Shear Attachments for Composite Construction—Building

1. BASIC REQUIREMENTS

The concrete floor may be attached to the top flanges of the steel girders or beams by the use of suitable shear connectors. These allow the slab to act with the steel and form a composite beam having greater strength and rigidity.

The concrete slab becomes part of the compression flange of this composite element. As a result, the neutral axis of the section will shift upward, making the bottom flange of the beam more effective in tension. By such an arrangement, beam cross-sections and weight can be reduced. Since the concrete already serves as part of the floor, the the only additional cost will be the shear connectors.

The types of shear connectors in use today take various shapes and sizes. Some typical ones are shown in Figure 1.

In addition to transmitting the horizontal shear forces from the slab into the steel beam making both beam and slab act as a unit, the shear connector provides anchorage for the slab. This prevents any tendency for it to separate from the beam. While providing for these functions, connector placement must not present difficulty in the subsequent placing of reinforcing rods for the concrete slab.

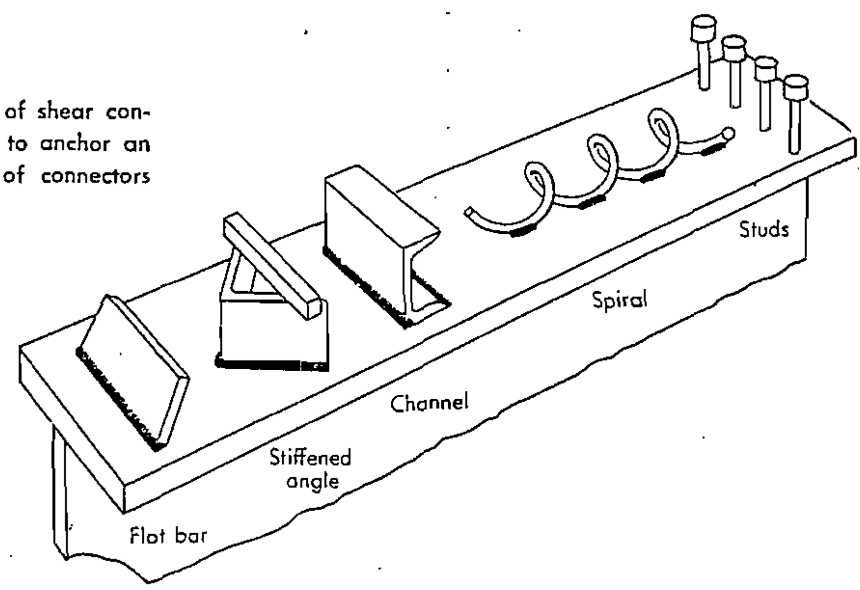
Because of lower shop costs and better conditions, it is more economical to install these connectors in the shop. However, this may be offset by the possibility of damage to them during shipping, and by the difficulty presented to walking along the top flanges during erection before the slab is poured. For the latter reasons, there is a growing trend toward field installation of connectors.

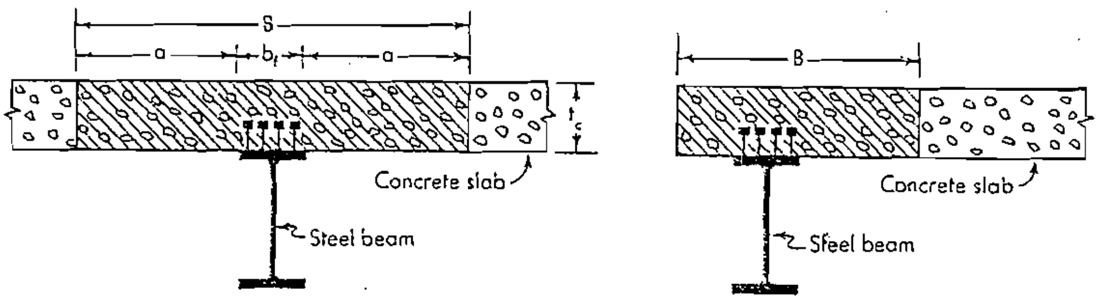
The previous AISC Specifications had no information on the use of shear attachments for use in composite construction. If shear attachments were to be used, AASHO allowables were followed. These require the use of rather long formulas to determine the individual factor of safety to be used on the connector. It also made a difference whether the beam was to be shored or not shored during the placing of the concrete floor.

