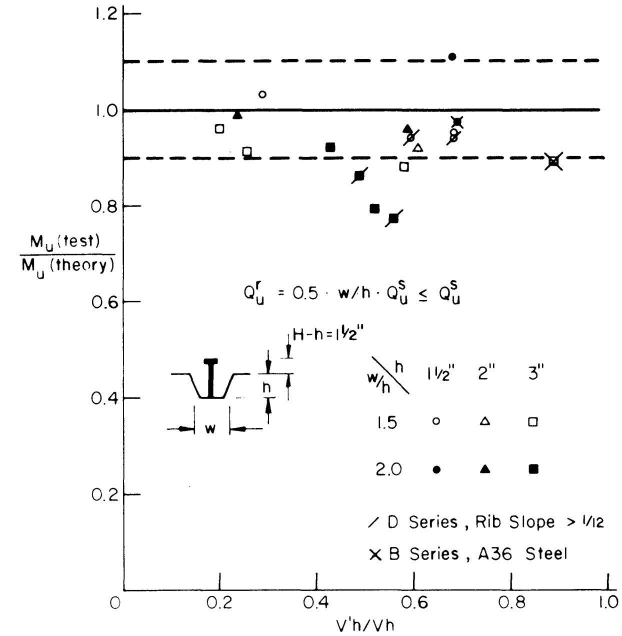

Fig. 12. Moment ratio vs. degree of partial shear connection for Eq. (1)

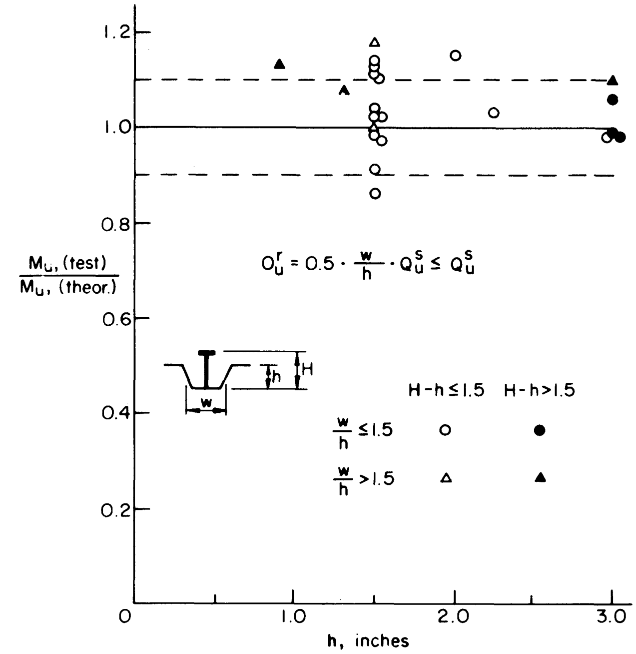

Fig. 14. Moment ratio vs. rib height for Eq. (1) with Ref. 5 data base

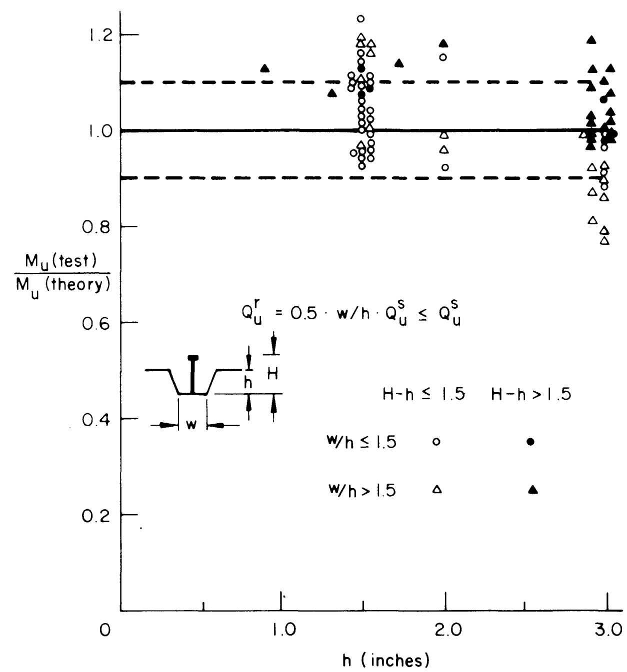

Fig. 13. Moment ratio vs. rib height for Eq. (1)

One such variable was found to be the height of the rib. Figure 13 shows a similar plot of the moment ratio as a function of the rib height. It includes the 17 Lehigh tests and the 58 tests conducted by other investigators with various other controlled parameters. The plot shows that the connector model defined by Eq. (1) is inadequate for the rib heights of 3 in. and is slightly conservative for rib heights of 1½ in. It is apparent, however, that the average rib width-height ratio remains as a major variable, as indicated by the relatively even dispersion of the symbols depicting this parameter throughout the clusters of data points.

Included in the 58 beam tests supplementing the 17 Lehigh tests are most of the beams comprising the data base that was used in Ref. 5. A few of the beams were excluded because either a premature connector failure prevented the development of a plastic hinge in the steel beam or because the beams were small scale with small shear connectors and did not simulate full size beam behavior. Figure 14 summarizes the data base used in Ref. 5. It is apparent that the connector model given by Eq. (1) was reasonable considering the data base at the time. Very few of those beams had rib heights of 3 in.

