Web Framing Angles

1. GENERAL REQUIREMENTS

Web framing angles are usually shop welded to the web of the beam, extending about ½″ beyond the end of the beam, and field welded to the supporting member.

Erection bolts are usually placed near the bottom of the angle, so they do not restrain the beam end from rotating under load. For deeper girders, the erection bolts may be placed near the top of the angle for better stability during erection. If there is concern about any restraining action, the bolts may be removed after field welding.

The thickness of the framing angles must be limited to that which will allow sufficient flexibility, otherwise the connection would restrain the end of the simply supported beam from rotating and thus would load up in end moment. AISC has a table of typical framing angle connections. It lists 3″ and 4″ angles of 5/16″ to 7/16″ thickness. When thicker angles are used the leg against the supporting member must be increased in about the same proportion as the thickness in order to maintain the same order of flexibility.

The analysis of this type of connection is divided into two parts: a) the field weld of the angle to the supporting member and b) the shop weld of the angle to the web of the beam.

2. ANALYSIS OF FIELD WELDS TO THE SUPPORT

When the reaction (R) is applied, the framing angles tend to twist or rotate, pressing against each other at the top, and swinging away from each other at the bottom.

It is assumed the two angles bear against each other for a vertical distance equal to 1/6 of their length. The remaining 5/6 of the length is resisted by the connecting welds. It is assumed also that these forces on the welds increase linearly, reaching a maximum (f_h) at the bottom of the connection, Figure 1.

horizontal force on weld

Applied moment from load = Resisting moment of weld

$$\frac{R}{2} L_h = \frac{2}{3} P L_v$$

where L_h = leg length of angle

$$\text{or } P = \frac{.75 R L_h}{L_v}$$

From force triangle, find—

$$P = \tfrac{1}{2} (f_h)(\tfrac{5}{6} L_v)$$

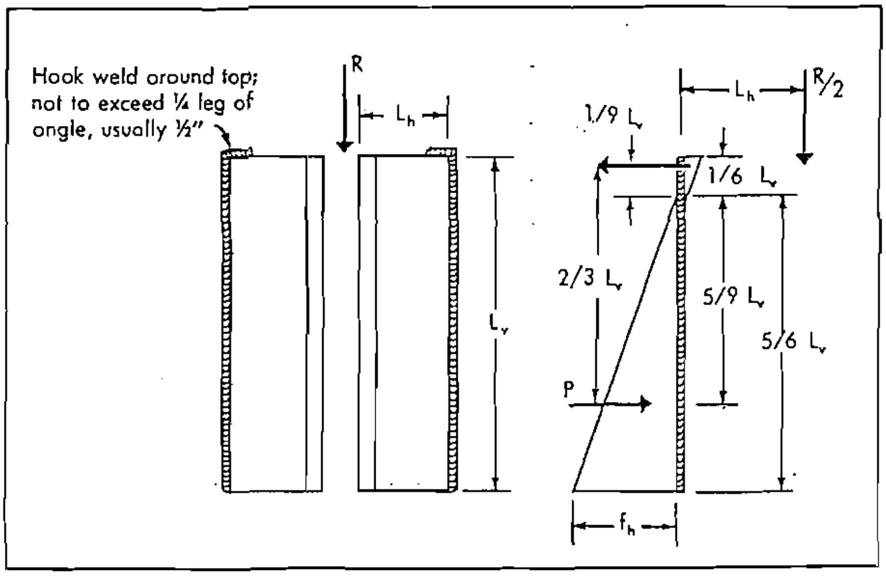
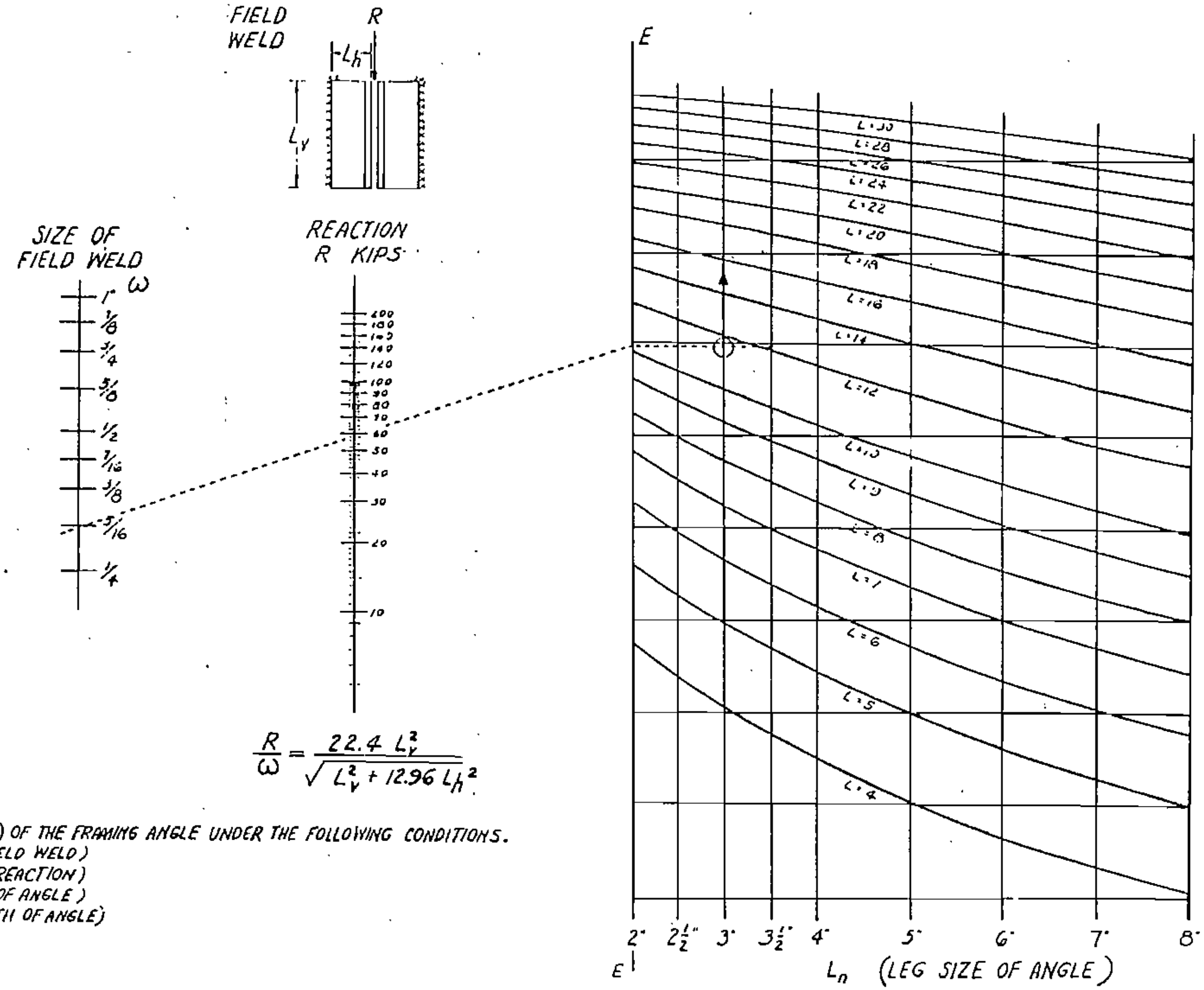


