Shear Connections for Composite Construction—Bridges

1. BASIC REQUIREMENTS

Concrete roadway decks may be attached to the top flanges of steel girders or beams by the use of suitable shear connectors. These connectors allow the slabs 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-section and weight can be reduced. Since the concrete already serves as part of the floor, 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.

Erection procedures influence the design of the composite beam. If the girder or beam has proper temporary support during construction, its design can be based on the dead loads plus live loads being carried by the composite section after the concrete has attained 75% of its 28-day strength.

If the girder is not shored, then the steel alone must be designed to support the entire dead load during the curing period, and the composite section designed for any live, impact, and additional dead loads. This usually requires greater steel cross-section than is required for the composite design using temporary shoring. However, in bridge construction this savings in steel usually cannot offset the high shoring costs for the long spans involved. As a result, most bridges are designed without shoring.

In the negative moment regions at the supports of continuous beams, the concrete slab would be stressed in tension and cannot be considered effective in the design. Some bridge designers assume the reinforcing

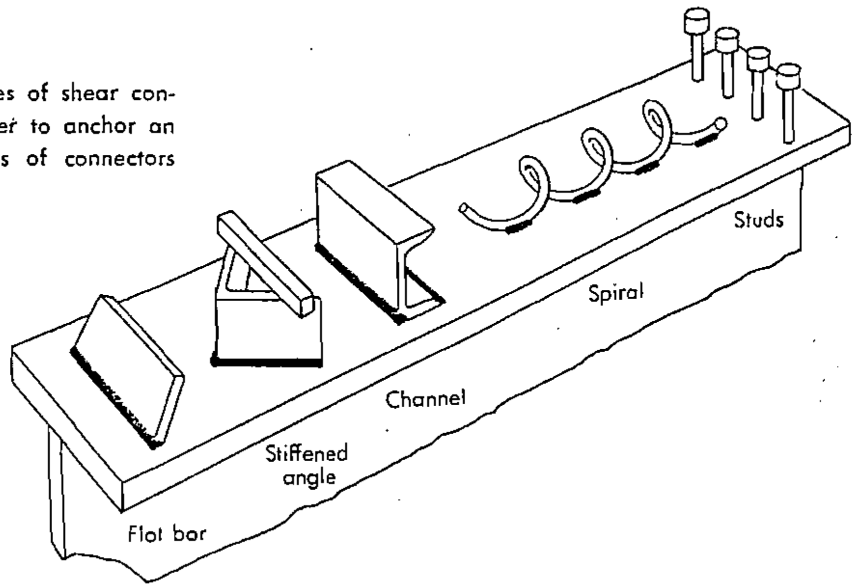


FIG. 1 Representation of five common types of shear connectors welded to top flange of steel girder to anchor an overlayer of concrete. Only short portions of connectors are sketched.

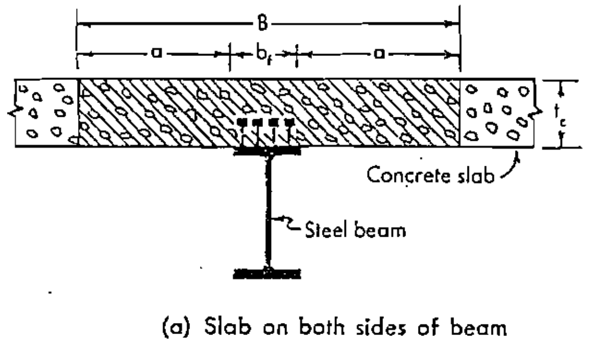


FIGURE 2

steel in this area to be effective in tension when proper shear attachments are continued throughout the area. This approach slightly reduces the beam's cross-sectional area.

2. DESIGN OF CONNECTORS

Shear connectors should have at least 1″ of concrete cover in all directions. They should be designed for only the portion of the load carried by the composite section.

horizontal shear

$$\boxed{V_h = \frac{V_c\ m}{I_c}} \qquad \text{(1)}$$

where:

V_h = horizontal shear of steel flange, at junction of slab and beam, lbs/linear in.

V_c = total external shear acting on composite section after concrete has attained 75% of its 28-day strength, lbs

m = statical moment of transformed concrete area about neutral axis of composite section, or the statical moment of the area of reinforcement embedded in slab for negative moment, $in.^3$

I_c = moment of inertia of transformed composite section

transformed area

In order to get the transformed area of the concrete deck, 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 (AASHO 1.9.3).

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 concrete.

