Design of Composite Beams with Formed Metal Deck

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This paper was presented at the AISC National Engineering Conference, Pittsburgh, Pa., in May, 1970

FOR SOME TIME steel formed plate and concrete have been used compositely in building floor systems. Usually the formed metal deck is used as a permanent form to carry the fresh concrete and serve as a working platform. After hardening of the concrete, the metal deck and concrete slab act compositely to carry the applied live loads if the metal deck is provided with embossments to provide the shear connection.

A natural consequence was to develop composite action for the steel beams over which the formed metal deck was placed. This was first suggested by A. H. Atkinson of Hamilton, Ontario.

The AISC Specification for buildings does not provide for composite beams with a formed metal deck between the steel and concrete slab. When the metal deck corrugations are parallel to the beam, they do not interfere with the steel-concrete interaction and the condition is similar to a haunched slab for which the Specification provisions are applicable. However, when the metal deck is placed perpendicular to the steel beams and shear connectors are placed in the ribs of the corrugations, the behavior of the composite system may differ substantially from the expected behavior.

After many studies had been made for specific proprietary products or building applications,[1,2,3,4] a more detailed study was reported by Robinson,[5] who observed that the horizontal shear capacity was a function of the rib geometry. The shear capacity of the stud alone did not determine the overall shear behavior. In many instances the concrete in the deck flutes cracked and substantially reduced the shear transfer capacity. This study, as well as several others, indicated that small corrugations had little or no influence on beam behavior, and that the beams could be designed as though the slabs were solid.

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BEHAVIOR OF COMPOSITE BEAMS WITH METAL DECK

Beam Stiffness—For rib heights up to 1½ in., there is usually no significant reduction in beam stiffness in the working load range, provided the compressive stress block does not extend below the top of the rib corrugation. This has been verified by tests on a variety of metal deck profiles.[4,5,6,7] Figure 1 shows test results for Bethlehem Slabform and special Robertson Q-Lock deck, compared with the behavior of a conventional composite beam.[6] Beams with 4-in. thick solid slabs of normal weight concrete and flat-soffit deck having 0.875-in. and 1.312-in. high ribs were connected to a 12WF steel beam by ¾-in. studs. No tack welds were used between the metal deck and the rolled section of these beams. Within the working load range the metal deck did not significantly influence beam stiffness.

Figure 1 also shows test results for Beams B1 and B2 from Ref. 8 on beams with 3-in. rib heights. Except for the 4-in. ribs, the test beams were nearly identical in geometry to the beam tests reported in Ref. 6. The major differences were: (1) special Robertson Q-Lock deck with 3-in. high ribs, (2) a 5½-in. thick slab, (3) lightweight concrete, and (4) fewer shear connectors. The moment of inertia was 631 in.^{4} as compared to 617 in.^{4} for the solid slab, 615 in.^{4} for Slabform with h = 0.875 in., and 610 in.^{4} for Slabform with h = 1.312 in. No tack welds were used on Beam B1 and holes were cut in the metal deck to place it after studs were welded directly to the steel beams. The same procedure was used for the beams with Slabform. Beam B2 had tack welds placed in each valley of the floor units that contained stud shear connectors.

A comparison of beams with formed metal deck indicates that within the working load range, beam stiffness is not greatly affected by the height of the rib. The increased flexibility of the partial shear connection with 3-in. ribs became apparent as the working load

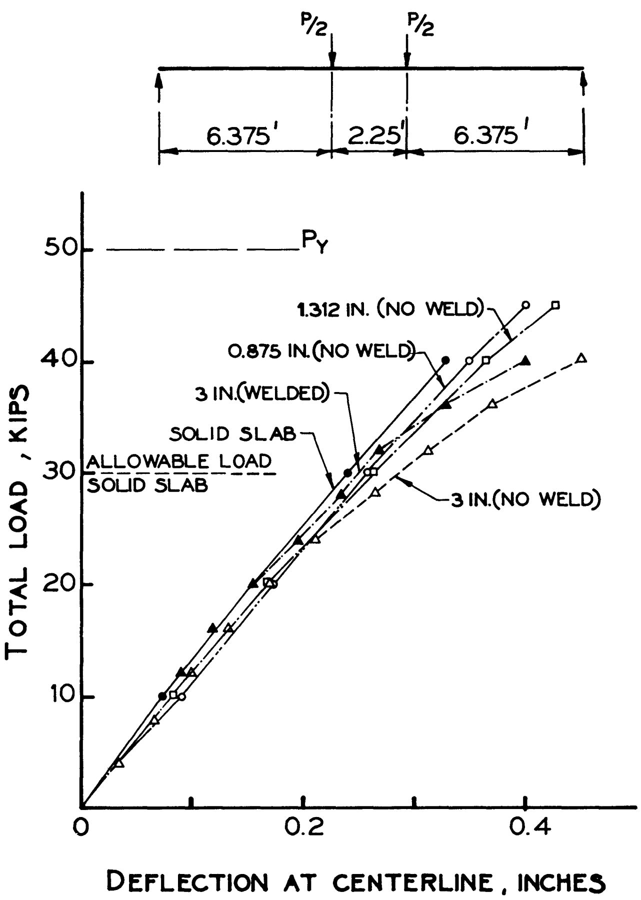

Fig. 1. Influence of metal deck on composite beam stiffness

level was approached. Similar behavior was reported by Robinson[5] for a wide variety of cell geometries. Figure 1 shows that tack welds provided a stiffer shear connection. Even with a 3-in. rib, Beam B2 exhibited a stiffness equal to the solid slab test beam at the lower load levels.