FIGURE 1

FIGURE 2—Framing Angles and Size of Field Welds
For A36 Steel & E70 Welds

NOMOGRAPH NO. 6



$$\frac{R}{\omega} = \frac{22.4\, L_v^2}{\sqrt{L_v^2 + 12.96\, L_h^2}}$$

PROBLEM: FIND THE LENGTH (L_v) OF THE FRAMING ANGLE UNDER THE FOLLOWING CONDITIONS.
$\omega = 5/16$" (SIZE OF FIELD WELD)
$R = 58$ KIPS (END REACTION)
$L_n = 3$" (LEG SIZE OF ANGLE)
READ $L_v = 12$" (LENGTH OF ANGLE)

From these two equations, determine—

$$f_h = \frac{9\ R\ L_h}{5\ L_v^2}$$

vertical force on weld

$$f_v = \frac{R}{2L_v}$$

resultant force on weld

$$f_r = \sqrt{f_h^2 + f_v^2} = \sqrt{\left(\frac{9\ RL_h}{5\ L_v^2}\right)^2 + \left(\frac{R}{2L_v}\right)^2}$$

or:

$$\boxed{f_r = \frac{R}{2L_v^2}\sqrt{L_v^2 + 12.96\ L_h^2}} \qquad \ldots\ldots\ldots\ldots(1)$$

leg size of fillet weld

$$\omega = \frac{\text{actual force on weld}}{\text{allowable force}} \quad \text{and:}$$

A7, A373 Steel; E60 Welds	A36 Steel; E70 Welds	
$\frac{R}{\omega} = \frac{19.2\ L_v^2}{\sqrt{L_v^2 + 12.96\ L_h^2}}$	$\frac{R}{\omega} = \frac{22.4\ L_v^2}{\sqrt{L_v^2 + 12.96\ L_h^2}}$	.(2)

Be sure the supporting plate is thick enough for this resulting weld size (ω).

The two vertical welds connecting framing angles to supporting member should be "hooked" around the top of the angles for a distance of about twice the leg size of the weld, or about ½". (Original tests indicated that a distance not to exceed ¼ of the angle's leg length helped the carrying capacity of the connection.)

Nomograph No. 6 (Fig. 2) may be used for the field welding. This nomograph is for A36 steel and E70 welds. In the chart on the right-hand side, from the point of intersection of the angle's leg size (L_h) and the length of the angle (L_v), draw a horizontal line to the vertical axis E-E. From this point, draw a line through the reaction (R) to the left-hand axis. Read the leg size (ω) of the field weld on this axis.

Table 1, for A36 steel and E70 welds, gives values of R/ω in terms of leg size of angle (L_h) and length of angle (L_v).

AISC, Sect 1.17.5 specifies that the leg size of a fillet weld used in calculating its length (L_v) should not cause the web of the supporting member to be overstressed in shear.

For a single pair of framing angles on just one side of the supporting web, assume the leg size of the fillet weld not to exceed 1.3 t_w.

