Discussion

Design of Composite Beams with Formed Metal Deck

Paper presented by JOHN W. FISHER (July, 1970 issue)

Discussion by **George Winter**

With the widely established use of concrete slabs in composite action with metal deck cold-formed from sheet steel, the utilization of the additional composite action of such slabs with the supporting steel beams becomes increasingly attractive and important. Professor Fisher's contribution toward clarifying the composite action of steel beams with such slabs is, therefore, timely and welcome. It is realized that a brief presentation in the AISC ENGINEERING JOURNAL does not constitute complete research documentation. It is also realized that the author has drawn on numerous in-house reports not readily accessible to the general reader. With all this in mind, clarification of the following questions and comments would enhance the utility of this investigation:

1. Equation (1) relates the ultimate connector strength in a ribbed composite slab to that in a solid composite slab, the multiplier being $0.36(w/h)$, where w = average rib width and h = rib height. One should expect, then, that since $Q_{u-rib}/Q_{u-sol} = 0.36(w/h)$, the allowable loads on connectors in the two types of slabs should be held to the same ratio as the ultimate strengths. Instead, in Eq. (2), for normal weight concrete, the author proposes a ratio of $0.50(w/h)$ for allowable connector loads. This means that for connector strength, while AISC uses "a safety factor of approximately 2.50" (see AISC Commentary on the Specification for the Design, etc., Feb. 12, 1969, p. 5-150), the author proposes in effect a safety factor of approximately 2.5 (0.36/0.50) = 1.80. In fact, the author himself states that his Eq. (2) is based on a safety factor of 1.67. It is not clear why the safety factor on connector strength in ribbed slabs should be as low as 1.67, when for the same situation in a solid slab the nationally recognized AISC Specification maintains a factor of approximately 2.5. The author states that his value of 1.67 is the same safety factor as for bending stress in steel structures design. It is universally accepted that safety factors on connectors of all kinds are generally larger than the basic safety factors on the base materials properties; the AISC factor of 2.5 is quite in line with this procedure, but the author's is not.

2. The author states that the strength of such composite slabs is adversely affected by increasing rib height when ribs are deeper than 1½ in. Figure 5 is supposed to document this statement. However, of all the beams represented on this graph, only two pairs of lightweight concrete slabs exhibit this tendency. These are the pair with h = 1.5 in. and w/h = 1.5, compared with the pair with h = 3.0 in. and w/h = 1.4. These two pairs, which seem otherwise identical except for rib height (disregarding the somewhat smaller w/h-ratio of the second pair) do indeed show a decrease of strength with increasing rib height. It would be interesting to know whether the lightweight concretes for these two pairs were, in fact, identical. However, this evidence is offset in the same figure by the tests on normal weight concrete with w/h = 1.0 and h = 1.5 in. on the one hand and 2.25 in. on the other. For this set of otherwise presumably identical specimens, increasing rib height resulted in a slight increase, rather than the stated decrease in strength. All other points on Fig. 5 have such widely varying w/h-ratios that no conclusions can be drawn as to the effect of rib height. Hence, the author's statement that the absolute value of rib height affects slab strength does not seem to be supported by the presented evidence.

3. From Fig. 7 it appears that for these composite slabs the strength-reducing effect of lightweight concrete is more pronounced than for solid slabs. This is evident in this nondimensional plot from the fact that all points for lightweight concrete fall significantly below those for normal stone concrete. It appears that this strength reduction for ribbed slabs is not sufficiently represented by the ratio $\sqrt{E_{c-l}/E_{c-n}}$ because this ratio presumably applies to solid slabs, while Figure 7 seems to show that the effect of lightweight concrete for ribbed slabs is more pronounced than for solid slabs. To put it differently, the proposed straight line with slope $0.36(w/h)$ appears to be too conservative for normal-weight concretes, and excessively unconservative for lightweight concretes, as

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clearly reflected in Fig. 7, particularly when used with a safety factor as low as 1.67.

4. It would seem that the author is using the depth of the equivalent rectangular stress block, a, as defined in the ACI Building Code, for the actual depth to the neutral axis, c. Actually, it is the latter depth which should be used, which is $c = a/0.85$ for concretes up to $f_c = 4000$ psi.

5. In his Tentative Design Recommendations, the author distinguishes three cases, but does not indicate how to treat Case 2, unless the implication, not clearly stated, is that it should be treated the same as Case 3.

To repeat, it cannot be expected that in an abbreviated report of this type all problems are treated exhaustively and unambiguously. The only purpose of this discussion is to suggest that the above questions, and possibly some others, should be clarified and their clarification documented, before the proposed design procedure is put into practical use.

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Professor Winter has raised a number of important points regarding the behavior of composite beams with formed metal deck. Several cannot be answered directly because the existing data involves so many different variables that it is not possible to isolate an effect directly.