The comparative tests on beams with and without tack welds reported in Ref. 8 showed clearly that beam stiffness and strain agreed well with the predicted values when the metal deck was fastened to the steel beam. Figure 1 illustrates the effective beam stiffness and Fig. 2 provides a similar comparison for strain in the steel beam.

On the basis of the comparative behavior of the 3-in. rib deck, it is probable that formed metal deck with ribs will develop a beam stiffness equal to or greater than the conventional slab at or below the working load level when the shear connectors are welded directly through the deck or tack welds are used to connect the sheet to the beam. Since stud welding through the deck is now a routine procedure, it should not be necessary to consider incomplete interaction when metal deck is used, unless it is not fastened to the steel beam as described above.

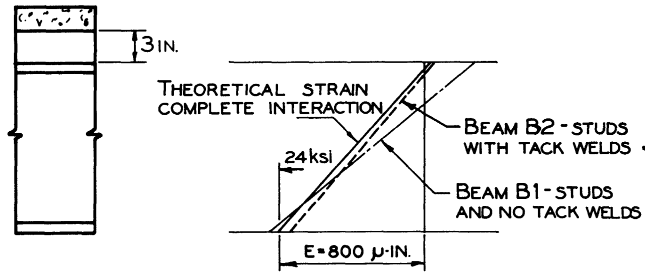

Fig. 2. Comparison of measured and computed strains in steel beam with 3-in. metal deck

Flexural Capacity—Complete load-deflection curves for a conventional composite beam and the three composite beams with metal deck are compared in Fig. 3. The beams with metal deck had formed ribs 0.875, 1.312 and 3.0 in. deep. The predicted elastic curve (sloped dashed line) and the predicted ultimate strength (horizontal dashed lines) are also shown. Conventional composite Beam 13 and Beams 11 and 12 with 0.875- and 1.312-in. high ribs all developed the predicted plastic strength. The 0.875- and 1.312-in. high ribs had no significant influence on the ultimate strength.

Beams B1 and B2 with 3-in. high ribs were not able to develop the predicted ultimate flexural capacity for full shear connection. This was expected, because of the fewer stud connectors and the greater influence of the cell geometry. With increasing rib height and usual rib geometry, a substantial reduction in shear connection strength is brought about by concrete cracking at the rib corners and the eventual shearing off of the concrete ribs[6, 9] before full flexural capacity is achieved. A review of available test results indicates that the yield load (the theoretical elastic limit) is about the limit of the beam capacity for the larger rib sizes.[5,8,14] When the shear connection strength is known, the ultimate flexural capacity can be computed using the concept of partial shear connection.[11]

Tests on composite beams with ribs up to 1½ in. high are summarized in Fig. 4. The ratio of measured to computed ultimate moment is plotted as a function of the ratio of rib width to rib height. When the rib width to height ratio was less than about 1.75, a decrease in flexural capacity was noted due to partial shear connection. All beams plotted in Fig. 4 had their shear connection designed according to the AISC Specification provisions. The decrease in flexural capacity for smaller ratios of w/h reflects the decrease in shear connector strength caused by the rib geometry.

Figure 5 summarizes the results of available beam tests, including those reported by Robinson.[5] The flexural capacity is plotted as a function of the rib height. None of Robinson's test beams had a rib width greater than 1½ times the rib height and none were able to develop