For two pairs of framing angles, one on each side of the supporting web, assume the leg size of the fillet weld not to exceed ⅔ t_w.

These factors of (⅔) and (1.3 = 2 x ⅔) may be adjusted for the exact type of steel used by referring to Table 2.

TABLE 1—Values of R/ω
For Field Weld of Framing Angle to Support For A36 Steel & E70 Welds

$$\frac{R}{\omega} = \frac{\text{Reaction, kips}}{\text{Leg size of fillet weld}} \qquad \boxed{\frac{R}{\omega} = \frac{22.4\ L_v^2}{\sqrt{L_v^2 + 12.96\ L_h^2}}}$$

Length of framing angle (L_v)	Leg of Angle (L_h) 2"	3"	4"	5"	6"	7"	8"
4"	43	30	22	19	16	14	12
5"	63	46	37	29	25	21	19
6"	84	64	51	41	35	30	27
7"	108	84	67	55	47	41	36
8"	131	105	85	71	61	53	47
9"	156	127	105	88	76	66	59
10"	180	150	125	106	92	81	72
12"	229	197	169	146	128	113	101
14"	277	246	215	189	167	149	122
16"	321	294	262	234	209	188	170
18"	372	343	312	281	254	219	209
20"	416	392	360	328	300	274	251
22"	466	440	409	385	347	319	294
24"	514	487	457	426	394	365	338
26"	560	536	505	476	443	413	384
28"	607	583	547	523	491	461	425
30"	654	630	603	571	540	509	480

3. ANALYSIS OF SHOP WELDS TO THE BEAM

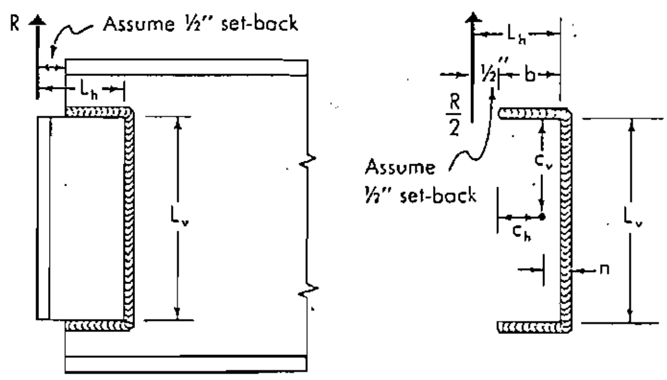


FIGURE 3

In Figure 3, analysis of the shop weld shows—

$$n = \frac{b^2}{2b + L_v}$$

$$c_h = L_h - n - \tfrac{1}{2}''$$

$$c_v = \frac{L_v}{2}$$

$$b = L_h - \tfrac{1}{2}''$$

$$J_w = \frac{(2b + L_v)^3}{12} - \frac{b^2(b + L_v)^2}{2b + L_v}$$

twisting (horizontal)

$$\boxed{f_h = \frac{T\ c_v}{J_w} = \frac{R}{2}\frac{(L_h - n)c_v}{J_w}} \quad \ldots\ldots\ldots\ldots\ldots (3)$$

twisting (vertical)

$$\boxed{f_{v1} = \frac{T\ c_h}{J_w} = \frac{R}{2}\frac{(L_h - n)c_h}{J_w}} \quad \ldots\ldots\ldots\ldots\ldots (4)$$

shear (vertical)

$$\boxed{f_{v2} = \frac{R/2}{2b + L_v}} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (5)$$

resultant force on outer end of connecting weld

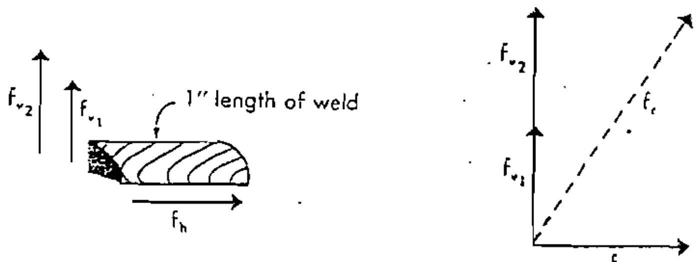


FIGURE 4

$$\boxed{f_r = \sqrt{f_h^2 + (f_{v1} + f_{v2})^2}} \quad \ldots\ldots\ldots\ldots\ldots (6)$$

leg size of fillet weld

$$\omega = \frac{\text{actual force on welds}}{\text{allowable force}}$$

A7, A373 Steel; E60 Welds	A36 Steel; E70 Welds
$\omega = \frac{f_r}{9600}$	$\omega = \frac{f_r}{11{,}200}$

(7)

Unfortunately there is no way to simplify these

TABLE 2—Maximum Leg Size to Use in Calculating Vertical Length of Weld

FOR VARIOUS COMBINATIONS OF WELD METALS AND STEEL

Given these conditions:

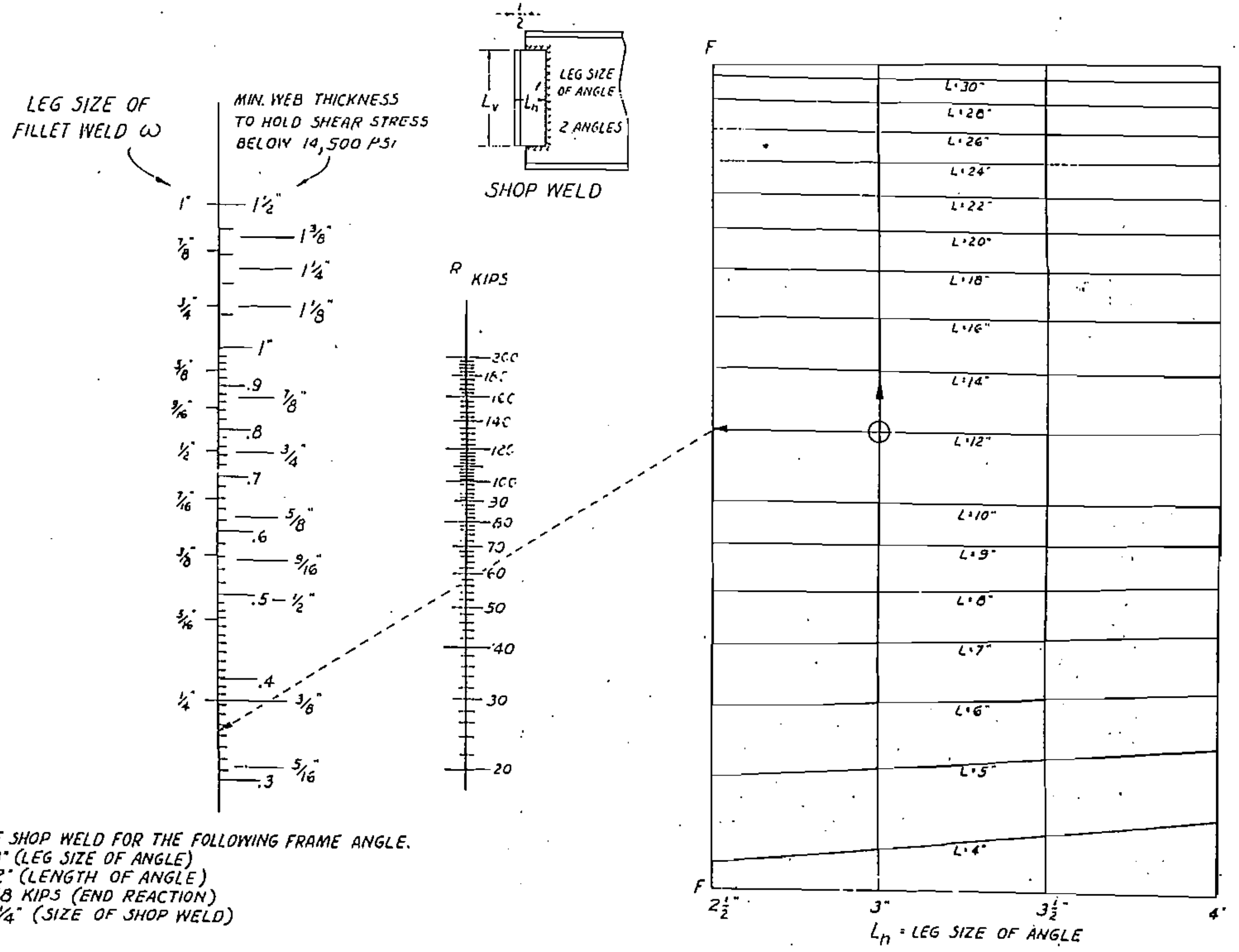
Steel	A7 A373	A36	A242, A441		
thickness	-		Over 1½" To 4"	Over ¾" To 1½"	¾" or less
σ_y	33,000	36,000	42,000	46,000	50,000
τ	13,000	14,500	17,000	18,500	20,000
weld	E60 or SAW-1	E70 or SAW-2	E70 or SAW-2	E70 or SAW-2	E70 or SAW-2
f	9,600 ω	11,200 ω	11,200 ω	11,200 ω	11,200 ω
$\omega/t \leqq$	.667	.648	.759	.826	.893

Then: Maximum leg size of fillet weld to use in calculating vertical length

leg size ω	Web thickness (t_w) over —				
¼"	.375	.386	.329	.325	.280
5⁄16"	.468	.482	.412	.378	.350
⅜"	.562	.579	.494	.454	.420
7⁄16"	.656	.676	.576	.529	.490
½"	.750	.772	.659	.605	.560
9⁄16"	.844	.868	.741	.681	.630
⅝"	.937	.964	.824	.756	.700

FIGURE 5—Size of Shop Weld of Framing Angles
For A36 Steel & E70 Welds

NOMOGRAPH NO. 7



PROBLEM: FIND SIZE OF SHOP WELD FOR THE FOLLOWING FRAME ANGLE.
L_h = 3" (LEG SIZE OF ANGLE)
L = 12" (LENGTH OF ANGLE)
R = 58 KIPS (END REACTION)
READ ω = 1/4" (SIZE OF SHOP WELD)