A reexamination of this test data, as shown in Fig. 14, indicates that all of the beams with 3-in. deck except one had a stud embedment length greater than 1½ in. above the rib. This effect of embedment length can also be seen in Fig. 13. Beams with embedment lengths greater than 1½ in. are indicated by dark symbols. The data points for beams with rib heights of 3 in. and embedment lengths greater than 1½ in. fall at the top of the cluster. Although not considered as a variable in the Lehigh test program, it is obvious from Fig. 13 that embedment length is a key parameter in connector capacity. This observation has also been made by Robinson.[12]

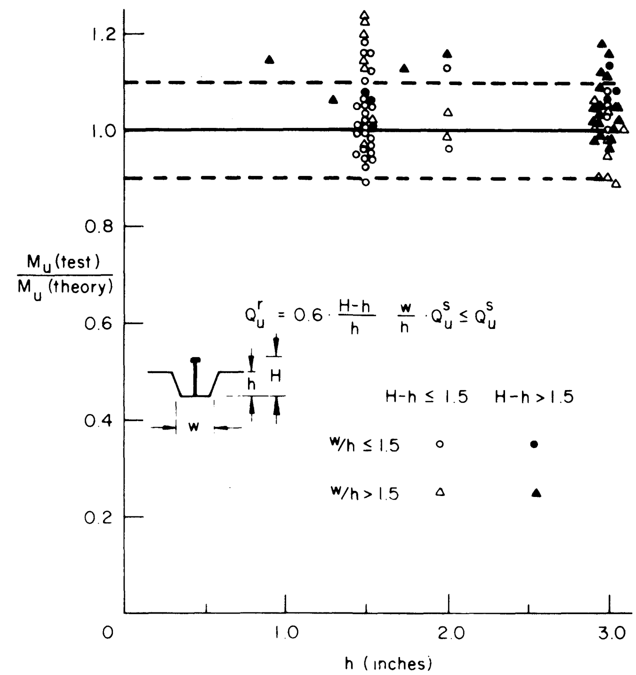

Fig. 15. Moment ratio vs. rib height for Eq. (5)

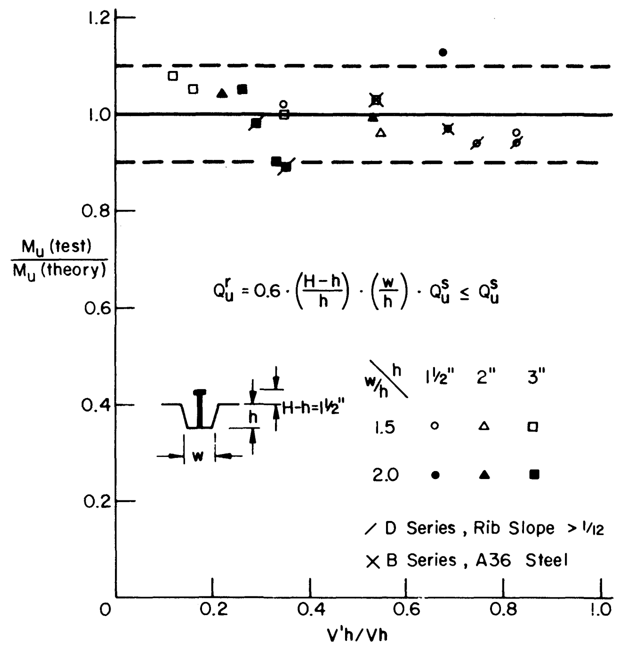

Fig. 16. Moment ratio vs. degree of partial shear connection for Eq. (5)

Thus, additional modifications to the connector capacity model proposed in Ref. 5 are required. Besides the average rib width-height ratio, the height of the rib and the embedment of the connector must be taken into account to correctly predict the flexural capacity of composite beams with formed steel deck. To reflect these additional governing parameters, the following revised model was initially developed ignoring the differences between one or two connectors in a rib:

$$Q_{rib} = A\left(\frac{H-h}{h}\right)\left(\frac{w}{h}\right) Q_{sol} \le Q_{sol} \qquad (5)$$

where

Q_{rib} = strength of stud shear connector in a rib
w = average rib width
h = height of rib
H = height of stud shear connector
Q_{sol} = strength of a stud shear connector in a solid slab
A = 0.6 for one or two connectors in a rib

Figure 15 shows all 75 beam test results in terms of test moment nondimensionalized by theoretical moment as a function of rib height. Equation (5) was used in predicting beam capacity. Figure 16 shows the same moment ratio as a function of the degree of partial shear connection, $V'h/Vh$, but for the 17 Lehigh tests only. The plots indicate that the connector capacity defined by Eq. (5) provides a better estimate of flexural capacity for beams with 3-in. deck. About the same flexural capacity is provided for beams with 1½- and 2-in. deck. Equation (5) continues to account for the varying width-height ratios as indicated by the relatively even dispersion of the test beams for all rib heights.

The scatter of the results can be primarily attributed to two factors. The first is the methods of acquiring the data. Many of the earlier tests were done on a proprietary basis, and some of the governing parameters were not similarly defined or were not reported at all. This variation in data is indicated in Table 2(B). A typical example of such variation is the reported yield point of the steel. Values were given in the test reports for static yield, dynamic yield, yield at 0.2% offset, or not at all and, consequently, had to be estimated. Typical values could easily differ by as much as 10%. This corresponds to an approximately equal change in the theoretical flexural capacity of a composite beam. Consequently, a proportional amount of scatter among the ratios of test moment to theoretical moment would be expected. A related example for the Lehigh tests is beam 1A5R. Its excessive capacity was primarily due to the strain hardening of the steel at the onset of yielding.