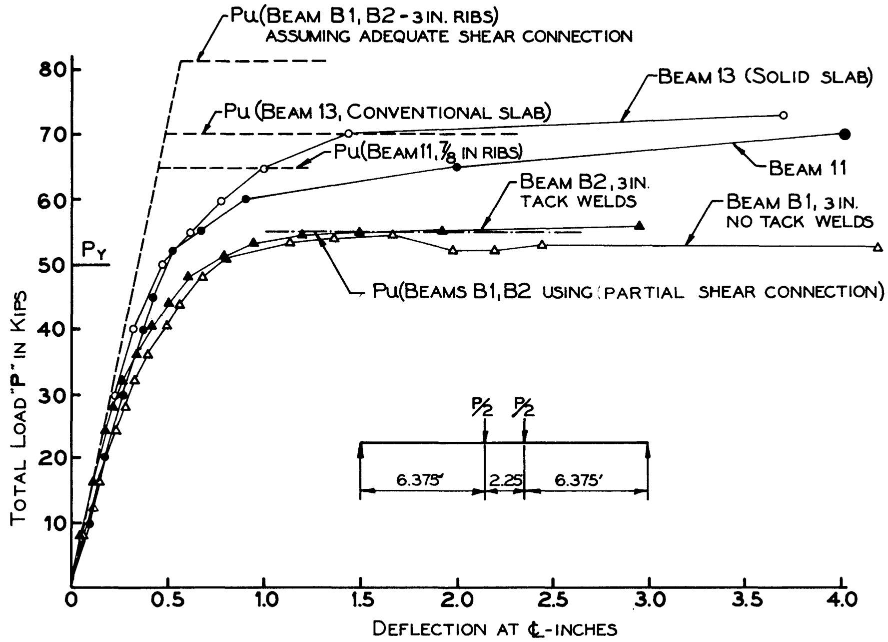

Fig. 3. Load deflection characteristics of composite beams with and without metal deck

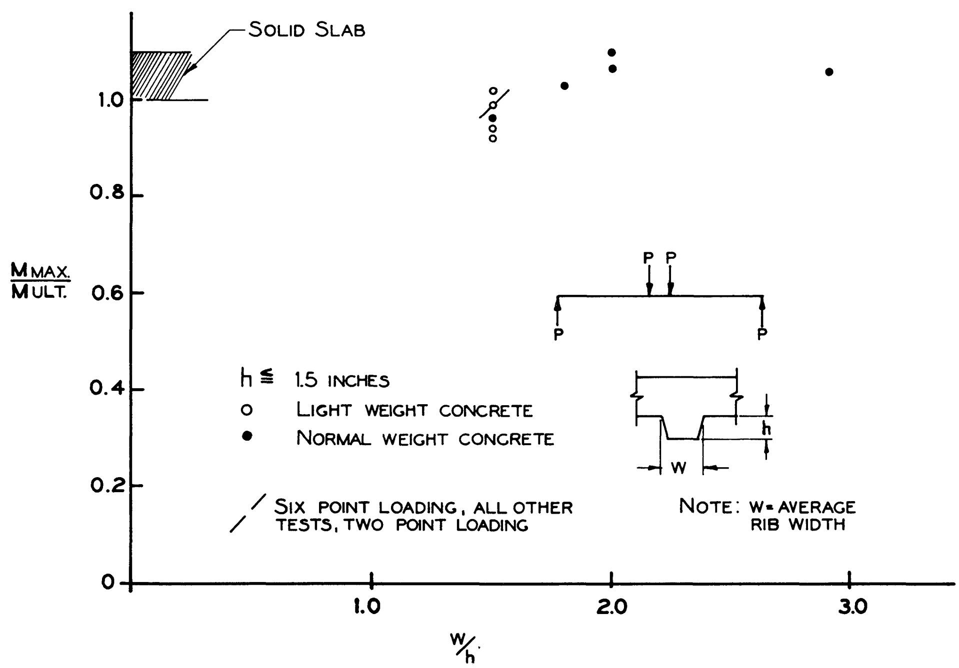

Fig. 4. Effect of rib geometry on ultimate strength of composite beam when rib height is equal or less than $1\frac{1}{2}$ in.

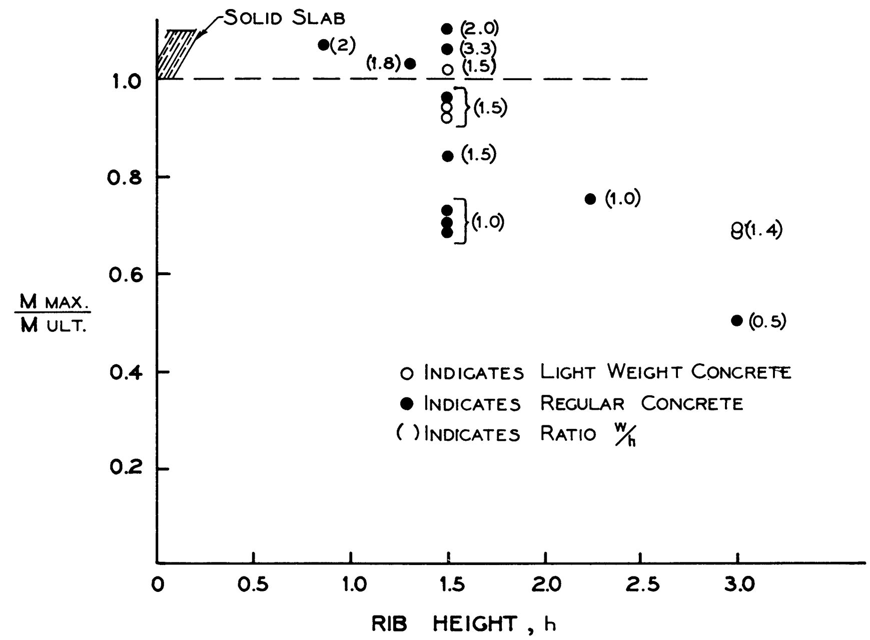

Fig. 5. Effect of rib height and geometry on flexural capacity

the full flexural capacity of the beam. Beams that failed by longitudinal splitting were not considered, as they had inadequate transverse reinforcement. Generally a 6 x 6—10/10 welded wire mesh is used for transverse reinforcement. This has permitted the development of the flexural capacity for most beams with full or partial shear connection. If a large slab force is anticipated because of the use of high strength steel beams or full shear connection, attention should be given to the transverse reinforcement so that the longitudinal shear in the slab is not critical. Johnson[15] has suggested a means of checking the adequacy of the transverse reinforcement for solid slabs.