Factor of Safety

The new AISC Specifications recognize the use of shear attachments and, as a result of recent research on this subject, has taken a more liberal stand on this. The design work has been greatly reduced, and no longer is it necessary to compute the factor of safety. A more liberal factor of safety is now included in the shear connection formulas. The use of shoring is no longer a factor in the design calculations of the connector, since it has been found that the ultimate load carrying

FIG. 1 Representation of five common types of shear connectors welded to top flange of steel girder to anchor an overlayer of concrete. Only short portions of connectors are sketched.





(a) Slab on both sides of beam (b) Slab on one side of beam

FIGURE 2

capacity of the composite beam is unaffected whether shores have or have not been used.

Shear Connector Spacing.

AASHO requires the determination of shear connector spacing, which may vary along the length of the beam. Now AISC requires just one determination of spacing, and this value is used throughout the length of the beam, greatly simplifying the work. This is because the allowables are such that at ultimate loading of the composite beam, some of the connectors will yield before the others. This movement provides a redistribution of shear transfer so that all connections are ultimately loaded uniformly, hence uniform spacing is allowed.

Composite Section Properties

A further help is a series of tables listing properties of possible combinations of rolled beams with typical concrete slab sections, similar to tables in wide use for available rolled beam sections.

These new tables have been published in the AISC "Manual of Steel Construction," Sixth Edition, 1963, and in Bethlehem Steel Co.'s "Properties of Composite Sections for Bridges and Buildings."

The new tables eliminate the various calculations for composite sections. A simple calculation will indicate the required section modulus of the composite section, and a quick reference to the tables will indicate possible combinations of rolled beam and concrete slab.

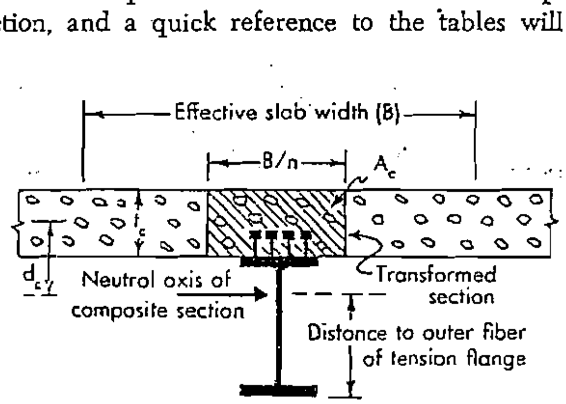


FIGURE 3

2. DESIGN OF CONNECTORS

In order to get the transformed area of the concrete floor, it is necessary to decide how large a width of the concrete acts along with the steel beam to form the composite section. This is known as the effective width (B) of the slab. AISC (1.11.1) requires the following:

slab on both sides of beam, Figure 2(a)

$B \leqq$ ¼ beam span

$a \leqq$ ½ distance to adjacent beam

$a \leqq$ 8 times least thickness of slab (t_c)

slab on one side of beam, Figure 2(b)

$B \leqq$ 1⁄12 beam span

$B \leqq$ ½ distance to adjacent beam

$B \leqq$ 6 times least thickness of slab (t_c)

This effective width of concrete is now transformed into an equivalent steel section, having the same thickness as the concrete (t_c), but having a width equal to 1/n that of the concrete. See Figure 3. Here n, the modular ratio, is the ratio of the modulus of elasticity of the steel to that of the concrete.

From this transformed section, the various properties of the section may be determined.

I = moment of inertia of transformed section, in.^{4}

S = section modulus for the extreme tension fibers of the steel beam (bottom flange), in.^{3}

Beams may be totally encased within the floor slab as a Tee section in which the top of the beam is at least 1½" below the top and 2" above the bottom of the slab, and encased with at least 2" of concrete around the sides of the beam. With these conditions, shear attachments are not used (AISC 1.11.1).

If no temporary shores are used, the total bending stress in the tension flange of the encased steel beam is figured under two conditions:

1. The steel beam acting alone for any dead loads applied prior to hardening of the concrete.

2. The steel beam acting with the concrete for any live loads and additional dead loads applied after hardening of the concrete.

The beam shall be so proportioned that the above stress under either condition does not exceed .66 σ_y* (AISC 1.11.2.1).

If temporary shores are used, the tension steel flange of the encased beam acting with the concrete slab to form the composite section shall be designed at $\sigma = .66\ \sigma_y$* to carry all dead and live loads applied after hardening of the concrete.