As noted in the paper, most of the experimental work was not coordinated. Often more than one variable was changed. This makes direct comparisons difficult, if not impossible, when assessing observed differences in beam capacity or component behavior. Because of these factors, much judgment was used when developing the tentative design recommendation. However, the design procedure provided a factor of safety of at least two against the flexural capacity. This is illustrated in Table 1, where results from available beam tests are compared with the design moment computed by the recommended procedure. The ratio of maximum test moment to working load moment is seen to vary from 2.05 to 2.67. The results are for both normal weight and lightweight concrete composite beams with substantial variations in rib geometry (see Table 1). The margin of safety between working load and the flexural capacity does not differ systematically with either the type of concrete or rib geometry now in common use.

The items raised by Prof. Winter point out the need for additional research. A systematic evaluation should be made which utilizes a well planned experiment design. Plans are being made to undertake such studies in the near future. The paper provides an interim procedure pending completion of such work. It is believed to provide a satisfactory margin of safety for the composite beam with formed metal decks now in common use.

The following comments are intended to provide clarification on the specific points raised.

Items 1, 3. It would have been better to express Eq. (1) as

$$Q_{u-rib} = A \frac{w}{h} Q_{u-sol} \tag{1}$$

Equation (1) was only intended to represent a model that established the major factors which influenced the shear connection strength and behavior. Since push-off specimens were used for this purpose, the numerical value of coefficient A is not applicable to beams.

The differences between the lightweight concrete and normal-weight concrete push-off test data exists because the two test series are not directly comparable. As noted in the paper, the test data for the normal-weight concrete was from push-off tests on specimens with two slabs and with small $\frac{3}{8}$-in. connectors. Data for lightweight concrete was from single slab push-off specimens with $\frac{3}{4}$-in. connectors. The apparent difference in Fig. 7 is caused by the type of specimen, connector size, and type of concrete. Substantial differences in strength and behavior have also been observed between these two types of specimens in work on solid slabs.[18] It was found that single slab push-off specimens tended to give up to 30% less strength than two slab push-off tests.

Push-off tests do not completely simulate the beam condition. The divergence is substantial for composite beams with formed metal deck. Beam tests have always indicated an improvement in shear connector behavior and strength. Therefore, Eq. (2) was developed from beam tests with formed metal deck, using Eq. (1) as a model to account for changes in rib geometry. The factor of safety between the apparent connector strength and the suggested design value given by Eq. (2) is between 1.9 and 2.3 for available tests. The factor differs from the ratio of flexural capacity to working load moment because the relationship between the two is not direct.

This margin was considered satisfactory because a recent study[16,19] has determined the strength of stud shear connectors in lightweight and normal-weight concrete solid slabs as

$$Q_{u-sol} = \frac{1}{2} A_s \sqrt{f_c' E_c} \tag{9}$$

where f_c' and E_c are in ksi and A_s is the nominal stud area. The current AISC design values for normal-weight concrete were found to be about half the connector

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Table 1. Comparison of Proposed Working Load with Flexural Capacity of Available Beam Tests

	Beam	Rib Geometry w (in.)	h (in.)	w/h	f_c' (ksi)	Stud Diam (in.)	Studs per Rib	[a] V_h'/V_h	[b] M_w (in.-k)	M_u (in.-k)	M_u/M_w	Ref.
Lightweight Concrete	B1	4.06	3.00	1.35	4.4	3/4	2	0.38	1002	2233	2.23	8
	B2	4.06	3.00	1.35	4.9	3/4	2	0.38	1002	2183	2.18	8
	1	2.25	1.50	1.50	4.3	3/4	1	0.71	1332	2908	2.18	22
	2	2.25	1.50	1.50	3.4	3/4	1	0.81	1274	3404	2.67	4
	3	2.25	1.50	1.50	4.6	3/4	1	0.60	1284	3024	2.36	23
	4	2.25	1.50	1.50	5.1	3/4	1	0.57	2304	5294	2.30	24
	A	2.25	1.50	1.50	3.3	3/4	1&2	1.00	1411	3165	2.24	25
	D	2.25	1.50	1.50	3.3	1/2	1&2	0.54	1238	3033	2.45	25
	C	5.63	3.00	1.88	3.3	3/4	1&2	0.58	3288	6733	2.05	25
Normal-weight Concrete	B52	1.50	3.00	0.50	5.5	3/8	2	0.21	156	366	2.34	5
	HR	2.63	3.00	0.88	4.8	3/4	1	0.27	660	1520	2.30	14
	A31	1.50	1.50	1.00	7.4	3/8	2	0.66	91	209	2.30	5
	A32	1.50	1.50	1.00	7.9	1/2	2	0.60	87	204	2.35	5
	A51	1.50	1.50	1.00	7.7	3/8	2	0.59	187	391	2.10	5
	B54	2.25	2.25	1.00	5.9	3/8	2	0.42	182	460	2.52	5
	B53	2.25	1.50	1.50	5.8	3/8	2	0.66	197	465	2.36	5
	CU3	2.25	1.50	1.50	3.2	3/4	1&2	0.79	1130	2389	2.12	7
	C2	3.00	2.00	1.50	4.0	3/4	1	0.96	1210	2860	2.36	26
	BS12	2.25	1.31	1.72	4.0	3/4	1	1.00	1238	2651	2.14	6
	BS11	1.75	0.88	2.00	4.0	3/4	1	1.00	1238	2746	2.22	6
	CU2	3.63	1.50	2.42	4.2	3/4	1	1.00	1210	2807	2.32	7
	CU1	5.00	1.50	3.35	4.3	3/4	1	1.00	1360	3008	2.21	7