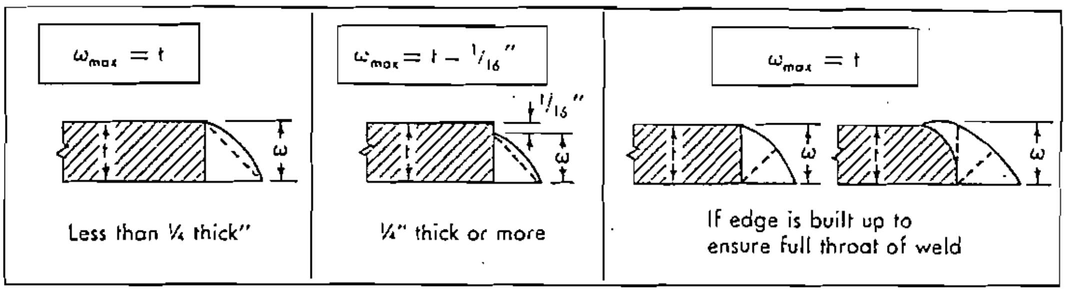


FIGURE 6

formulas into one workable formula. It is necessary to work out each step until the final result is obtained.

The leg size of this shop weld may be determined quickly by means of Nomograph No. 7 (Fig. 5), for A36 steel and E70 welds. In the chart on the right-hand side, from the point of intersection of the angle's horizontal leg length (L_h) and its vertical length (L_v) draw a horizontal line to the vertical axis F-F. From this point, draw a line through the reaction (R) to the left-hand axis. Read the leg size (ω) of the shop weld along the left-hand scale of this axis.

If the nomograph is used from left to right to establish an angle size, be sure that the leg size of the fillet weld does not exceed a value which would overstress the web of the beam in shear (AISC Sec 1.17.5) by producing too short a length of connecting weld (L_v).

The following limits apply to the fillet weld leg size (ω) relative to the thickness of the beam web (as used in calculating the vertical length of connecting weld):

A7, A373 Steel and E60 Weld

$$(\tau = 13{,}000 \text{ psi}) \quad (f_w = 9600\ \omega \text{ lbs/in.})$$

$$\omega \leqq t_w \frac{13{,}000}{2 \times 9600} \leqq .676\ t_w \text{ or } < \tfrac{2}{3}\ t_w$$

A36 Steel and E70 Weld

$$(\tau = 14{,}500 \text{ psi}) \quad (f_w = 11{,}200\ \omega \text{ lbs/in.})$$

$$\omega \leqq t_w \frac{14{,}500}{2 \times 11{,}200} \leqq .648\ t_w \text{ or } < \tfrac{2}{3}\ t_w$$

or $\boxed{\omega \leqq \tfrac{2}{3}\ t_w}$ ....(8)

However, the actual leg size of the fillet weld used may exceed this value.

Table 2 reflects the limiting value of $\omega = \tfrac{2}{3}\ t_w$. AISC holds to this limit for shop weld of the angle to the beam (AISC Manual, pages 4-25).

Notice the left-hand axis of Nomograph No. 7 also gives the minimum web thickness of the beam in order to hold its shear stress (τ) within 14,500 psi. Just be sure the actual web thickness of the supported beam is equal to or exceeds this value found just opposite the resulting leg size of the weld.

Some engineers feel this limiting shear value (A36 steel, τ = 14,500 psi) is to insure that the web of the beam does not buckle, and that a higher allowable value might be used here, perhaps ¾ of the allowable tensile strength. In this case the maximum leg size of the weld would be held to ¾ of the web thickness.

$\boxed{\omega = \tfrac{3}{4}\ t_w}$ ....(9)

AISC (Sec 1.17.5) specifies the maximum leg size of fillet weld relative to angle plate thickness to be as shown in Figure 6.

Table 3 will give values of R/ω in terms of leg size of angle (L_h) and length of angle (L_v). Table 3 is for direct use with A36 steel, and E70 welds.

TABLE 3—Values of R/ω

For Shop Weld of Framing Angle To Beam Web For A36 Steel & E70 Welds

$$\frac{R}{\omega} = \frac{\text{Reaction, kips}}{\text{Leg size of fillet weld}}$$

Length of framing angle (L_v)	Leg of Angle (L_h)			
	2½"	3"	3½"	4"
4"	75.8	78.6	81.6	86.1
6"	116.7	119.3	120.2	122.8
8"	160.3	160.3	162.2	163.2
10"	205.3	204.6	204.7	205.2
12"	251.7	250.7	249.7	249.5
14"	298.5	295.5	296.3	294.4
16"	347.0	345.2	343.0	340.5
18"	395.0	393.0	390.7	389.0
20"	443.0	439.0	436.5	434.5
22"	490.5	487.0	484.0	481.5
24"	537.0	535.5	533.5	530.0
26"	586.0	583.5	580.5	576.7
28"	635.0	631.0	628.5	625.0
30"	681.0	680.0	676.5	673.5

As indicated by Figure 3 and the related weld analysis, the fillet welds connecting angle to beam web should be hooked around the ends of the angle, top and bottom, for the distance (b) to the end of the beam web. They should not be continued around the end of the web, Figure 7.