If shear attachments are used, encasement is not needed and it does not matter in the design whether temporary shores are used or not used. In either case, the steel tension flange acting with the concrete slab to form the composite section shall be designed at $\sigma = .66\ \sigma_y$* to carry all of the loads (AISC 1.11.2.2). If no temporary shoring is used, the section modulus of the composite section (S_c) in regard to the tension flange of the beam shall not exceed the following:

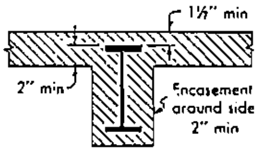
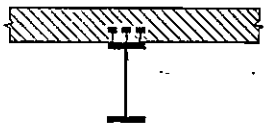
$$\boxed{S_c \leqq \left(1.35 + 0.35\,\frac{M_L}{M_D}\right) S_s} \quad \text{(AISC Formula 17)} \qquad \ldots\ldots\ldots\ldots\ldots (1)$$

where:

S_c = section modulus of composite section (relative to its tension steel flange)

* If steel section is not compact: $\sigma = .60\ \sigma_y$.

TABLE 1—Design of Section for Composite Construction

	Encased Beams (1.11.2.1) (no shear attachments)		With Shear Attachments (1.11.2.2)	
				
	Section Modulus Used	Loads Used *	Section Modulus Used	Loads Used*
With Shoring	composite section	with all loads after hardening of concrete	composite section	with all loads
	$\sigma_s = \frac{M_D + M_L}{S_c} \leqq .66\ \sigma_y \leqq .60\ \sigma_y$		$\sigma_s = \frac{M_D + M_L}{S_c} \leqq .66\ \sigma_y \leqq .60\ \sigma_y$	
Without Shoring	steel beam	with all loads prior to hardening of concrete	composite section	with all loads
	and			
	composite section	with all loads after hardening of concrete		
	$\sigma_s = \frac{M_D}{S_s} + \frac{M_L}{S_c} \leqq .66\ \sigma_y \leqq .60\ \sigma_y$		$\sigma_s = \frac{M_D + M_L}{S_c} \leqq .66\ \sigma_y \leqq .60\ \sigma_y$	
	or		also	
	steel beam	with all loads	$S_c \leqq \left(1.35 + 0.35\,\frac{M_L}{M_D}\right) S_s$ (AISC formula 17)	
	$\sigma_s = \frac{M_D + M_L}{S_s} \leqq .76\ \sigma_y$			

* $\sigma = .66\ \sigma_y$ for "compact" beams; otherwise $\sigma = .60\ \sigma_y$

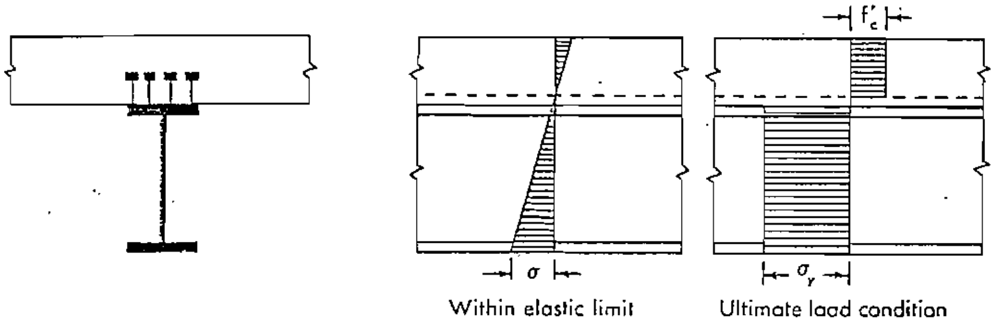


FIGURE 4

S_s = section modulus of steel beam (relative to its tension flange)

M_D = dead-load moment prior to hardening of concrete

M_L = moment due to live and additional dead load after hardening of concrete

Table 1 summarizes these requirements for encased beams without shear attachments and for composite beams with shear attachments.

Forces Carried by Connectors

For elastic design, the horizontal unit shear force is obtained from the well-known formula:

$$f = \frac{V\ a\ y}{I}$$

However in the new AISC Specification for building applications, the design is based on the shear connectors allowing the composite beam to reach ultimate load. In the usual composite beam, the ultimate load is reached after the full depth of the steel beam reaches yield stress in tension. This force is resisted by the compressive area of the concrete slab. See Figure 4.