[a] Design Moment based on Proposed Criteria.
[b] V_h' = Total horizontal shear for the partial shear connection; V_h = horizontal shear for full shear connection [see Eq. (8)].

strength. The relationship suggested by Slutter and Driscoll[11] and used to develop the AISC values overestimated the connector strength. More recent beam and push-off tests with solid slabs have indicated that the safety factor of shear connectors varies between 2.0 and 2.5.[12,16,19,20,21] The major difference between the Slutter and Driscoll expression and Eq. (9) is the concrete modulus E_c. This was introduced to provide a further correction in addition to rib geometry. Table 1 illustrates that the design criteria that resulted provides about the same margin of safety for both lightweight and normal-weight concrete beams.

Item 2. The author agrees with Prof. Winter that Fig. 5 does not adequately document the fact that an increase in rib height adversely affects the strength. All of the beams cited had partial shear connection. With most commercially produced deck, an increase in rib height is accompanied by a decrease in the ratio w/h. As shown in Table 1, this results in a decrease in the ratio V_h'/V_h. Hence, the flexural capacity is reduced when compared to the full shear connection condition ($V_h'/V_h = 1.0$). In Fig. 5 the lighweight concrete beams with $h = 1.5$ in. (see Table 1, Beams 1, 2) had twice as much shear connection ($V_h'/V_h = 0.7$ - 0.8) as the lightweight concrete beams with $h = 3.0$ in. (see Table 1, Beams B1, B2 with $V_h'/V_h = 0.38$). The normal-weight concrete beams with $w/h = 1.0$ (see Table 1, Beams A31, A32, A51 and B54) had ratios of $V_h'/V_h = 0.42$ - 0.66. It is believed that the apparent differences are caused by the changes in beam geometry and the degree of partial shear connection.

Often it is not possible to increase the number of connectors because of insufficient space for installation. Since w/h generally decreases with an increase in h, a decrease in flexural capacity will result.

Item 4. Prof. Winter is correct in noting that use is made of the equivalent stress block with depth a. Because of the large steel area in composite steel-concrete beams, this has a negligible effect on the flexural capacity. The depth to the neutral axis is not used for any design computation. The equivalent stress block is used to provide an equilibrium check to determine if the shear connection will be governed by the area of steel beam or the area of slab and to provide a rough indication of whether or not the beam section properties are influenced by the ribs. Comparisons of the section properties of composite beams with formed metal deck with composite beams with equivalent solid slabs indicated that these properties were not significantly affected when Case 1 existed. The cross section properties were influenced for Cases 2 and 3 because the neutral axis of the elastic section is usually below the top of the ribs.

Item 5. The author's intent was to have Case 2 treated the same as Case 3. The word "and" should be inserted between the two captions.

18. *Ollgaard, J. G.* The Strength of Stud Shear Connectors in Normal and Lightweight Concrete, *M.S. Thesis, Lehigh University, Bethlehem, Pa., June 1970.*
19. *Ollgaard, J. G., Slutter, R. G., and Fisher, J. W.* Shear Strength of Stud Connectors in Lightweight and Normal-Weight Concrete, (*paper expected to be published in April, 1971 AISC Engineering Journal*).
20. *McGarraugh, J. B. and Baldwin, J. W.* Lightweight Concrete-on-Steel Composite Beams, *Dept. of Civil Engr., Univ. of Mo., Columbia, 1970.*
21. *Baldwin, J. W.* Composite Bridge Stringers, *Dept. of Civil Engr., Univ. of Mo., Columbia, Highway Research Report 69-4, 1969.*
22. *Rivera, U. and Slutter, R. G.* Tests of Lightweight Concrete Composite Beams with Metal Deck, *Fritz. Lab. Report 200.67.458.1, Jan. 1968.*
23. *Slutter, R. G.* Tests of Lightweight Concrete Members with Metal Decking, *Fritz. Lab. Report 200.68.458.1, March 1969.*
24. *Slutter, R. G.* Tests of a Lightweight Composite Beam with Metal Decking, *Fritz Lab. Report 200.69.458.2, August 1969.*
25. *Seek, W. G., Fisher, J. W., and Slutter, R. G.* Tests of Lightweight Concrete Composite Beams with Metal Decking, *Fritz Lab. Report 200.70.468.1, November 1960.*
26. *Hanson. R. E.* Full Scale Beam Test of Stud Shear Connectors and C2 Cofar, *Granco Steel Products Co. Research Report T-1151A, June 1970.*