The second factor is the difference between the assumed and the actual location of the slab force at ultimate. As was shown earlier, flexural capacity is very sensitive to the slab force location. For the beam analyzed in Ref. 9, for example, a difference of 12% in the moment arm of the slab force results in a 7% difference in the predicted ultimate moment. Various models for slab force locations were studied, including the model for solid slab beams presented

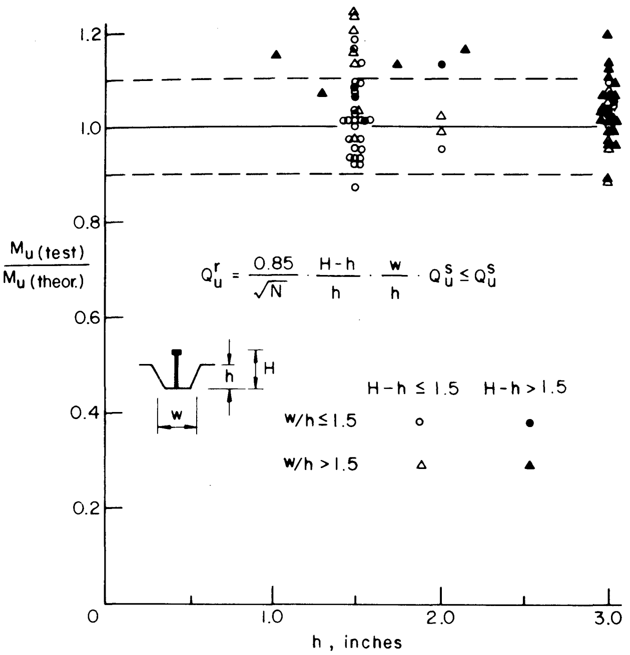

Fig. 17. Moment ratio vs. rib height for Eq. (6)

in Ref. 6. Assuming the slab force to act at the center of the solid portion of the slab above the rib was found to have the least scatter of all the models.

To a lesser extent, variations in the connection force have an effect on the predicted flexural capacity. However, for composite beams with a partial shear connection greater than 40%, it can be shown that the ratio of percentage change in connector force to flexural capacity is approximately 4 to 1. Thus, while there is always the possibility that additional unidentified variables exist which are not accounted for, the effect of such error is believed small.

Several studies[5,12,14] on push-out and push-off specimens have indicated that increasing the number of connectors per rib does not increase the shear capacity of the rib by the same proportion. Yet because of the limited number of ribs available, more than one stud per rib is usually required. The Lehigh test program limited the number of studs per rib to two connectors. This resulted in combinations of doubles and singles which were common for the additional 58 beam tests considered. The effect of these various combinations of doubles and singles may also contribute to the scatter of the test results. Measurements on the Lehigh test beams gave no indication of a consistent difference between one or two connectors per rib.

After additional tests were available, all test data was reevaluated, considering the variation in connector capacity to be affected by the number of connectors in a rib as suggested by Lim.[37] This resulted in the following relationship:

$$Q_{rib} = \frac{0.85}{\sqrt{N}} \left(\frac{H-h}{h}\right) \left(\frac{w}{h}\right) Q_{sol} \le Q_{sol} \qquad (6)$$

where N is the number of studs in a rib and the other parameters remain as defined previously in Eq. (5). Figure 17 shows the test data replotted, and indicates about the same degree of correlation as provided by Eq. (5) (see Fig. 15). Many beams with single connectors in a rib were not significantly affected, as they had more than enough connectors to ensure full composite action. Also, there is no difference between Eqs. (5) and (6) for two studs in a rib, as $0.85/\sqrt{2} \simeq 0.6$. Only one test is available with three studs in a rib; however, Eq. (6) gave a better prediction of flexural capacity than Eq. (5). A second test with a combination of two and three connectors in a rib was also in agreement with Eq. (6). Push-off tests reported in Ref. 14 with as many as five studs per rib depict a variation in connector capacity due to the number of studs in a rib similar to that indicated by Eq. (6). Thus it appears that Eq. (6) provides a reasonable prediction of connector capacity for varying numbers of shear connectors in a rib.

A variable whose effect may be questioned is the embedment length of the connector above the rib. This variable, as mentioned previously, was not considered in the Lehigh test program. Beams with an embedment length greater than 1½ in. generally have higher test moments than predicted. Thus, Eq. (5) or (6) provides a conservative estimate of capacity. Additional tests carried out at the University of Texas and at Lehigh have verified the effect of embedment length above the rib.[36,38]

Despite the scatter in Figs. 12 through 17 due to material properties and the indeterminate location of the stress resultant in the slab, Eqs. (5) or (6) enable one to reasonably predict the flexural capacity of a beam with formed steel deck. The key variables are the rib geometry and stud embedment length above the rib. No other variables were found to affect the scatter band of the test data uniformly, including three of the additional variables considered in the Lehigh test program: yield strength of steel, degree of partial shear connection, and rib slope. The mean value of the ratio of test moment to predicted moment in Fig. 15 was 1.024, with a standard deviation of 0.082. Thus only a few beams exhibited less capacity and a substantial number provided more capacity than predicted. For Fig. 17, these values were 1.015 and 0.081, respectively.