With increasing rib height, the flexural capacity of the beams shown in Fig. 5 was decreased as a result of the reduction in shear connection strength. The reduction in flexural capacity is analogous to the reduction observed in conventional composite beams when partial shear connection is available.[11] This reduced flexural capacity can be determined when the strength of the shear connection is known.[8, 14]

Shear Connection—With the ratio of rib width to height greater than 1.75, no difficulty has been experienced in developing the flexural capacity of the composite beam with full shear connection (see Figs. 3 and 4). If the bottom of the compressive stress block at ultimate load is above the top of the rib ($a < t - h$), the AISC rule based on steel area provides the number of shear connectors.

If the compressive stress block required to balance the tensile area is greater than the area of concrete above the ribs, the shear connection need not resist the greater force. The concrete area above the ribs is used to determine the number of shear connectors required by the AISC design procedure.

When the rib height is less than 1 in., standard cell geometry has provided satisfactory connector behavior. In these cases, the rib w/h ratio is usually about 2. When the rib height is increased, a decrease in the shear capacity of the connectors occurs and partial shear connection may result, depending on the ratio w/h and the number of ribs in which the connectors can be placed.

The studies by Robinson[5] and others[3, 7, 8] have shown that the shear connection strength may be significantly affected by the cell geometry. Usually cracks initiate in the concrete at the corners of the rib and eventually the concrete slab shears off above the ribs. The concrete around the shear connector governs the strength, as this is where the failure occurs. Usually the shear surface passes over the studs placed in the rib. With increasing width of the concrete slab, an increase in the shear strength was noted, although the increase was not directly proportional to the width. This fact was well established by Inland-Ryerson tests.[9] Doubling the push-off specimen slab width increased the apparent shear connection strength only 35 percent This was probably due to the shear surface not extending the full slab width. Since rib cracking is initiated in the vicinity of the shear studs, the shear cone penetrates into the slab

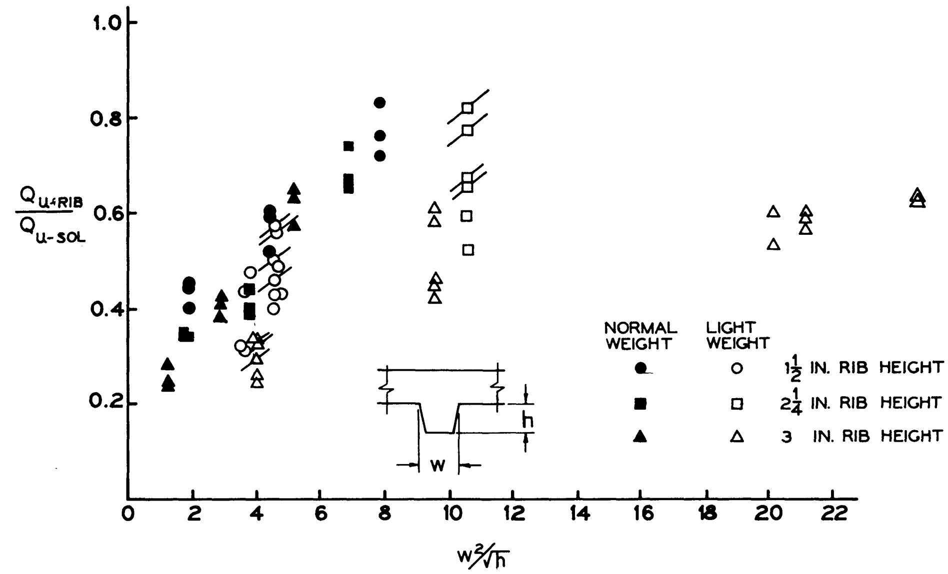

Fig. 6. Connector strength as a function of $w^2/\sqrt{h}$

near the connectors. As the crack propagates through the rib, it follows the path of least resistance for very wide slabs and does not completely shear off the ribs for the full slab width. Hence, the 35 percent increase in strength appears reasonable.

Robinson has suggested that the shear connection strength is proportional to the ratio of rib width squared divided by the square root of rib height. Figure 6 summarizes the apparent strength of the rib expresssed as a ratio of the shear connector strength in a solid slab. Although reasonable correlation is apparent with his own test series, the agreement with other geometric configurations and lightweight concrete is not good. The data for lightweight concrete is from Refs. 8, 9 and 10, and a series of recent push-off tests undertaken by Inland-Ryerson. The strength of lightweight concrete solid slab specimens was about 80 percent of the strength observed in normal weight concrete. Obviously, some of the scatter is caused by specimen geometry as well as type of concrete. As was noted earlier, slab width and connector size both affected the connector strength.