From this transformed section, the various section properties may be determined:

m = statical moment = $A_c\ d_c$ of concrete about neutral axis of composite section

I_c = moment of inertia of transformed composite section, $in.^4$

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

The moment of inertia of the transformed concrete section (I_c) may be read directly from Table 1, the section modulus (S) from Table 2, and the coefficient value of m/I_c for horizontal shear (V_h) from Table 3. Tables 1, 2 and 3 are from "Composite Construction in Steel and Concrete" by Viest, Fountain and Singleton; McGraw-Hill.

where:

n = E_s/E_c = 10, the modular ratio

B = effective slab width

t = slab thickness

design load (working value) for one shear connector

$$\boxed{q = \frac{Q}{F.S.}} \qquad \text{(2)}$$

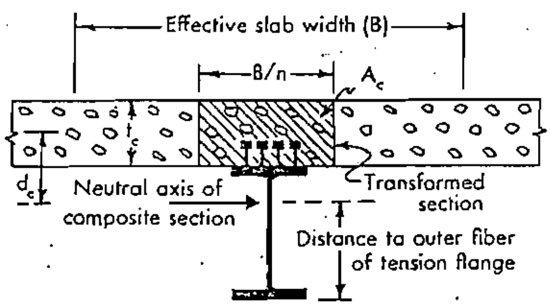


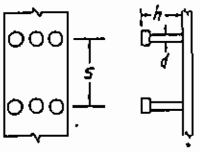
FIGURE 3

where:

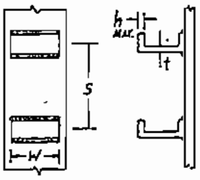
Q = useful capacity of one shear connector, beyond which the connector permits an appreciable slip between concrete slab and steel beam, lbs

F.S. = factor of safety

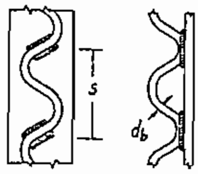
useful capacity of one shear connector



$$\boxed{\begin{array}{l} Q = 330\, d^2 \sqrt{f'_c} \\ \text{when } h/d \geq 4.2 \end{array}} \quad \ldots\ldots(3)$$



$$\boxed{\begin{array}{l} Q = 80\, h\, d\, \sqrt{f'_c} \\ \text{when } h/d < 4.2 \end{array}} \quad \ldots\ldots(4)$$



$$\boxed{Q = 3840\, d\, \sqrt[4]{f'_c}} \quad \ldots\ldots(5)$$

$$\boxed{Q = 180 \left(h + \frac{t}{2}\right) w \sqrt{f'_c}} \quad \ldots\ldots\ldots(6)$$

Note: f'_c = 28-day compressive strength of concrete

For most conditions, the useful capacity (Q) of the shear connector may be read directly from Table 4, 5, or 6 which make it unnecessary to work the above formulas.

factor of safety

The factor of safety to be used in computing the allowable design load for one shear connector, is obtained from the following formula*:

$$\boxed{F.S. = \frac{2.7(1 + C_{mc} + C_{ml}\, C_s) - (C_{mc} + C_{ml}) + C_v}{1 + C_v}} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots(7)$$

* AASHO (1.95) now allows as an alternate, a factor safety of 4 in lieu of calculating it with the above formula.