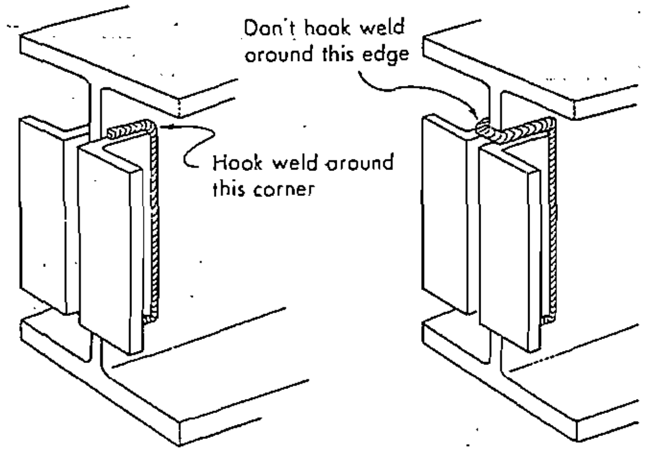


FIGURE 7

Problem 1

To design a web framing angle connection to support a 20″ 85# I beam, having an end reaction of R = 58 kips. Use A36 steel and E70 welds.

See Figure 8.

Field Weld of Framing Angle to Column

Nomograph No. 6 shows that for a ⅜″ fillet weld (ω), a reaction (R) of 58 kips and an angle with a leg (L_h) of 3″, its length (L_v) should be 10½″. However, for a $\frac{5}{16}$″ fillet weld (ω) the angle length (L_v) would only have to be increased to 12″.

Shop Weld of Framing Angle to Beam Web

Nomograph No. 7 shows that for a reaction (R) of 58 kips, an angle leg (L_h) of 3″ and length (L_v) of 12″, a ¼″ fillet weld (ω) would be required. Hence use 3″ × 3″ × ⅜″ framing angles, 12″ long, $\frac{5}{16}$″ field weld to column and ¼″ shop weld to beam web.

4. STANDARD WEB FRAMING ANGLE CONNECTIONS

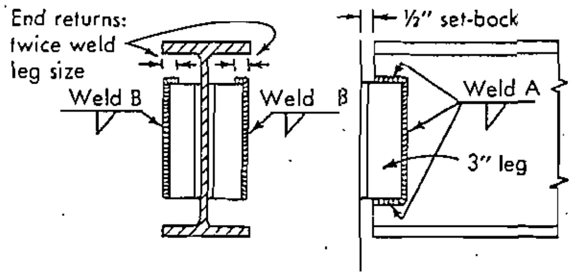


FIGURE 9

Table 4 gives the AISC allowable loads (kips) on web framing angle connections, using A36, A242 and A441 steels and E70 welds. The table gives the capacity and size of (Shop) Weld A connecting the framing angle to the beam web, and of (Field) Weld B connecting the framing angle to the beam support.

Problem 2

To select a web framing angle connection for a 16″ B 26# beam (0.25″ web thickness and T = 14″) of A441 steel, with end reaction of R = 35 kips. Use E70 welds. Allowable shear is 20 ksi.

This beam would take an angle with length L_v = 10″ or 12″. In Table 4, the (Shop) Weld A capacity

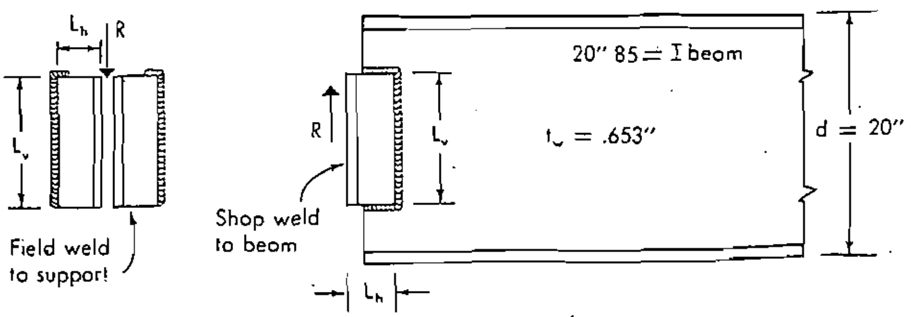
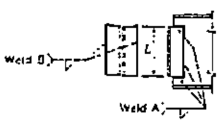