The flexibility of the shear connector has some effect on the behavior of the rib and its ability to resist shear. A more flexible connector permits earlier rotation of the rib, which leads to cracking at the rib corner and precipitates the failure of the concrete by shearing off the ribs. Studies at Inland-Ryerson have illustrated this effect.[10] Increasing the number of connectors in a rib from three ¾-in. studs to four ¾-in. studs had little effect. However, when either five ¾-in. or five ⅝-in. studs were placed in a cell, the increased stiffness and greater area of the concrete shear cone over the connectors increased the shear strength 40 percent.

A better correlation of connector strength and rib geometry is obtained for all specimen configurations if strength is expressed as a function of the ratio of rib width and depth. The results of such a comparison are given in Fig. 7. Included are the tests by Robinson on normal weight concrete and the various tests with lightweight concrete. All lightweight concrete push-off specimens were non-dimensionalized, using the strength of connectors in lightweight solid slabs. Since beam tests have usually indicated some increase over the basic pushout strength, the mean strength of stud connectors installed in metal deck ribs can be defined as

$$Q_{u\text{-}rib} = 0.36 \frac{w}{h} Q_{u\text{-}sol} \tag{1}$$

where

$Q_{u\text{-}rib}$ = ultimate shear connector strength in a cellular rib
$Q_{u\text{-}sol}$ = shear strength in a solid slab
w = average rib width
h = rib height

The data plotted in Fig. 7 shows that the shear capacity is improved when the slab reinforcing steel is clipped to the stud. Also, smaller diameter studs tend to provide a more efficient shear connection. The maximum spacing of shear connectors does not appear to be as critical with formed metal deck. Often when connectors are omitted from a rib, the metal deck is tack welded into place. This provides the necessary tie-down. A number of beams have been tested with utility cells or with connectors spaced at 26 in. or more and have provided satisfactory behavior.

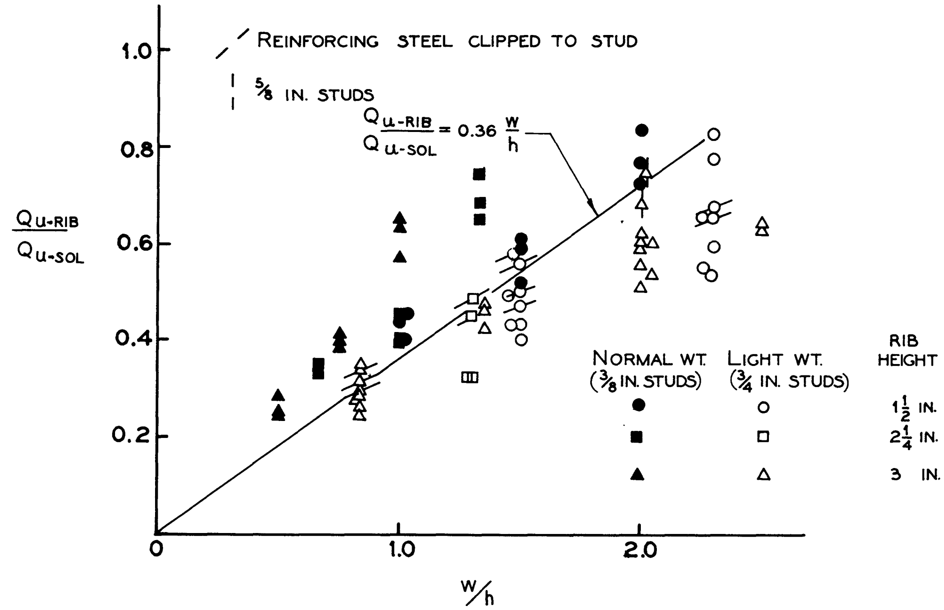

Fig. 7. Connector strength as a function of w/h

Effective Flange Width—Since specifications normally define the effective slab width of a composite beam as a function of the slab thickness, two possibilities exist with the formed metal deck. One is to consider the thickness above the ribs, and the second to consider the total slab thickness including the ribs as though it were a solid slab. All experimental studies have verified that the full width of slab as defined by the total slab thickness should be used.[2, 3, 6, 8] Strain measurements across the slab width have indicated that shear lag is no more severe in the formed metal deck slab than in a solid slab.

It should be noted that it is not conservative to assume otherwise. Selecting a narrower effective slab width may cause the forces acting on the shear connection to be underestimated.

Type of Concrete—Most of the shear connection behavior has been attributed to the type of shear connector, even though the failure was in the concrete slab. In nearly all cases, the slab was sheared off above the ribs of the formed metal deck. Since small cracks formed at the corner of the ribs because of their rotation, the shear strength was apparently decreased with increasing rib height as is illustrated in Figs. 6 and 7.

From the data plotted in Figs. 1, 3, 4 and 5, not much difference is apparent in the behavior of lightweight and regular weight concrete in the slabs of formed metal deck with ribs equal or greater than 1½ in. The tests summarized in Figs. 4 and 5 had compressive strengths that varied from 3 to 6 ksi.