A concern is the fact that Eqs. (5) and (6) imply a reduced capacity of systems constructed with 3-in. decks with connectors proportioned by Eq. (1). Although a reduced shear connector capacity results with 3-in. ribs and 1½-in. embedment, no adverse behavior or failures have resulted. Such systems have given satisfactory performance to date because there is structural benefit gained by a composite beam in a building due to the diaphragm action of the slab-floor system and other inherent redundancies. In addition, most composite beams were designed at or near full composite action. Having a reduced shear connection capacity results in a beam with partial shear connection. It has been well established from this study and others[6,15] that the reduction in flexural capacity is small compared to the corresponding reduction in the degree of partial shear connection for beams at or near full composite action.

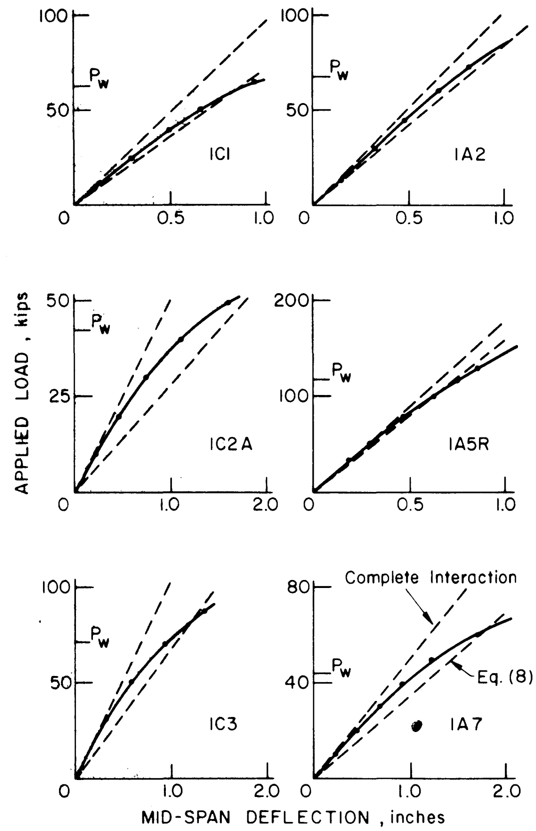

Fig. 18. Load-deflection curves, working load range

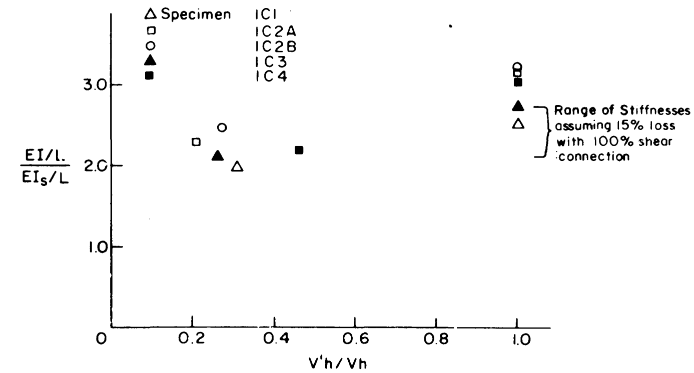

Fig. 19. Stiffness vs. degree of partial shear connection

Therefore, the reduced shear connection capacity provided by Eq. (5) or (6) would in most cases only provide a slight reduction in flexural capacity of these composite beams.

However, these reasons do not justify the continued use of Eq. (1). The capacity gained from an entire slab-floor system in a structure is not well defined. Also, with lower partial shear connection the decrease in flexural capacity becomes more pronounced; yet it is more efficient, and sometimes necessary, to design for partial shear connection. Thus, the best estimate of shear connection capacity should be utilized in design.

Stiffness—The load-deflection plots shown in Fig. 5 are shown again in Fig. 18 for the working load range. These plots show that with partial shear connection beams are less stiff than assumed for full composite action. This is due to the loss of interaction accompanying partial shear connection. For the Lehigh test beams with the least amount of shear connection, the stiffness was found to be between 70% and 80% of that calculated for full composite action at the working load level.

Early studies at the University of Illinois[8] and more recent studies at the University of Missouri[15] have shown that composite beams with flat soffit slabs designed for full composite action have 85% to 90% of their calculated stiffness at the working load level. This loss in stiffness can be attributed to the fact that the shear connectors are flexible. Thus, the connectors permit some slip or loss of interaction between the slab and the steel beam of a composite member, even though they will take all the force required for full composite action.

The shear connectors in a composite beam with formed metal deck behave similarly. Thus, one would expect the same sort of difference to exist between actual and assumed stiffnesses of such beams designed for full composite action. The Lehigh test beams with the lowest degree of partial shear connection exhibited 20% to 30% loss of the stiffness, which is about twice as much as experienced for full composite action in flat soffit slabs.[8,15] The stiffness of five Lehigh test beams with low degrees of partial shear connection is demonstrated graphically in Fig. 19. The stiffness is defined as that required to produce the measured deflection (minus the contribution due to shear) at the working load. This stiffness is nondimensionalized by the stiffness of the steel beam alone and plotted as a function of the degree of partial shear connection, using the connector capacity defined by Eq. (5). The calculated stiffness of these same beams for full composite action is also shown in Fig. 19. In addition, the range of stiffness is shown which corresponds to a 15% loss in stiffness. The plot clearly shows that composite beams with formed steel deck provide at least twice the stiffness of a non-composite system, even for extremely low degrees of partial shear connection. Thus, a low degree of partial shear connection is very efficient in terms of stiffness.