FIGURE 8

TABLE 4—Standard Web Framing Angle Connections

From American Institute of Steel Construction



FRAMED BEAM CONNECTIONS

Welded—E60XX electrodes

TABLE V

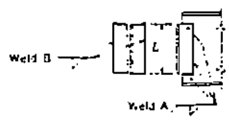
Weld A Capacity Kips	Weld A [c]Size In.	Weld B [d]Capacity Kips	Weld B [c]Size In.	L In.	Angle Size (ASTM A36)	[a,b]Minimum Web Thickness for Welds A: A36 F_v=14.5	A242 and A441 F_v=18.5	A242 and A441 F_v=20.0
195	5/16	210	3/8	32	4×3×7/16	.41	.32	.30
156	1/4	175	5/16	32	4×3×3/8	.33	.26	.24
117	3/16	140	1/4	32	4×3×5/16	.25	.19	.18
182	5/16	195	3/8	30	4×3×7/16	.41	.32	.30
146	1/4	162	5/16	30	4×3×3/8	.33	.26	.24
109	3/16	130	1/4	30	4×3×5/16	.25	.19	.18
169	5/16	179	3/8	28	4×3×7/16	.41	.32	.30
135	1/4	149	5/16	28	4×3×3/8	.33	.26	.24
101	3/16	120	1/4	28	4×3×5/16	.25	.19	.18
156	5/16	164	3/8	26	4×3×7/16	.41	.32	.30
125	1/4	136	5/16	26	4×3×3/8	.33	.26	.24
93.8	3/16	109	1/4	26	4×3×5/16	.25	.19	.18
143	5/16	148	3/8	24	4×3×7/16	.41	.32	.30
115	1/4	124	5/16	24	4×3×3/8	.33	.26	.24
86.1	3/16	98.8	1/4	24	4×3×5/16	.25	.19	.18
131	5/16	133	3/8	22	4×3×7/16	.41	.32	.30
104	1/4	110	5/16	22	4×3×3/8	.33	.26	.24
78.4	3/16	88.4	1/4	22	4×3×5/16	.25	.19	.18
119	5/16	117	3/8	20	4×3×7/16	.41	.32	.30
94.2	1/4	97.4	5/16	20	4×3×3/8	.33	.26	.24
70.7	3/16	77.9	1/4	20	4×3×5/16	.25	.19	.18
105	5/16	101	3/8	18	4×3×7/16	.41	.32	.30
84.0	1/4	84.4	5/16	18	4×3×3/8	.33	.26	.24
63.0	3/16	67.5	1/4	18	4×3×5/16	.25	.19	.18
92.2	5/16	95.5	3/8	16	3×3×7/16	.41	.32	.30
73.8	1/4	79.6	5/16	16	3×3×3/8	.33	.26	.24
55.3	3/16	63.6	1/4	16	3×3×5/16	.25	.19	.18
79.6	5/16	79.8	3/8	14	3×3×7/16	.41	.32	.30
63.6	1/4	66.5	5/16	14	3×3×3/8	.33	.26	.24
47.7	3/16	53.2	1/4	14	3×3×5/16	.25	.19	.18
67.1	5/16	64.2	3/8	12	3×3×7/16	.41	.32	.30
53.7	1/4	53.5	5/16	12	3×3×3/8	.33	.26	.24
40.3	3/16	42.8	1/4	12	3×3×5/16	.25	.19	.18
54.9	5/16	48.9	3/8	10	3×3×7/16	.41	.32	.30
43.9	1/4	40.8	5/16	10	3×3×3/8	.33	.26	.24
32.9	3/16	32.6	1/4	10	3×3×5/16	.25	.19	.18
48.9	5/16	41.5	3/8	9	3×3×7/16	.41	.32	.30
39.1	1/4	34.6	5/16	9	3×3×3/8	.33	.26	.24
29.3	3/16	27.6	1/4	9	3×3×5/16	.25	.19	.18
42.9	5/16	34.1	3/8	8	3×3×7/16	.41	.32	.30
34.1	1/4	28.6	5/16	8	3×3×3/8	.33	.26	.24
25.8	3/16	22.8	1/4	8	3×3×5/16	.25	.19	.18
37.3	5/16	27.4	3/8	7	3×3×7/16	.41	.32	.30
29.8	1/4	22.9	5/16	7	3×3×3/8	.33	.26	.24
22.4	3/16	18.1	1/4	7	3×3×5/16	.25	.19	.18
31.7	5/16	21.0	3/8	6	3×3×7/16	.41	.32	.30
25.3	1/4	17.5	5/16	6	3×3×3/8	.33	.26	.24
19.0	3/16	14.0	1/4	6	3×3×5/16	.25	.19	.18
26.3	5/16	15.1	3/8	5	3×3×7/16	.41	.32	.30
21.0	1/4	12.6	5/16	5	3×3×3/8	.33	.26	.24
15.8	3/16	10.1	1/4	5	3×3×5/16	.25	.19	.18
21.1	5/16	10.0	3/8	4	3×3×7/16	.41	.32	.30
16.9	1/4	8.4	5/16	4	3×3×3/8	.33	.26	.24
12.7	3/16	6.7	1/4	4	3×3×5/16	.25	.19	.18

[a]When a beam web is less than the minimum, multiply the connection capacity furnished by welds A by the ratio of the actual thickness to the tabulated minimum thickness. Thus, if 5/16" weld A, with a connection capacity of 54.9 kips and a 10" long angle, is being considered for a beam of web thickness .270", ASTM A36, the connection capacity must be multiplied by .270/.41, giving 36.2 kips.

[b]When beam material is ASTM A7 or A373, with F_v = 13.0 ksi, minimum web thicknesses to develop 5/16", 1/4" and 3/16" welds A are .46", .37" and .28" respectively.

[c]Should the thickness of material to which connection angles are welded exceed the limits set by AISC Specification, Sect. 1.17.4, for weld sizes specified, increase the weld size as required, but not to exceed the angle thickness.

[d]For welds on outstanding legs, connection capacity may be limited by the shear capacity of the supporting member as stipulated by AISC Specification, Sect. 1.17.5. See examples (d) and (e), pages 4-26, 4-27.