The type of concrete has influenced the strength of the shear connection. Previous work with solid slabs has shown that the shear connector strength is reduced when the connectors are embedded in lightweight concrete.[8, 12] This same reduction is apparent with formed metal deck, as illustrated in Figs. 6 and 7. Recent research[16] has shown that shear connectors embedded in lightweight concrete are reduced in strength in proportion to the square root of the ratio of the moduli of elasticity of lightweight to normal weight concrete $(E_{c\text{-}l}/E_{c\text{-}n})^{1/2}$. This same reduction has been observed for formed metal deck as is apparent in Fig. 7.

TENTATIVE DESIGN RECOMMENDATIONS FOR COMPOSITE BEAMS WITH FORMED METAL DECK UP TO 3 IN. HIGH

When formed metal deck is placed with the ribs perpendicular to the steel beam and shear connectors are placed in the rib troughs, the following design procedures are recommended. These procedures provide at least a factor of safety of two against the flexural capacity of existing beam tests with formed metal deck up to 3 in. high.

Flexural stresses for the composite section with a formed metal deck should be determined at the working load level on the basis of the moment of inertia of the transformed composite section. The full slab depth including the ribs should be used when determining the effective width of the slab.

The transformed area of the composite section should be calculated on the basis of the modular ratio. The value of n for lightweight concrete can be assumed to be the same as for normal weight concrete of the same strength when the rib height is less than $1\frac{1}{2}$ in., except for calculations of deflections. When the rib height exceeds $1\frac{1}{2}$ in., the value of n should be selected on the basis of the type of concrete used.

The stud shear connectors should be designed on the basis of an allowable load, Q_{rib}, determined for studs embedded in concrete, as

$$Q_{rib} = 0.50 \frac{w}{h} Q_{sol} \sqrt{\frac{E_{c-l}}{E_{c-n}}} \qquad (2)$$

where

Q_{rib} = Allowable load for a formed metal rib in which a shear connector is installed
Q_{sol} = AISC allowable horizontal shear load when connectors are embedded in concrete made with ASTM C33 aggregates
w = Average rib width for open rib decks. When inverted trapezoidal ribs are used, w = width at the top of the rib
h = Rib height
E_{c-l} = Modulus of elasticity of lightweight concrete
E_{c-n} = Modulus of elasticity of normal weight concrete

Q_{rib} should not be greater than Q_{sol} and Eq. (2) is not applicable to rib heights which exceed 3 in. When normal weight concrete is used the ratio of the modulus of elasticity $(E_{c-l}/E_{c-n})^{1/2}$ should be taken as unity. For stud shear connectors embedded in structural concrete made with ASTM C330 lightweight aggregates, the allowable load determined from Eq. (2) is directly applicable.

Equation 2 is based on the same margin of safety against the yield load as used for bending stress (1.67). Equation 2 was derived from push-off test data in which two $\frac{3}{4}$-in. studs were placed in a cell. All studs extended above the top of the rib into the solid portion of the slab. It provides a lower bound to the strength of the shear connection. As noted earlier, smaller diameter studs and single studs placed in a rib tend to provide a greater margin of strength. Also, beam tests have indicated good behavior up to the yield load, even when partial shear connection was provided. Results of special tests on beams may permit more liberal shear connector values, depending upon the connector spacing, slab width and reinforcement, connector size and rib geometry.

Until further work is available, studs greater than $\frac{3}{4}$-in. diameter should not be used. Also, stud shear connectors should be as long as possible, so that they extend above the rib into the solid slab. It is recommended that the stud extend $1\frac{1}{2}$ in. above the top of rib. The positioning and placement of the shear connectors should be in accordance with the AWS specification.[17] The clearance between the stud and the formed metal deck is not critical and need not be considered. Any decrease associated with it is reflected in Eq. (2) for the open rib type deck considered in this study. (Open rib type deck has the top rib width equal to or greater than the bottom rib width.) The normal clearances established for stud shear connectors are for ease of installation and to minimize the possibility of spalling of the slab surface.

Except for the procedures recommended in this section, the design considerations should be in accordance with the AISC Specification. The following cases illustrate the application for several design cases.

Case 1—Ribs neglected; $h \leq 1\frac{1}{2}$ in. and $a < (t - h)$: The concrete slab may be treated as though it had full thickness if the bottom of the compressive stress block a does not extend below the top of the rib. The compressive stress block depth is determined by:

$$a = \frac{A_s F_y}{0.85 f_c' b} \qquad (3)$$

where

A_s = area of steel section, in.^{2}
F_y = specified yield point of steel beam, ksi
f_c' = compressive strength of concrete, ksi
b = effective width of concrete slab, in.

The standard AISC provisions should be used except for the shear connector capacity. The allowable value for connectors placed in ribs is given by Eq. (2).