Because of the complexity of the nonlinear variation of stiffness with the degree of partial shear connection, several empirical relationships have been examined.[13,16] A relationship of the form

$$I_{eff} = I_s + \left(\frac{V'h}{Vh}\right)^{\alpha} (I_{tr} - I_s) \tag{7}$$

where

I_{eff} = effective moment of inertia
I_s = moment of inertia of the steel section
I_{tr} = moment of inertia of the transformed composite section
α = numerical exponent

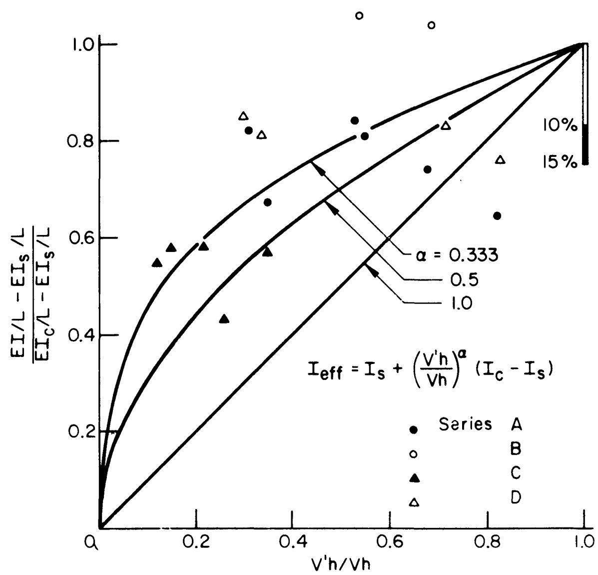

Fig. 20. Variation of stiffness with degree of partial shear connection

was found to provide a reasonable fit to test data when α was taken equal to ½ or ⅓, as is demonstrated in Fig. 20. The stiffness provided by the 17 test beams is plotted for comparative purposes. With no shear connection, the stiffness is essentially that of the steel beam alone. A composite beam with full composite action (as provided by 100% shear connection) will be assumed to have the stiffness of a transformed section with no loss of interaction between slab and beam. The straight line running from 0 to 1 in Fig. 20 would represent a linear variation of stiffness with α equal to 1. The curved lines provide an exponential variation. The solid vertical line at $V'h/Vh$ equal to 1.0 shows the possible 15% variation between actual and assumed stiffness for a fully composite member. The plot clearly shows that the variation provided by the exponent α equal to ½ is generally conservative, yet representative. The maximum deviation occurs as the degree of shear connection approaches unity. In no case is the loss of interaction greater than expected for a full shear connection.

The scatter in the test data can be attributed to several factors. First, one needs to recall the theoretical relationship between stiffness and the degree of partial shear connection. Stiffness was defined in terms of beam deflection. Deflection is found from double integration of the curvature which is a function of the applied moment, the force in the slab, and the combined stiffnesses of the slab and the steel beam. Thus the deflection, for a given applied moment, is a function of the slab force between zero and full shear connection. A moment of inertia for a composite beam between the limits of zero and full shear connection cannot be calculated from the cross section dimensions alone. However, a pseudo moment of inertia for estimating beam deflections may be defined. The approximate nature of an empirical expression for an effective moment of inertia is the primary reason for the scatter exhibited in Fig. 20. Secondly, Fig. 20 shows the difference in stiffness between that of the steel beam alone and that of the transformed composite section. Consequently there is a magnification of the deviation. Finally, the stiffness was determined from the measured deflection at working load, which is a direct function of the section modulus. In Fig. 20 the effective section modulus was defined by the relationship provided in the AISC Specification,[10] which is an approximation.

Studies at the University of Missouri showed comparable behavior for composite beams with flat soffit slabs.[15] A comparison of these data indicated general agreement with Eq. (7) when α was taken as ½.

Until recently the AISC Specification neglected any loss of stiffness, because earlier studies showed that the loss was not excessive if the degree of partial shear connection was more than 50%. However, as a result of the studies reported herein, Supplement No. 3[10] permitted a decrease in the degree of partial shear connection to 25%, provided an adjustment was made in the beam stiffness and a linear relationship for section modulus was used.

The effective moment of inertia was defined as

$$I_{eff} = I_s + \sqrt{\frac{V'h}{Vh}}\,(I_{tr} - I_s) \tag{8}$$

This relationship is equally applicable to beams with flat soffit slabs and beams with slabs cast on formed steel deck. Several load-deflection curves are plotted in Fig. 18, showing the effect of this relationship.

Stresses and Margin of Safety—The 1969 AISC Specification[7] provides a formula for an effective section modulus, S_{eff}, to determine stresses in the bottom flange of composite beams with partial shear connection. The formula provides a linear interpolation with respect to the degree of partial shear connection between the section modulus of the steel beam alone, S_s, and the section modulus of the transformed section, S_{tr}.