The total horizontal shear (V_h) at ultimate load to be transferred from concrete slab to steel beam between section of maximum moment and ends of the beam, is equal to the total horizontal forces (F_h) from bending acting on either the slab or the beam. See Figure 5.

where:

B = effective width of slab

t_c = thickness of slab

f'_c = compressive strength of concrete

A_s = cross-sectional area of steel beam

A_c = cross-sectional area of effective concrete slab

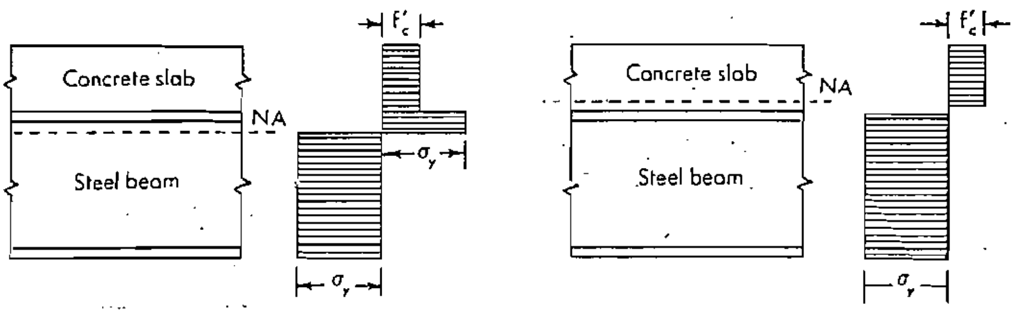
σ_y = yield strength of steel

Figure 6 diagrams the bending moment that results in horizontal forces; compression in the concrete slab and tension in the steel beam.

These horizontal ultimate forces are then reduced by a factor of safety of 2, and concrete is taken at 85% of its strength. These formulas become:

$$\boxed{V_h = \frac{.85\ f'_c\ A_c}{2}} \qquad (2)$$

(AISC Formula 18)



(a) Neutral axis lies within steel beam

$V_h = F_h = b\ t_c\ f'_c$

(b) Neutral axis lies within concrete slab

$V_h = F_h = A_s\ \sigma_y$

FIGURE 5

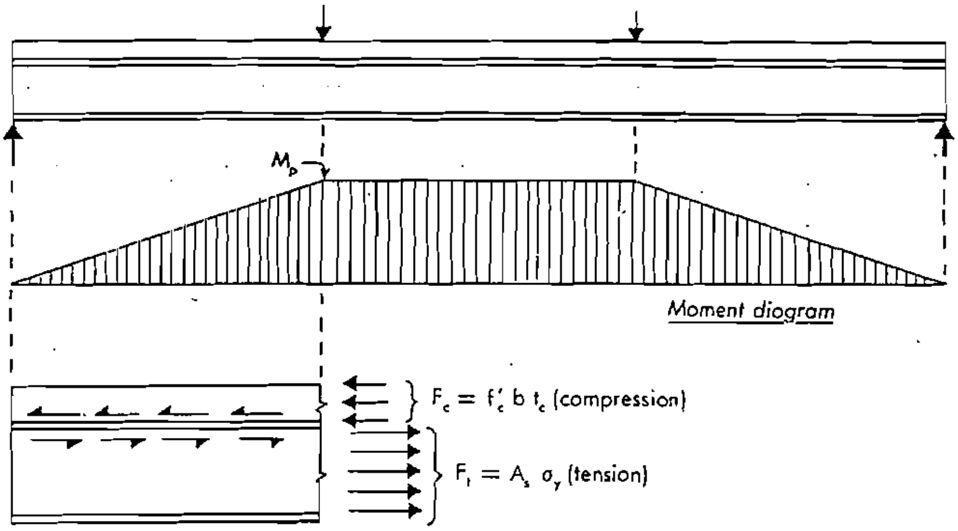


FIGURE 6

$$\boxed{V_h = \frac{A_s\,\sigma_y}{2} \quad \text{(AISC Formula 19)}} \quad \ldots\ldots\ldots\ldots\ldots\ldots (3)$$

The smaller of the two values above (V_h) is taken as the total horizontal shear force to be carried by all of the connectors between the point of maximum moment and the ends of the beam, or between the point of maximum moment and a point of contra-flexure in continuous beams.

The number of shear connectors needed within this region is found by dividing the above force (V_h) by the allowable (q) for the type of connector used.