Case 2—Corrugations considered; $h \leq 1\frac{1}{2}$ in. and $a > (t - h)$:

Case 3—Corrugations considered; 3 in. $\geq h \geq 1\frac{1}{2}$ in.: When the depth of the compression block given by Eq. (3) is greater than the slab thickness less the height of ribs up to $1\frac{1}{2}$ in., the strength of the composite beam is limited by the concrete slab and section properties are affected by the ribs. The beam strength and section properties are also affected when the height of the rib corrugations exceed $1\frac{1}{2}$ in.

The following provisions provide for these effects.

(a) Use the full slab thickness to the bottom of the steel deck to determine the effective width.

(b) Calculate section properties using the transformed section; concrete below the top of the steel deck ribs should not be considered effective. With $a > (t - h)$ the neutral axis of the elastic section is usually well below the top of the ribs and some section properties are affected by the loss of concrete area in compression. The moment of inertia and the section modulus for the top of the tee-section are affected, but the section modulus for the bottom of the tee-section is not affected. The steel stress in the bottom beam fibers usually governs the design, so it is possible to utilize existing charts and tables for solid slab sections when selecting the steel section. (See the AISC Manual or Bethlehem Steel Handbook 2346: Properties of Composite Sections for Buildings.)

Approximate values of the effective moment of inertia and top of the tee beam section modulus can be taken as

$$I_{eff} = \left(1 - \frac{h}{5t}\right)I \tag{4}$$

$$S_{t-eff} = \left(1 - \frac{h}{2t}\right)S_t \tag{5}$$

where I and S_t refer to cross-section properties of solid slab composite beams and I_{eff} and S_{t-eff} are approximate cross-section properties of the composite beam with formed metal deck.

(c) For full flexural capacity, the horizontal shear connection should be proportioned to resist the maximum force possible in the concrete slab. This may be taken as the smaller of the values given by:

$$V_h = 1/2\ [0.85\, f_c'b(t - h)] \tag{6}$$

or

$$V_h = \frac{A_sF_y}{2} \tag{7}$$

The required number of connectors between the points of zero and maximum moment is obtained by dividing the horizontal shear force by the allowable load per connector given by Eq. (2).

In cases where it is not feasible or necessary to provide adequate connectors to satisfy Eq. (6) or (7) for full composite action, the effective section modulus should be determined as

$$S_{eff} = S_s + \frac{V_h'}{V_h}(S_b - S_s) \tag{8}$$

where

S_s = section modulus of the steel beam
S_b = section modulus of the bottom of the composite tee-section
V_h = smaller value given by Eqs. (6) or (7)
V_h' = total horizontal shear for the partial shear connection (ΣQ_{rib}).

Robinson[13] has shown that the resulting effective section modulus is a conservative value for a composite beam whose flexibility is increased due to the presence of ribbed or cellular metal deck. The partial shear connection should not be less than 50 percent of the shear connection necessary to develop the full flexural capacity. Further reduction reduces the beam stiffness and subjects the shear connection to excessive loads at the working load level.

(d) If lightweight concrete is used in the deck slab, the value of n should be determined from the modulus for lightweight concrete. The allowable shear connector values provided by Eq. (2) should be reduced by the ratio $(E_{c\text{-}l}/E_{c\text{-}n})^{1/2}$.

Illustrative example for Case 2—

Given:

Span of Beams: 40 ft
Beam Spacing: 7 ft
ASTM A36 Steel: $F_y = 36{,}000$ psi
$F_b = 0.66F_y = 24$ ksi
Concrete Strength: $f_c' = 3000$ psi ($n = 9$)
(normal weight) $f_c = 0.45\, f_c' = 1.35$ ksi
Loading: Uniform live load of 200 lbs/ft²
A 4-in. concrete slab with a 1½-in. thick composite metal deck $w/h = 1.5$.

Solution:

Step 1: Determine the required section modulus and make preliminary selection of steel section:

Effective slab width: $b = 16 \times 4 + 8 = 72$ in.
Transformed slab width: $b/n = 8$ in.

Assumed beam weight:	60 lbs/ft
Slab weight:	260
Live Load:	1400
	1720 lbs/ft

$$\text{Moment} = \frac{wl^2}{8} = 1.72\frac{(40)^2}{8} = 344 \text{ kip-ft}$$

Required section modulus:

$$S_b = \frac{344 \times 12}{24} = 172 \text{ in.}^3 \text{ req'd.}$$

From pg. 2–97 of the 6th Edition AISC Manual, select 21W62 with $S_b = 172.4$ in.³ furnished

Step 2: Check a:

$$t - h = 4 - 1.5 = 2.5 \text{ in.}$$

$$a = \frac{18.23 \times 36}{0.85 \times 3 \times 72} = 3.58 \text{ in.} > t - h$$

Step 3: Compute section properties and check design: From pg. 2–100 of the 6th Edition AISC Manual, the following properties are obtained:

$$S_t = 488.1 \text{ in.}^3, \quad I = 3184.9 \text{ in.}^4$$

In lieu of exact calculations, Eqs. (4) and (5) can be used to obtain the effective section properties

$$S_{t-eff} = \left(1 - \frac{h}{2t}\right) S_t = \left(1 - \frac{1.5}{2 \times 4}\right) 488.1$$

$$= 396.1 \text{ in.}^3 \text{ (Note exact value } = 397.7 \text{ in.}^3)$$

The effective moment of inertia would be calculated if a deflection check is necessary.