Slutter and Driscoll[6] developed the theoretical basis for the ultimate capacity of beams with partial shear connection after observing the results of tests to destruction of composite beams. A relatively linear decrease in ultimate moment, based on full composite action to 50% partial shear connection, was observed. An empirical approximation for flexural capacity was recently suggested by Johnson[35] for both rigid and flexible shear connectors. The flexural capacity for partial shear connection was related to the ultimate capacity of the steel beam alone and the capacity for full composite action. Both of these studies demonstrated that a linear variation was very conservative. Studies at the University of Missouri have shown that the effective section modulus, S_{eff}, in the AISC Specification[7] is extremely conservative for low degrees of partial shear connection.[15]

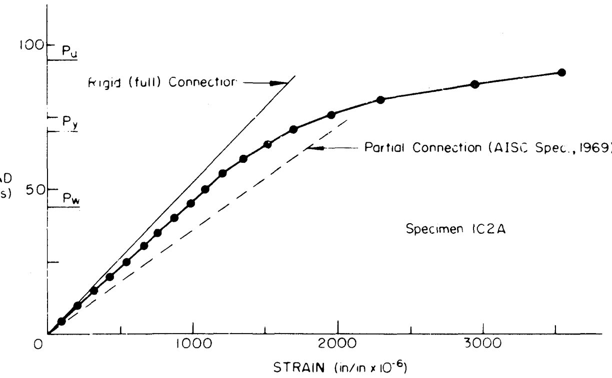

Fig. 21. Load vs. midspan, bottom flange strain

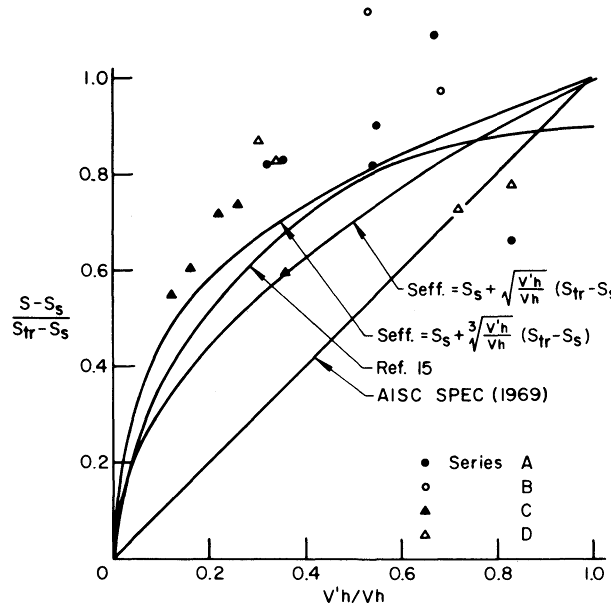

Fig. 22. Variation of section modulus with degree of partial shear connection

Measurements on the 17 Lehigh test beams show that the AISC formula for S_{eff} is also conservative in the ranges of low partial shear connection when applied to beams with formed steel deck. Figure 21 demonstrates this point with a plot of load versus midspan bottom flange strain for one of the test beams. The slope of any curve drawn on these axes is a function of the modulus of elasticity, the section modulus, and the span length. The nonlinearity of the test curve near the yield load reflects the nonlinearity of the stress-strain curve near yield and the nonlinear behavior of the shear connectors. The data points from test measurements fall just below the solid line which defines the theoretical slope for a beam designed fully composite. The dashed line reflects the slope determined by the linear S_{eff} of Ref. 7.

Test measurements were taken at several locations along the span and indicated that the section modulus varied along the span by approximately 5%. This behavior is not due solely to experimental inaccuracies. The reason for this behavior is that the slab force, F, does not vary uniformly with the resisting moment, M, along the span of the beam. Rather, the force-moment relationship is defined by the governing differential equation for the composite beam and, consequently, is dependent upon the loading and boundary conditions.[8,9,17]

Fortunately, a variation of F does not affect the section modulus significantly. The section modulus only varies between S_s and S_{tr}. Thus, for composite beams commonly used in building construction, with an approximate S_{tr}/S_s ratio of 1.5, F only affects about ⅓ of the section modulus. Hence a 30% variation in F would affect the section modulus by less than 10%. As F becomes smaller and incomplete interaction develops, the effect of F on the section modulus is decreased, because the difference between the section modulus and S_s is less. Similarly, the effect of F on the section modulus for composite beams with large steel beams and small slabs is less, because the difference between S_{tr} and S_s is less, relative to S_s. A variation of F of 30% along the span would represent an extreme loading condition. Typical plots of F vs. span are presented in Ref. 9 for three loading conditions.

To determine a more accurate relationship for an effective section modulus in terms of the degree of shear connection, the test data were evaluated by the same procedure outlined for stiffness. In Fig. 22 the difference between the effective section modulus determined from test measurements and the section modulus of the steel section alone is plotted as a function of the shear connection for the 17 Lehigh test beams. Superimposed on the plot is a line representing the 1969 AISC formula for S_{eff}. As the plot shows, it is extremely conservative for low degrees of partial shear connection. Also plotted in Fig. 22 are two parabolas with respect to the degree of partial shear connection with exponent $\alpha = \frac{1}{2}$ and $\frac{1}{3}$. These parabolas yield the following formula for effective section modulus:

$$S_{eff} = S_s + \left(\frac{V'h}{Vh}\right)^{\alpha} (S_{tr} - S_s) \qquad (9)$$

The higher order parabolas are more representative of the Lehigh test data than the AISC design relationship,[7] and are conservative as well. The curves exhibit about the same degree of unconservatism as the AISC relationship as full composite action is approached. The result of the study at the University of Missouri[15] on beams with flat soffit slabs is also shown in Fig. 22 and is seen to fall between the two parabolic relationships. The comparison shows that Eq. (9) with $\alpha = \frac{1}{2}$ or $\frac{1}{3}$ is reasonable for conventional composite beams as well. The scatter in Fig. 22, as in Fig. 20 for stiffness, is caused by the approximate nature of the empirical expression for S_{eff} and the magnification provided by the plot.