TABLE 1—Moment of Inertia, Transformed Composite Section

Modular ratio n = 10, b = effective slab width, t = slab thickness

Steel beam		Moment of inertia I_c of composite beams, in.^{4}							
		b = 5 ft			b = 6 ft			b = 7 ft	
Shape	I_s, in.^{4}	t = 6 in.	t = 7 in.	t = 8 in.	t = 6 in.	t = 7 in.	t = 8 in.	t = 7 in.	t = 8 in.
36 WF 300	20,200	32,062	34,056	36,085	33,948	35,820	35,017	37,380	39,715
36 WF 280	18,819	30,237	32,147	34,056	31,743	33,813	35,903	35,285	37,494
36 WF 260	17,234	28,265	30,083	31,926	29,082	31,646	33,624	33,019	35,104
36 WF 245	16,092	26,816	28,561	30,328	28,164	30,043	31,932	31,338	33,323
36 WF 230	14,988	25,403	27,076	28,766	26,683	28,477	30,279	29,696	31,583
36 WF 194	12,103	22,172	23,713	25,259	23,328	24,903	26,594	26,036	27,732
36 WF 182	11,282	21,026	22,496	23,967	22,116	23,668	25,215	24,670	26,274
36 WF 170	10,470	19,880	21,270	22,663	20,896	22,363	23,823	23,292	24,802
36 WF 160	9,739	18,845	20,172	21,500	19,805	21,198	22,586	22,000	23,498
36 WF 150	9,012	17,800	19,060	20,319	18,699	20,017	21,330	20,823	22,174
33 WF 220	12,312	21,334	22,806	24,298	22,425	24,001	25,591	25,038	26,703
33 WF 200	11,048	19,646	21,019	22,409	20,647	22,108	23,582	23,047	24,564
33 WF 141	7,442	15,002	16,099	17,200	15,754	16,899	18,047	17,572	18,753
33 WF 130	6,699	13,896	14,919	16,046	14,584	15,648	16,715	16,257	17,353
30 WF 124	5,347	11,376	12,255	13,145	11,930	12,855	13,781	13,355	14,309
30 WF 116	4,919	10,704	11,535	12,376	11,226	12,090	12,963	12,550	13,449
30 WF 108	4,461	9,963	10,763	11,553	10,463	11,271	12,090	11,692	12,533
27 WF 102	3,604	8,187	8,858	9,541	8,578	9,271	9,981	9,614	10,345
27 WF 94	3,267	7,612	8,234	8,871	7,964	8,608	9,269	8,916	9,597
24 WF 100	2,987	6,739	7,316	7,914	7,056	7,657	8,281	7,940	8,585
24 WF 94	2,683	6,379	6,936	7,512	6,681	7,260	7,861	7,527	8,148
24 WF 84	2,364	5,791	6,298	6,822	6,054	6,579	7,126	6,811	7,377
24 WF 76	2,096	5,292	5,757	6,239	5,524	6,005	6,508	6,210	6,730
21 WF 73	1,600	4,202	4,603	5,026	4,390	4,807	5,250	4,975	5,435
21 WF 68	1,478	3,935	4,334	4,733	4,127	4,521	4,939	4,675	5,111
21 WF 62	1,327	3,640	3,990	4,361	3,793	4,157	4,545	4,294	4,700
18 WF 60	984	2,834	3,137	3,464	2,957	3,274	3,619	3,389	3,751
18 WF 55	890	2,622	2,905	3,211	2,732	3,029	3,353	3,133	3,474
18 WF 50	801	2,412	2,674	2,959	2,510	2,786	3,088	2,879	3,199
16 WF 50	655	2,056	2,300	2,567	2,144	2,401	2,680	2,486	2,780
16 WF 45	583	1,876	2,100	2,349	1,953	2,190	2,456	2,268	2,551
16 WF 40	516	1,697	1,903	2,133	1,764	1,983	2,230	2,053	2,316
16 WF 36	446	1,532	1,723	1,937	1,592	1,795	2,027	1,859	2,107
14 WF 34	339	1,230	1,401	1,597	1,281	1,465	1,677	1,522	1,751
14 WF 30	290	1,096	1,253	1,435	1,141	1,311	1,510	1,363	1,579

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TABLE 2—Section Modulus, Bottom Flange of I Beam