Step 4: Check stresses in composite beam slab:

$$\text{Top of slab} = \frac{344 \times 12}{396.1 \times 9}$$

$$= 1.16 \text{ ksi} < 1.35 \text{ ksi}$$

(Note: if unshored beam, only live load should be considered.)

Step 5:

Shear connectors:

Use ¾-in. x 3 in. stud shear connectors.

Horizontal shear:

$$V_h = 1/2(0.85)(3)(72)(2.5)$$
$$= 229.5 \text{ kips}$$

Allowable load for connectors from Eq. (2):

$$Q_{rib} = 0.50(1.5)(11.5) = 8.63 \text{ kips}$$

Required number of connectors:

$$N = \frac{V_h \times 2}{Q_{rib}}$$

$$= \frac{229.5 \times 2}{8.63} = 53.2$$

Use 54 studs.

ACKNOWLEDGMENTS

This report was made possible through the support of Nelson Stud Welding Co., A United-Carr Division of TRW Inc. The author is also indebted to the Inland-Ryerson Construction Products Co. and the H. H. Robertson Co. for making available test data and co-operating with this study.

Special thanks are also due Messrs. I. M. Hooper, R. C. Singleton, J. H. Ryan, A. B. Thelen, R. Van Sicklin and B. T. Jones for their comments and advice.

REFERENCES

1. *Jones, B. T.* The Flexural Strength of Steel and Concrete Composite Beams Using Cellular Floor *Technical Data Report No. 64-23, H. H. Robertson Co., Pittsburgh, Pa., June 1964.*
2. *Slutter, R. G. and Adams, R. G.* Tests of Composite Beams with Holorib Composite Slabs *Report No. 200.63.408.2, Fritz Laboratory, Lehigh University, Bethlehem, Pa., July 1964.*
3. *Seaburg, P. A.* Static Load Tests of Composite Beams with Hi-Bond and Lightweight Concrete—Final Report *Inland Steel Products Co., December 1966.*
4. *Slutter, R. G.* Test of Cincinnati Center Composite Beam *Report No. 200.67.458.2, Fritz Laboratory, Lehigh University, January 1968.*
5. *Robinson, H.* Tests on Composite Beams with Cellular Deck *Journal of the Structural Division, ASCE, Vol. 93, No. ST4, August 1967.*
6. *Bethlehem Steel Corporation.* Bethlehem Slabform in Composite Construction *Technical steel talk No. 26, March 1965.*
7. *Errera, S. J.* Analysis of Tests of Composite Steel and Concrete Beams with Mahon Steel Decking.
8. *Fisher, J. W., Kim, S. W. and Slutter, R. G.* Tests of Lightweight Concrete Composite Beams and Pushout Specimens with Cellular Steel Deck *Report 200.67.438.1, Fritz Laboratory, Lehigh University, July 1967.*
9. *Thelen, A. B., Weiler, T. J. and Wiker, R. L.* Push-Off Tests with ¾ in. and ⅝ in. Studs, 3000 psi Solite Concrete and Inland's 3N, V Hi-Bond Floor Deck *Project Report 7065.5, November 1968.*
10. *Weiler, J. J. and Thelen, A. B.* Push-Off Tests with ¾ in. and ½ in. Nelson Studs, Materialite Concrete and Inland's 3N, V Hi-Bond Floor Deck *Project Report 7065.2, July 1968.*
11. *Slutter, R. G. and Driscoll, G. C.* Flexural Strength of Steel-Concrete Composite Beams *Journal of the Structural Division, ASCE, Vol. 91, No. ST 2, April 1965.*
12. *Baldwin, J. W., Henry, J. R. and Sweeney, G. M.* Studies of Composite Bridge Stringers *Phase II, University of Missouri, May 1965.*
13. *Robinson, H. and Uppal, A. S.* Variation of Section Modulus Along the Length of a Composite Beam *presented at Symposium on Composite Construction Queen's University at Kingston, Ontario, June, 1969.*
14. *Robinson, H.* Composite Beam Incorporating Cellular Steel Decking *Journal of the Structural Division, ASCE, Vol. 95, No. ST5, March, 1969.*
15. *Johnson, R. P.* Longitudinal Shear in Composite Beams *Report of the College of Engineering, University of Missouri, Columbia, April 1969.*
16. *Ollgaard, J., Slutter, R. G. and Fisher, J. W.* The Strength of Stud Shear Connectors in Lightweight and Normal Weight Concrete *Fritz Engineering Laboratory Report 360.2, in preparation.*
17. *American Welding Society* Supplement to AWS D1.0.66 Code for Welding in Building Construction and AWS D2.0-66 Specifications for Welded Highway and Railway Bridges on Requirements for Stud Welding *September 1968.*