Allowable Loads

Formulas have been established to give the useful capacity of three types of shear connections. These are used by AASHO in the bridge field with the proper values of (K):

stud

$$q = K_1\, d_s^2\, \sqrt{f'_c} \qquad \text{(lbs/stud)}$$

channel

$$q = K_2\,(h + ½\, t)\,w\, \sqrt{f'_c} \qquad \text{(lbs/channel)}$$

where:

w = channel length in inches

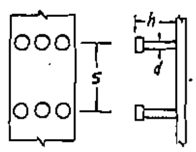
spiral

$$q = K_3\, d_b\, \sqrt[4]{f'_c} \qquad \text{(lbs/turn of spiral)}$$

Later the Joint ASCE-ACI Committee on Composite Construction recommended these same basic formulas, but applied a factor of safety of 2 and these became allowable loads for the connectors.

In the meantime additional testing has indicated the connectors to have greater strength than previously thought. Although AISC did not publish these final formulas with their constants (K), they did produce Table 1.11.4 of values for allowable loads on some of the typical standard shear connectors. See Table 2.

Working back from this table, the basic formulas for allowable loads on shear connectors would be the following:



$$\boxed{q = 372\, d^2\, \sqrt{f'_c} \quad \text{(when } h/d = 4.2\text{)}} \quad \ldots\ldots\ldots (4)$$

TABLE 2—Allowable Horizontal Shear Load (q), Kips
(Applicable Only to Stone Concrete)

Connector	f'_c = 3,000	f'_c = 3,500	f'_c = 4,000
½" diam. × 2" hooked or headed stud	5.1	5.5	5.9
⅝" diam. × 2½" hooked or headed stud	8.0	8.6	9.2
¾" diam. × 3" hooked or headed stud	11.5	12.5	13.3
⅞" diam. × 3½" hooked or headed stud	15.6	16.8	18.0
3" channel, 4.1 lb.	4.3w	4.7w	5.0w
4" channel, 5.4 lb.	4.6w	5.0w	5.3w
5" channel, 6.7 lb.	4.9w	5.3w	5.6w
½" diam. spiral bar	11.9	12.4	12.8
⅝" diam. spiral bar	14.8	15.4	15.9
¾" diam. spiral bar	17.8	18.5	19.1

w = length of channel in inches.

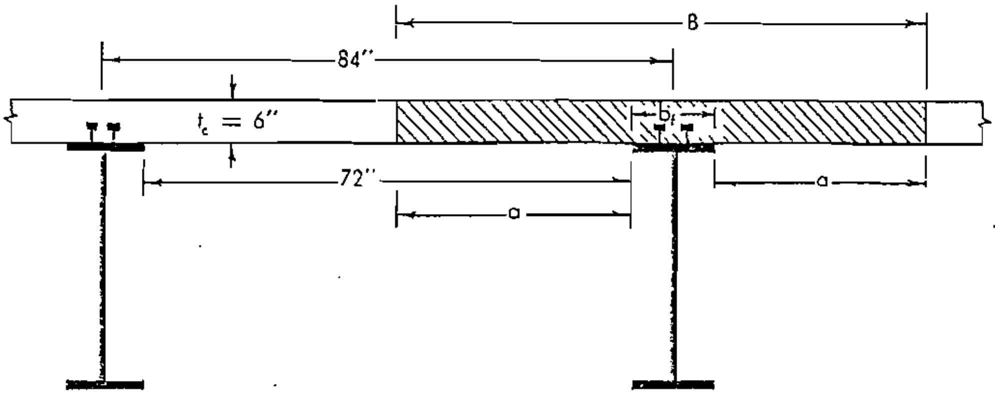
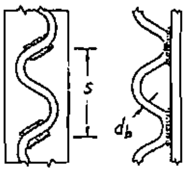
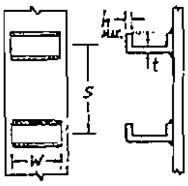


FIGURE 7



$$\boxed{q = 166(h + \tfrac{1}{2}\, t)w\sqrt{f'_c}} \ldots (5)$$



$$\boxed{q = 3200\, d\, \sqrt[4]{f'_c}} \ldots\ldots\ldots\ldots (6)$$

These will enable the engineer to compute the value for a shear connector not covered in the AISC table.

The connectors may be spaced evenly along this region and shall have at least 1″ of concrete cover in all directions.