Modular ratio n = 10, b = effective slab width, t = slab thickness

Steel beam			Section modulus S_{cb} of composite beam, in.^{3}							
			b = 5 ft			b = 6 ft			b = 7 ft	
Shape	S_s, in.^{3}	S_s/A_s, in.	t = 6 in.	t = 7 in.	t = 8 in.	t = 6 in.	t = 7 in.	t = 8 in.	t = 7 in.	t = 8 in.
36 WF 300	1,105.1	12.5	1,300.0	1,340.3	1,375.2	1,325.8	1,361.5	1,397.7	1,379.0	1,416.9
36 WF 280	1,031.2	12.5	1,223.2	1,255.2	1,288.7	1,241.9	1,275.5	1,309.9	1,292.0	1,327.2
36 WF 260	951.1	12.4	1,130.1	1,166.9	1,198.4	1,153.1	1,185.2	1,217.4	1,200.3	1,233.5
36 WF 245	892.5	12.4	1,070.9	1,100.2	1,130.0	1,087.0	1,117.3	1,147.8	1,130.9	1,162.7
36 WF 230	835.5	12.3	1,007.7	1,035.4	1,063.8	1,022.7	1,051.2	1,080.6	1,064.4	1,094.0
36 WF 194	663.6	11.6	838.3	863.9	889.4	851.7	878.1	904.3	889.5	916.5
36 WF 182	621.2	11.6	788.5	812.7	836.8	801.0	825.8	850.7	836.3	861.7
36 WF 170	579.1	11.6	739.0	761.5	784.2	750.3	773.3	796.8	783.2	806.8
36 WF 160	541.0	11.5	695.4	716.8	738.6	706.1	728.2	750.4	737.0	759.5
36 WF 150	502.9	11.4	651.5	672.1	692.5	661.4	682.2	703.3	690.6	711.8
33 WF 220	740.6	11.4	902.5	929.3	956.6	916.1	943.8	971.9	955.6	984.3
33 WF 200	669.6	11.4	821.7	846.5	871.6	833.9	859.2	885.2	869.7	896.2
33 WF 141	446.8	10.8	581.9	600.9	620.3	590.5	609.9	629.7	617.2	637.4
33 WF 130	404.8	10.0	533.8	551.7	573.5	541.8	559.9	578.4	566.4	585.5
30 WF 124	354.6	9.7	472.8	489.6	507.1	479.7	497.1	515.0	503.0	521.3
30 WF 116	327.9	9.6	441.6	457.6	474.2	448.0	464.5	481.4	469.9	487.3
30 WF 108	299.2	9.4	408.8	423.9	439.4	414.7	430.2	446.1	435.3	451.6
27 WF 102	266.3	8.9	363.1	377.4	392.3	368.3	383.0	398.3	387.5	403.2
27 WF 94	242.6	8.8	334.4	347.7	361.6	339.0	352.8	367.1	356.8	371.7
24 WF 100	245.9	8.5	332.8	346.6	361.0	337.3	351.4	366.0	355.6	371.2
24 WF 94	220.9	8.0	308.0	321.4	335.5	312.5	326.3	341.0	330.3	345.6
24 WF 84	196.3	7.9	276.3	288.5	301.3	280.1	292.8	306.2	296.3	310.2
24 WF 76	175.4	7.8	249.9	261.2	273.2	253.3	265.0	277.5	268.3	281.3
21 WF 73	150.7	7.0	219.4	230.5	242.6	222.6	234.3	246.7	237.4	250.5
21 WF 68	139.9	7.0	205.1	215.7	227.1	208.1	219.1	231.1	222.0	234.6
21 WF 62	126.4	6.9	187.1	197.0	207.7	189.8	200.1	211.4	202.7	214.6
18 WF 60	107.8	6.1	164.2	174.2	185.0	166.8	177.2	188.8	179.9	192.3
18 WF 55	98.2	6.1	150.9	160.2	170.5	153.2	163.1	174.2	165.7	177.4
18 WF 50	89.0	6.1	137.7	146.5	156.1	139.6	149.1	159.6	151.5	162.7
16 WF 50	80.7	5.5	128.3	137.4	147.4	130.5	140.2	151.1	142.7	154.3
16 WF 45	72.4	5.5	116.2	124.6	134.2	118.1	127.3	137.5	129.6	140.6
16 WF 40	64.4	5.5	104.2	112.1	120.9	106.0	114.5	124.2	116.8	127.2
16 WF 36	56.3	5.3	93.6	101.0	109.5	95.3	103.4	112.6	105.6	115.6
14 WF 34	48.5	4.9	82.9	90.5	99.2	84.7	93.0	102.5	95.3	105.6
14 WF 30	41.6	4.7	73.5	80.6	88.7	75.2	82.0	92.0	85.2	95.2

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where:

$$C_{mc} = \frac{M_{Dc}}{M_L}$$

$$C_{mt} = \frac{M_{Ds}}{M_L}$$

$$C_s = \frac{S_c}{S_s}$$

$$C_v = \frac{V_D}{V_L}$$

where:

M_{Dc} = max. moment caused by dead loads acting on composite section

M_{Ds} = max. moment caused by dead loads acting on steel beam alone

M_L = max. moment caused by live load

S_c = section modulus of composite beam for extreme tension fibers

S_s = section modulus of steel beam for extreme tension fibers

TABLE 3—Coefficient m/I_c for Horizontal Shear

Modular ratio $n = 10$, b = effective slab width, t = slab thickness

Steel beam	Coefficient $\frac{m}{I_c}$ of composite beam, 1/in.							
Shape	b = 5 ft			b = 6 ft			b = 7 ft	
	t = 6 in.	t = 7 in.	t = 8 in.	t = 6 in.	t = 7 in.	t = 8 in.	t = 7 in.	t = 8 in.
36 WF 300	0.0170	0.0183	0.0193	0.0184	0.0194	0.0205	0.0206	0.0215
36 WF 280	0.0176	0.0188	0.0198	0.0190	0.0201	0.0210	0.0211	0.0220
36 WF 260	0.0183	0.0195	0.0204	0.0196	0.0208	0.0216	0.0218	0.0226
36 WF 245	0.0188	0.0200	0.0209	0.0202	0.0213	0.0221	0.0222	0.0230
36 WF 230	0.0194	0.0205	0.0214	0.0207	0.0218	0.0226	0.0227	0.0234
36 WF 194	0.0212	0.0222	0.0230	0.0224	0.0233	0.0240	0.0242	0.0248
36 WF 182	0.0217	0.0227	0.0234	0.0229	0.0238	0.0244	0.0246	0.0251
36 WF 170	0.0222	0.0232	0.0238	0.0234	0.0242	0.0248	0.0250	0.0255
36 WF 160	0.0227	0.0237	0.0243	0.0239	0.0247	0.0252	0.0255	0.0259
36 WF 150	0.0233	0.0242	0.0248	0.0244	0.0252	0.0257	0.0260	0.0263
33 WF 220	0.0213	0.0225	0.0234	0.0227	0.0238	0.0246	0.0248	0.0255
33 WF 200	0.0222	0.0233	0.0242	0.0235	0.0246	0.0253	0.0255	0.0262
33 WF 141	0.0253	0.0261	0.0267	0.0264	0.0271	0.0276	0.0279	0.0283
33 WF 130	0.0261	0.0269	0.0273	0.0272	0.0279	0.0283	0.0286	0.0289
30 WF 124	0.0288	0.0296	0.0301	0.0299	0.0306	0.0309	0.0313	0.0315
30 WF 116	0.0295	0.0302	0.0306	0.0306	0.0311	0.0314	0.0318	0.0320
30 WF 108	0.0303	0.0309	0.0313	0.0313	0.0318	0.0320	0.0325	0.0325
27 WF 102	0.0331	0.0337	0.0340	0.0341	0.0346	0.0347	0.0352	0.0351
27 WF 94	0.0338	0.0343	0.0345	0.0346	0.0352	0.0352	0.0357	0.0356
24 WF 100	0.0361	0.0367	0.0369	0.0372	0.0376	0.0376	0.0383	0.0381
24 WF 94	0.0371	0.0376	0.0377	0.0382	0.0385	0.0384	0.0391	0.0388
24 WF 84	0.0381	0.0384	0.0383	0.0391	0.0392	0.0390	0.0397	0.0393
24 WF 76	0.0390	0.0392	0.0390	0.0399	0.0399	0.0395	0.0403	0.0396
21 WF 73	0.0436	0.0435	0.0432	0.0445	0.0442	0.0435	0.0447	0.0438
21 WF 68	0.0441	0.0440	0.0435	0.0450	0.0446	0.0438	0.0449	0.0439
21 WF 62	0.0449	0.0446	0.0439	0.0457	0.0451	0.0442	0.0453	0.0442
18 WF 60	0.0507	0.0501	0.0489	0.0514	0.0503	0.0490	0.0505	0.0489
18 WF 55	0.0514	0.0505	0.0492	0.0520	0.0508	0.0493	0.0509	0.0491
18 WF 50	0.0519	0.0509	0.0495	0.0525	0.0510	0.0494	0.0511	0.0492
16 WF 50	0.0560	0.0550	0.0531	0.0568	0.0552	0.0530	0.0551	0.0525
16 WF 45	0.0570	0.0554	0.0532	0.0573	0.0554	0.0528	0.0550	0.0522
16 WF 40	0.0575	0.0556	0.0532	0.0578	0.0554	0.0527	0.0550	0.0519
16 WF 36	0.0583	0.0561	0.0535	0.0583	0.0556	0.0526	0.0550	0.0517
14 WF 34	0.0635	0.0605	0.0571	0.0634	0.0599	0.0560	0.0591	0.0545
14 WF 30	0.0641	0.0607	0.0565	0.0636	0.0596	0.0553	0.0587	0.0541

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TABLE 4—Useful Capacity, Q, of One Stud Connector, lbs. (h/d > 4.2)

Stud dia., d, in.	CONCRETE STRENGTH, f'_c psi			
	2,500	3,000	3,500	4,000
5/8	6,500	7,100	7,600	8,200
3/4	9,300	10,200	11,000	11,700
7/8	12,600	13,800	15,000	16,000

Note: A factor of safety must be applied to the above useful capacity, Q, to arrive at the working value, q.

TABLE 5—Useful Capacity, Q, Per Turn of Spiral Connector

Spiral wire dia, in.	CONCRETE STRENGTH, f'_c, psi.			
	2500	3000	3500	4000
1/2	13,580	14,210	14,770	15,270
5/8	16,970	17,760	18,460	19,000
3/4	20,360	21,310	22,150	22,900
7/8	23,760	24,870	25,840	26,720

Note: A factor of safety must be applied to the above useful capacity, Q, to arrive at the working value, q.