Using Eq. (9), a comparison of the working load moment with flexural capacity may be made. This reflects the margin of safety for composite beams.

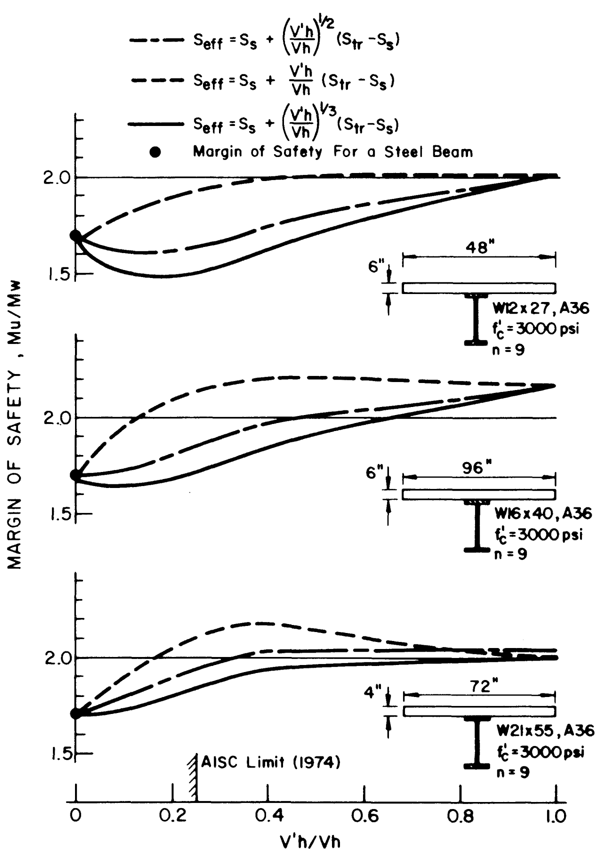

Fig. 23. Margin of safety vs. degree of partial shear connection

For the Lehigh test results, Eq. (9) with $\alpha = 1/3$ provides a factor of safety between 2.70 and 1.99, if the working load is based on the minimum specified yield used in design practice. If the actual yield stress* were used, the values would decrease to 2.04 and 1.52, respectively. However, it should be recognized that this represents a lower bound condition and has little likelihood of frequent occurrence.

The margin of safety was also examined theoretically for three cross sections which were selected from the 75 beam tests. These 75 beam tests represent the range of slab-beam combinations used in construction practice. This range extends from a shallow, light steel beam with a thick slab to a deep beam with a small slab, such as a wind girder in a building. The beam cross sections are shown in Fig. 23, along with plots of the ratio of ultimate moment to working load moment versus the degree of shear connection, $V'h/Vh$. Ratios are shown for three working load moments: one defined by the 1969 AISC Specification[7] and the other two defined by Eq. (9) with $\alpha = 1/2$ and $1/3$. The model used to predict flexural capacity is that outlined in Ref. 6.

* *See Table 2(A).*

The presence of steel deck in these beams has little effect on the moment ratios. Without steel deck, the location of the slab force would shift, theoretically, and the flexural capacity would increase slightly. Other variables such as steel strength, concrete strength, and concrete weight have negligible, if any, effect on this theoretical moment ratio. For high strength steel, both the flexural capacity and the working load moment are increased proportionally and do not change the moment ratio. The strength of the concrete slab may limit the flexural capacity rather than the high strength steel in some cases. When this occurs, the concrete stresses generally limit the working load moment, resulting in little, if any, net change in the moment ratio. Increasing the concrete strength would tend to increase the flexural capacity slightly; however, the working load moment, limited by the steel stress, would tend to increase the moment ratio. If lightweight concrete were used, the flexural capacity would remain unchanged, but the effective section modulus would be decreased slightly, resulting in a decreased working load moment and, thus, a slightly increased moment ratio.

The plots in Fig. 23 reveal several aspects of the relationship between the working load moment and the flexural capacity of composite beams. First, neither the 1969 AISC formula for S_{eff} nor Eq. (9) provides a consistent relationship for all three beams with a similar degree of shear connection. A working load design procedure based on a consistent relationship would be extremely difficult to develop, because it is dependent upon the geometrical and material properties of the slab and the steel beam. A consistent margin of safety could be established for composite beams if it were dependent on the flexural capacity alone and not on allowable stresses.

It is also apparent that the relationship between the working load moment and the flexural capacity is least conservative for the most critical cases when $\alpha = 1/3$. For example, Case 1 is a composite beam with a thick slab and a light shallow steel section. It is heavily dependent upon composite action for its flexural capacity. Case 3, on the other hand, is a beam with a small slab and a large steel section, and its flexural capacity is primarily dependent upon its large steel section. Yet, Fig. 23 shows Case 3 to have a higher margin of safety than Case 1, whether the 1969 AISC formula or Eq. (9) is used. Similarly, as the degree of shear connection is reduced in a composite beam, the flexural capacity of the beam becomes less dependent upon composite action and more reliant upon the steel section. Equation (9) with $\alpha = 1/2$ produces a margin of safety which decreases to that of the steel section alone, over the range of shear connection permitted by the 1973 AISC Specification Supplement No. 3[10] ($V'h/Vh \geq 0.25$). The AISC formula maintains an essentially constant or increasing margin of safety. When $\alpha = 1/3$, the margin of safety for Case 1 is less than that of the steel section alone near the lower limit of partial shear connection.