Problem 1

Check the composite beam of Figure 7, and its shear connectors. The following are given conditions:

36″ WF 150-lb beams on 7′ centers, with a 6″ thick concrete slab
A36 steel, E70 welds, and 3000 psi concrete
A uniformly distributed live load of 240 kips
Span of 40′ between supports

$$n = \frac{E_s}{E_c} = 10 \text{ (modular ratio)}$$

dead load moment

Steel beam	=	6,000 lbs
Concrete slab	=	20,160 lbs
Total W_D	=	26,160 lbs

$$M_D = \frac{W_D\, L}{8} = \frac{(26{,}160)(480)}{8} = 1570 \text{ in.-kips}$$

live load moment

$$M_L = \frac{W_L\, L}{8} = \frac{(240{,}000)(480)}{8} = 14{,}400 \text{ in.-kips}$$

projection of concrete slab

$$a \leqq 8\, t_c \leqq 8(6'') \leqq 48''$$

$$a \leqq \tfrac{1}{2} \text{ distance to adjacent beam} \leqq \tfrac{1}{2}(84 - 12) \leqq 36'' < 48'' \quad OK$$

effective width of concrete flange acting with beam

$$B \leqq \tfrac{1}{4} \text{ beam span} \leqq \tfrac{1}{4}(40) \leqq 10' \text{ or } 120''$$

$$B = 2a + b_f = 2(36) + (12) = 84'' < 120'' \quad OK$$

and width of transformed concrete area is

$$B/n = \frac{84''}{10} = 8.4''$$

properties of steel beam section

36″ WF 150-lb beam

$I = 9012.1$ in.^{4}

$S = 502.9$ in.^{2}

$A_s = 44.15$ in.^{2}

$d_b = 35.84''$

$b_f = 11.972''$

$t_f = .940''$

$t_w = .625''$

properties of composite section

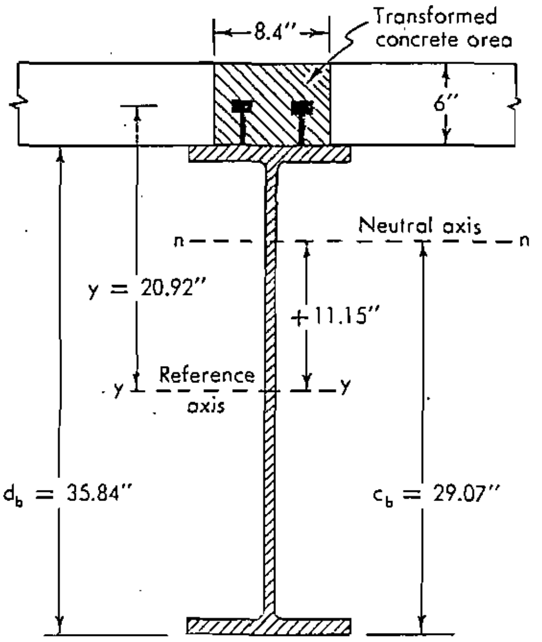


FIGURE 8

Taking reference section (y-y) through the beam's center of gravity:

Area	A	y	$M = A\,y$	$I_y = M\,y$	$I_g = \frac{(B/n)t^3}{12}$
Transformed slab	50.40	+ 20.92	1054.37	22,057.4	151.2
36″ WF 150 lb. beam	44.16	0	0	0	9,012.1
Total →	94.56		1054.37	31,220.7	

$$I_n = (I_y + I_g) - \frac{M^2}{A}$$

$$= (31{,}220.7) - \frac{(1054.37)^2}{(94.56)}$$

$$= 19{,}462 \text{ in.}^4$$

$$N.A. = \frac{M}{A} \text{ (distance from reference axis to neutral axis)}$$

$$= \frac{(1054.37)}{(94.56)}$$

$$= 11.15''$$

$$c_b = (11.15) + (17.92)$$

$$= 29.07''$$

$$S = \frac{I_n}{c_b}$$

$$= \frac{(19{,}462)}{(29.07)}$$

$$= 670 \text{ in.}^3 \text{ (relative to bottom tension flange in steel beam)}$$

check bending stress in beam

Check the tensile bending stress in bottom flange of steel beam. From Table 1—

$$\sigma_s = \frac{M_D + M_L}{S_c}$$

$$= \frac{(1570 + (14{,}400)}{(670)}$$

$$= 23{,}800 \text{ psi} < .66\ \sigma_y$$

check section modulus

Since no shores are to be used, a further requirement is that the section modulus of the composite section shall not exceed—

$$S_{c(max)} \leqq \left[1.35 + 0.35\ \frac{M_L}{M_D}\right] S_s$$

$$\leqq \left[1.135 + 0.35\ \frac{(14{,}400)}{(1570)}\right] (502.9)$$

$$\leqq 2290 \text{ in.}^3$$

$$S_{c(actual)} = 670 \text{ in.}^3 < 2290 \text{ in.}^3 \qquad OK$$

horizontal shear

The horizontal shear to be transferred by connectors will be the smaller of the following two values:

$$V_h = \frac{.85\ f'_c\ A_c}{2}$$

$$= \frac{.85\ (3000)(6 \times 84)}{2}$$

$$= 642.6 \text{ kips}$$

$$V_h = \frac{A_s\ \sigma_y}{2}$$

$$= \frac{(44.16)(36{,}000)}{2}$$

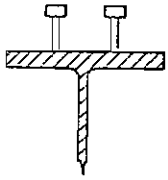
$$= 794.9 \text{ kips}$$

So, use $V_h = \underline{642.6 \text{ kips}}$

Stud Connectors

Use ¾" x 4" studs. From Table 2, q = 11.5 kips per stud.

number of studs



$$n = \frac{V_h}{q}$$

$$= \frac{(642.6)}{(11.5)}$$

$$= 55.9$$

or 60 studs from centerline to each end of beam.

If using 2 rows of studs, use 28 lines on each end of girder.

approximate spacing

$$s = \frac{240'' \text{ (half length)}}{28 \text{ (studs)}}$$

$$= 8.57'' \text{ or } \underline{8\tfrac{9}{16}''}$$

Place first line of studs at ½ of this space (or 4¼") from end of beam; from there on give all studs full spacing (8⁹⁄₁₆").

Channel Connectors

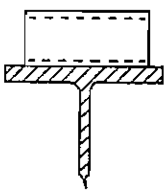
Use 4" 5.4-lb channel of 10" length. From Table 2,

$$q = 4.6 \text{ w}$$

$$= 4.6 \text{ (10)}$$

$$= 4.6 \text{ kips per channel}$$

number of channels



$$n = \frac{V_h}{q}$$

$$= \frac{(642.6)}{(4.6)}$$

$$= 14 \text{ channels}$$

from centerline to each end of beam, or 28 channels per beam.

approximate spacing

$$s = \frac{240'' \text{ (half length)}}{14 \text{ (channels)}}$$

$$= 17.15'' \text{ or } \underline{17\tfrac{3}{16}''}$$

and use ½ of this or 8½" for spacing first channel from end of beam.

To compute the required size of connecting weld:

$$F = 46 \text{ kips, each channel}$$

length of fillet weld

$$L = 2 \times 10''$$

$$= 20''$$

force on weld

$$f = \frac{F}{L}$$

$$= \frac{(46)}{(20)}$$

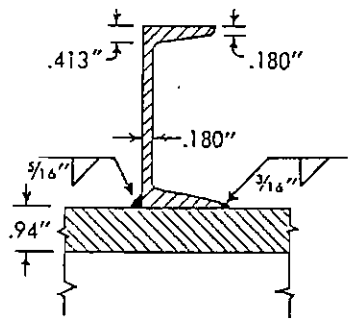
$$= 2300 \text{ lbs/in.}$$

leg size of weld (E70)

$$\omega = \frac{2300}{11{,}200}$$

$$= .205'' \text{ or } \underline{\textit{use } \tfrac{1}{4}''}$$

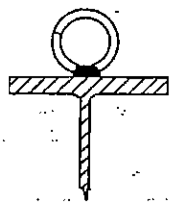
Check: Welding to .94" thick flange calls for minimum weld size of ⁵⁄₁₆" , but the weld need not exceed thickness of the thinner part joined, which is the flange of the channel. Hence, use ⁵⁄₁₆" at the heel and ³⁄₁₆" at the toe.



Spiral Connector

Use ¾" diameter bar. From Table 2, q = 17.8 kips per turn.

number of turns



$$n = \frac{V_h}{q}$$

$$= \frac{(642.6)}{(17.8)}$$

= 36.1 from end to end or 37 turns from centerline to each end of beam.

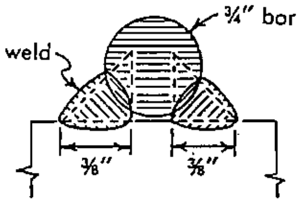
approximate pitch

$$s = \frac{240'' \text{ (half length)}}{37 \text{ (turns)}}$$

$$= 6.49'' \text{ or } \underline{\textit{use } 6\tfrac{7}{16}''}$$

To compute the required connecting welds (E70), assume weld size is equivalent to a ⅜″ fillet weld (has same throat). Force on the weld is—