V_D = vertical shear caused by dead load acting on composite section

V_L = vertical shear caused by live load

spacing of sheer connectors

$$\boxed{s = \frac{n\,q}{V_h}} \qquad (8)$$

where:

s = spacing or pitch of shear connectors in the direction of beam axis, in.

n = number of shear connectors at one transverse beam cross-section

q = capacity of one connector, lbs

V_h = horizontal shear to be transferred, lbs

The spacing of shear connectors should not exceed 24″.

3. DESIGN OF CONNECTING WELDS

Welds joining shear connectors to beams should be designed to the allowable fatigue force (f_w), for the range (K) of shear stress and the working load (q) of the connector. See Table 7.

where:

$$K = \frac{\text{min. shear (V)}}{\text{max. shear (V)}}$$

ω = leg size of fillet weld, in.

f_w = allowable force on fillet weld, lbs/lin. in.

TABLE 6—Useful Capacity Q, Per 1 In. of Channel (Lbs)

Channel type and size	FLANGE THICKNESS, IN. Max, h	FLANGE THICKNESS, IN. Min.	Web thickness t, in.	CONCRETE STRENGTH f'_c (psi) 2,500	3,000	3,500	4,000
American Standard:							
3-in:							
4.1-lb	0.377	0.170	0.170	4,160	4,560	4,920	5,260
5.0-lb	0.377	0.170	0.258	4,560	4,980	5,380	5,750
6.0-lb	0.377	0.170	0.356	4,990	5,460	5,910	6,310
4-in:							
5.4-lb	0.413	0.180	0.180	4,520	4,960	5,360	5,710
7.25-lb	0.413	0.180	0.320	5,160	5,640	6,100	6,510
5-in.:							
6.7-lb	0.450	0.190	0.190	4,910	5,370	5,810	6,200
9.0-lb	0.450	0.190	0.325	5,510	6,030	6,520	6,960
6-in.:							
8.2-lb	0.487	0.200	0.200	5,870	5,780	6,260	6,680
10.5-lb	0.487	0.200	0.314	5,790	6,350	6,860	7,330
13.0-lb	0.487	0.200	0.437	6,350	6,950	7,510	8,020
7-in.:							
9.8-lb	0.523	0.210	0.210	5,650	6,180	6,690	7,140
12.25-lb	0.523	0.210	0.314	6,110	6,700	7,240	7,740
14.75-lb	0.523	0.210	0.419	6,590	7,210	7,810	8,330
Car building:							
3-in.:							
7.1-lb	0.390	0.313	0.312	4,910	5,370	5,810	6,210
9.0-lb	0.390	0.313	0.500	5,760	6,310	6,810	7,280
4-in.:							
13.8-lb	0.531	0.469	0.500	7,250	7,690	8,310	7,870
Shipbuilding							
6-in.:							
12.0-lb	0.413	0.337	0.313	5,130	5,610	6,060	6,480
15.1-lb	0.521	0.429	0.313	6,070	6,680	7,210	7,810
15.3-lb	0.440	0.330	0.340	5,490	6,010	6,500	6,940
16.3-lb	0.521	0.429	0.375	6,380	6,980	7,550	8,060
18.0-lb	0.530	0.420	0.375	6,460	7,070	7,640	8,160
7-in.:							
17.6-lb	0.521	0.429	0.375	6,380	6,980	7,550	8,060
19.1-lb	0.554	0.446	0.350	6,560	7,190	7,760	8,300
22.7-lb	0.554	0.446	0.500	7,240	7,920	8,560	9,150

Note: A factor of safety must be applied to the above useful capacity (Q) to arrive at the working value (q).

4. COMPOSITE CONSTRUCTION SUMMARY

1.a. Without shoring, dead load carried by steel and live load is carried by the composite section.

b. With shoring, dead loads and live loads are carried by the composite section.