Initially composite construction was based entirely on elastic behavior by imposing deformation limitations on the connectors. However, it was found that this approach was unduly conservative with respect to the flexural capacity and performance of the composite beam. The 1969 AISC Specification relies on the ultimate capacity of the connectors and cross section, even though an allowable stress design procedure is used. The ultimate strength design of composite beams will provide the most consistent design approach. Continued application of allowable stress design will obviously lead to greater variation, as the three examples summarized in Fig. 23 have demonstrated.

Other factors should be kept in mind when considering the true margin of safety. For example, the working load moment is based on the minimum specified yield of steel. Yet, manufacturing practice insures that the average yield stress is well above the specified minimum. Thus the margin of safety is increased automatically. This consideration is independent of other inherent and unaccounted for increases brought on by incorporation of a composite beam into a structural system. Another factor is the degree of shear connection commonly used in construction practice. Although it is more efficient to design for low shear connection, this is seldom done in practice. Low degrees of shear connection usually occur out of necessity in beams with short spans and/or incorporating formed metal deck, for example. Here space limitations simply dictate the degree of shear connection. The actual loads on short spans produce a proportionally greater amount of friction between the slab and the beam, thus slightly increasing the apparent shear connection and the beam capacity. Also, for short spans the effective slab width is dictated by the span and thus results in smaller cross sections, decreased working load moments, and larger moment ratios.

Equation (9) defines a design moment based on allowable stress in the steel. It defines a decreasing margin of safety between design moment and flexural capacity for decreasing degrees of partial shear connection. It would not significantly lower this margin below that allowed for the steel beam alone with $\alpha = 1/2$. With $\alpha = 1/3$, the margin could be less than for the steel beam alone only in extreme cases. (A minimum shear connection of 40% would prevent this from occurring with $\alpha = 1/3$.) On the other hand, the formula for S_{eff} in the 1969 AISC Specification is overly conservative in determining a design moment based on allowable stress in the steel beam; it provides a relatively constant or increasing margin of safety with decreasing degrees of partial shear connection. Short of an ultimate strength design approach, Eq. (9) provides a reasonable design moment using either $\alpha = 1/2$ or $\alpha = 1/3$.

SUMMARY AND CONCLUSIONS

This report presents the results of 17 composite beam tests conducted at Lehigh University incorporating formed steel deck. These results were analyzed in conjunction with 58 additional tests conducted by other investigators. The purpose of this report was to evaluate shear connector capacity and beam flexural capacity and behavior and to compare such capacity and behavior with existing design criteria.

The following conclusions may be drawn from the analysis reported herein:

1. The capacity of stud shear connectors in the ribs of composite beams with formed steel deck may be determined from the following empirical expression:

$$Q_{rib} = \frac{0.85}{\sqrt{N}}\left(\frac{H-h}{h}\right)\left(\frac{w}{h}\right) Q_{sol} \leq Q_{sol}$$

   where N is the number of studs in a rib, H is the height of a stud shear connector in the rib, h is the height of the rib, w is the average rib width, and Q_{sol} is the strength of the stud shear connector in a flat soffit slab. Very little test data is available for beams with more than two studs in a rib. However, the relationship appears reasonable for multiple studs.

2. The flexural capacity of a composite beam with formed steel deck can be more accurately and conservatively estimated if the slab force is considered to act at the mid-depth of the solid portion of the slab above the ribs, rather than at the centroid of the concrete stress block as proposed in Ref. 6 for solid slab composite beams.

3. The deflection of a composite beam with partial shear connection, with or without formed steel deck, may be estimated with the expression for an effective moment of inertia contained in Ref. 10:

$$I_{eff} = I_s + \sqrt{\frac{V'h}{Vh}}\,(I_{tr} - I_s)$$

   where I_s is the moment of inertia of the steel beam, I_{tr} is the moment of inertia of the transformed composite section, $V'h$ is one-half the total horizontal shear to be resisted by connectors providing partial composite action, and Vh is one-half the total horizontal shear to be resisted by connectors under full composite action for allowable stress design.

4. The stress in the bottom steel fiber of a composite beam with partial shear connection, with or without formed steel deck, may be estimated using an effective section modulus:

$$S_{eff} = S_s + \left(\frac{V'h}{Vh}\right)^{\alpha} (S_{tr} - S_s)$$

   with $\alpha = 1/2$ or $1/3$ and where S_s is the section modulus of the steel beam used in composite design referred to the bottom flange, S_{tr} is the section modulus of the transformed composite section, referred to the bottom flange, and $V'h$ and Vh are the horizontal shears mentioned previously. If $\alpha = 1/3$ is used, the minimum shear connection should be about 40%.

ACKNOWLEDGMENTS

The testing described in this report was conducted at Fritz Engineering Laboratory, Lehigh University. The tests were carried out with the assistance of K. Harpel and his staff. R. Blough and K. Wickham assisted in the data reduction. The report was prepared by the secretarial and drafting personnel at Fritz Laboratory. The contribution of these people is gratefully acknowledged. Also appreciated were the helpful comments and criticisms of the members of the Sponsors Group for AISI Project 178 and the members of the AISC Specification Advisory Committee, Task Group No. 4.

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