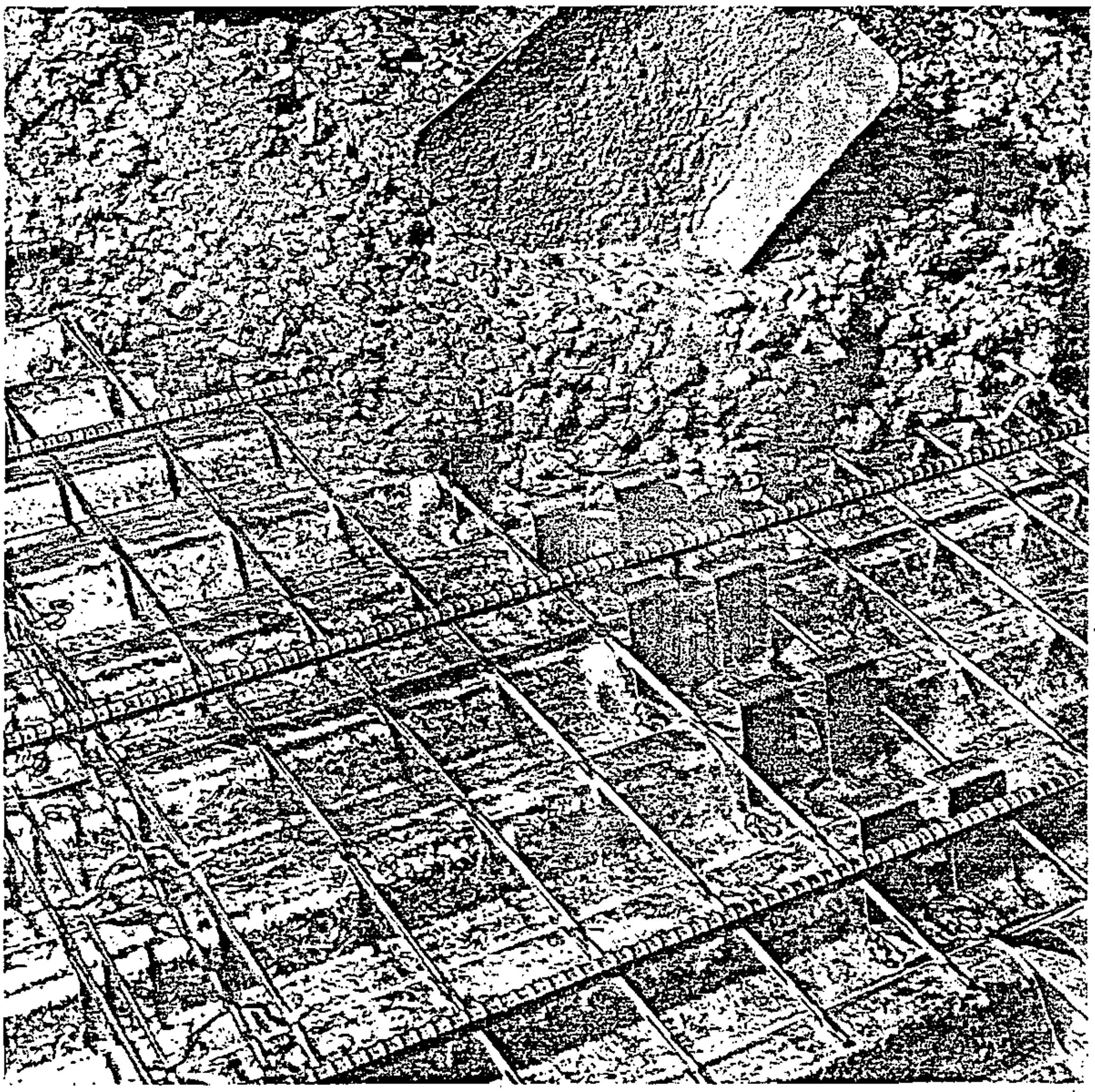
$$f = 11{,}200\ \omega$$
$$= 11{,}200\ (⅜)$$
$$= 4200\ \text{lbs/in.}$$



length of weld at each turn of spiral

$$L = \frac{q}{f}$$
$$= \frac{(17.8\ \text{kips})}{(4200\ \text{lbs/in.})}$$

$= 3.18''$ or *1⅝″ on each side* *OK*



Application of one type of proprietary shear connector for composite construction, providing equivalent strength with less steel tonnage. Connectors welded to beams makes concrete slab integral with supporting member.

Lightweight stud welders permit shear connectors to be attached to girder flanges at high speed. Studs are the most popular form of attachment for anchoring concrete floor slab to the steel girders, permitting steel and concrete to act together for greater strength and rigidity.