2. With shoring, there is reduction in steel but added cost of shoring.

3. Type and cost of shear connector must be balanced against installation cost.

4. In taking advantage of composite action, effort should be made to reduce weight and depth of steel beams.

5. Savings in steel from use of bottom cover plates must be evaluated against additional fabricating cost.

6. Composite construction has the advantage of greater rigidity.

7. Studs may also serve as "high chairs" to support steel reinforcing mesh for the concrete.

8. Future connector designs may be more efficient and reduce the number required.

TABLE 7—Allowable Fatigue Force on Fillet Welds

Cycles	Allowable force on weld, lbs/linear in.	Formula No. in AWS Bridge Spec.
N = 2,000,000	$f_\omega = \dfrac{5090\omega}{1 - \frac{k}{2}}$	No. 10
N = 600,000	$f_\omega = \dfrac{7070\omega}{1 - \frac{k}{2}}$	No. 14
N = 100,000	$f_\omega = \dfrac{8484\omega}{1 - \frac{k}{2}}$	No. 18

Note: But not to exceed 8800 ω lb/linear in

Problem 1

To determine the working load (q), spacing (s), and weld length (L_w) for each of several types of shear connectors, for a typical composite section.

In the building field, the total horizontal shear force to be carried by the shear connectors is based on the total bending force in either the concrete or the steel section resulting from the maximum positive moment on the beam. It is assumed this force will be transferred from the concrete slab into the steel beam by the connectors along a distance from the point of maximum positive moment out to the end of the beam, for simply supported beams; or from the point of maximum positive moment out to the point of contra-flexure, for continuous beams.

In the bridge field, this shear transfer is based on the vertical shear applied to the beam. In most cases this value will vary along the beam's length. For this reason, more than one section may have to be checked when the size and number of shear connectors are determined.

This example considers just one point of application, the section near the pier supports, and assumes certain conditions:

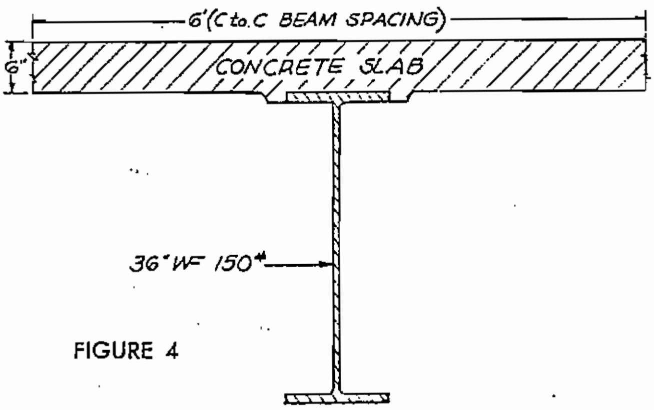


FIGURE 4

$f_c' = 3000$ psi (concrete)

$\frac{m}{I_c} = .0244$/in. (See Table 3)

F.S. $= 3.81$

$V_{max} = 49.6$ kips

$V_{min} = 5.06$ kips

calculating for horizontal shear

$$V_h = \frac{V_c\ m}{I_c}$$

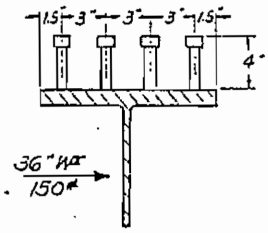
$$= (49.6)(.0244)$$

$$= 1.21 \text{ kips/in.}$$

Stud Connectors

Use ¾" dia. x 4" studs. From Table 4, Q = 10.2 kips/stud.

working load



$$q = \frac{Q}{F.S.}$$

$$= \frac{(10.2)}{(3.81)}$$

$$= 2.68 \text{ kips/stud}$$

spacing of connectors (use 4 studs per transverse section)

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

$$= \frac{(4)(2.68)}{(1.21)}$$

$$= 8.85'' \text{ or } \underline{\text{use } 8\tfrac{1}{2}''}$$

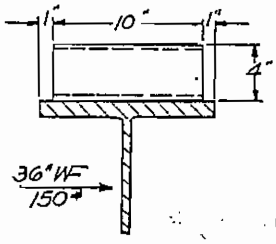
weld length

Complete contact surface of stud is joined to beam. No calculation of weld length is necessary.

Channel Connectors

Use a 4" 5.4-lb channel of 10" length. From Table 6, Q = 49.6 kips/channel.

working load



$$q = \frac{F.S.}{Q}$$

$$= \frac{(49.6)}{(3.81)}$$

$$\doteq 13.0 \text{ kips/channel}$$

spacing of connectors

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

$$= \frac{(1)(13.0)}{(1.21)}$$

$$= 10.75'' \text{ or } \underline{\text{use } 10\tfrac{1}{2}''}$$

allowable force on weld

Assume fillet leg size of $\omega = \frac{3}{16}''$ and N = 600,000 cycles:

$$K = \frac{V_{min}}{V_{max}}$$

$$= \frac{(+5.06 \text{ kips})}{(+46.6 \text{ kips})}$$

$$= +0.102$$

$$f_w = \frac{7070\ \omega}{1 - \frac{K}{2}} \qquad \text{(See Table 7)}$$

$$= \frac{7070(3/16)}{1 - (.051)}$$

$$= 1.4 \text{ kips/in. of weld}$$

required weld length

$$L_w = \frac{q}{f_w}$$

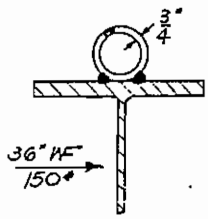
$$= \frac{(13.0)}{(1.4)}$$

$$= 9.3'' < 20'' \text{ actually used} \qquad \underline{OK}$$

This indicates most channels are overwelded.

Spiral Connectors

Use ¾″ dia rod. From Table 5, Q = 21.31 kips/turn.



working load

$$q = \frac{Q}{F.S.}$$

$$= \frac{(21.31)}{(3.81)}$$

$$= 5.6 \text{ kips/turn}$$

pitch

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

$$= \frac{(1)(5.6)}{(1.21)}$$

$$= 4.61'' \text{ or } \underline{\text{use } 4\tfrac{1}{2}''\text{/turn}}$$

force on weld

Assume fillet leg size of $\omega = 3/8''$ and N ≐ 600,000 cycles:

$$K = \frac{V_{min}}{V_{max}}$$

$$= \frac{(+5.06 \text{ kips})}{(+49.6 \text{ kips})}$$

$$= +.102$$

$$f_w = \frac{7070\ \omega}{1 - \frac{K}{2}} \qquad \text{(From Table 7)}$$

$$= \frac{7070(3/8)}{1 - (.051)}$$

$$= 2.8 \text{ kips/in. of weld}$$

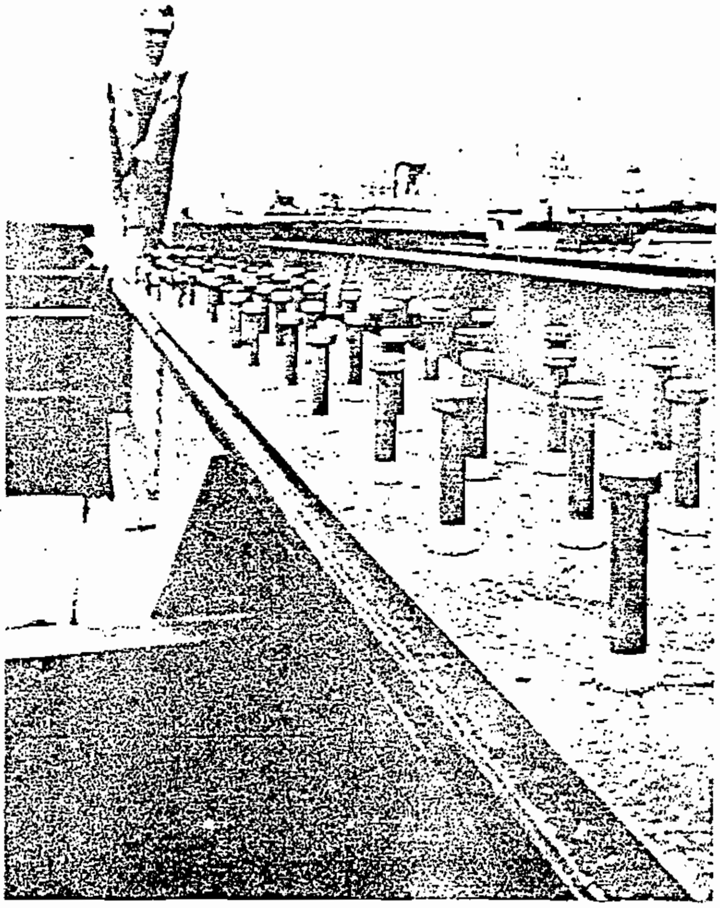
length of weld

$$L_w = \frac{q}{f_w}$$

$$= \frac{(5.6)}{(2.8)}$$

$$= 2.0'' \text{ or } \underline{1'' \text{ each side in contact area}}$$

Studs are widely used in both building and bridge work as shear connectors for composite construction. Quickly attached by efficient arc-welding equipment, studs serve to anchor the concrete slab to the steel beams. The composite beam provides high strength at lower cost.



Typical scenes of modern bridge work featuring composite construction. Prior to pouring the concrete deck, studs are attached to girder flanges by specialized arc-welding equipment. Connectors allow the concrete slab to act with the steel.