FRAMED BEAM CONNECTIONS

Welded—E70XX electrodes

TABLE VI

Weld A Capacity Kips	Weld A [b]Size In.	Weld B [c]Capacity Kips	Weld B [b]Size In.	L In.	Angle Size (ASTM A36)	[a]Minimum Web Thickness for Welds A: A36 F_v=14.5	A242 and A441 F_v=18.5	A242 and A441 F_v=20.0
227	5/16	245	3/8	32	4×3×7/16	.48	.38	.35
182	1/4	204	5/16	32	4×3×3/8	.39	.30	.28
136	3/16	163	1/4	32	4×3×5/16	.29	.23	.21
212	5/16	227	3/8	30	4×3×7/16	.48	.38	.35
170	1/4	189	5/16	30	4×3×3/8	.39	.30	.28
127	3/16	151	1/4	30	4×3×5/16	.29	.23	.21
197	5/16	209	3/8	28	4×3×7/16	.48	.38	.35
158	1/4	174	5/16	28	4×3×3/8	.39	.30	.28
118	3/16	139	1/4	28	4×3×5/16	.29	.23	.21
182	5/16	191	3/8	26	4×3×7/16	.48	.38	.35
146	1/4	159	5/16	26	4×3×3/8	.39	.30	.28
109	3/16	127	1/4	26	4×3×5/16	.29	.23	.21
167	5/16	173	3/8	24	4×3×7/16	.48	.38	.35
134	1/4	144	5/16	24	4×3×3/8	.39	.30	.28
100	3/16	115	1/4	24	4×3×5/16	.29	.23	.21
152	5/16	155	3/8	22	4×3×7/16	.48	.38	.35
122	1/4	129	5/16	22	4×3×3/8	.39	.30	.28
91.4	3/16	103	1/4	22	4×3×5/16	.29	.23	.21
137	5/16	136	3/8	20	4×3×7/16	.48	.38	.35
110	1/4	114	5/16	20	4×3×3/8	.39	.30	.28
82.4	3/16	90.9	1/4	20	4×3×5/16	.29	.23	.21
122	5/16	118	3/8	18	4×3×7/16	.48	.38	.35
98.0	1/4	98.4	5/16	18	4×3×3/8	.39	.30	.28
73.5	3/16	78.7	1/4	18	4×3×5/16	.29	.23	.21
108	5/16	111	3/8	16	3×3×7/16	.48	.38	.35
86.1	1/4	92.9	5/16	16	3×3×3/8	.39	.30	.28
64.6	3/16	74.3	1/4	16	3×3×5/16	.29	.23	.21
92.9	5/16	93.1	3/8	14	3×3×7/16	.48	.38	.35
74.3	1/4	77.6	5/16	14	3×3×3/8	.39	.30	.28
55.7	3/16	62.1	1/4	14	3×3×5/16	.29	.23	.21
78.3	5/16	74.9	3/8	12	3×3×7/16	.48	.38	.35
62.6	1/4	62.5	5/16	12	3×3×3/8	.39	.30	.28
47.0	3/16	50.0	1/4	12	3×3×5/16	.29	.23	.21
64.0	5/16	57.1	3/8	10	3×3×7/16	.48	.38	.35
51.2	1/4	47.6	5/16	10	3×3×3/8	.39	.30	.28
38.4	3/16	38.0	1/4	10	3×3×5/16	.29	.23	.21
57.1	5/16	48.4	3/8	9	3×3×7/16	.48	.38	.35
45.5	1/4	40.4	5/16	9	3×3×3/8	.39	.30	.28
34.2	3/16	32.3	1/4	9	3×3×5/16	.29	.23	.21
50.2	5/16	40.0	3/8	8	3×3×7/16	.48	.38	.35
40.1	1/4	33.4	5/16	8	3×3×3/8	.39	.30	.28
30.1	3/16	26.7	1/4	8	3×3×5/16	.29	.23	.21
43.5	5/16	32.0	3/8	7	3×3×7/16	.48	.38	.35
34.8	1/4	26.7	5/16	7	3×3×3/8	.39	.30	.28
26.1	3/16	21.3	1/4	7	3×3×5/16	.29	.23	.21
37.0	5/16	24.5	3/8	6	3×3×7/16	.48	.38	.35
29.6	1/4	20.4	5/16	6	3×3×3/8	.39	.30	.28
22.2	3/16	16.3	1/4	6	3×3×5/16	.29	.23	.21
30.7	5/16	17.6	3/8	5	3×3×7/16	.48	.38	.35
24.5	1/4	14.7	5/16	5	3×3×3/8	.39	.30	.28
18.4	3/16	11.8	1/4	5	3×3×5/16	.29	.23	.21
24.6	5/16	11.6	3/8	4	3×3×7/16	.48	.38	.35
19.7	1/4	9.7	5/16	4	3×3×3/8	.39	.30	.28
14.8	3/16	7.8	1/4	4	3×3×5/16	.29	.23	.21

[a]When a beam web is less than the minimum, multiply the connection capacity furnished by welds A by the ratio of the actual thickness to the tabulated minimum thickness. Thus, if 5/16" weld A, with a connection capacity of 50.2 kips and an 8" long angle, is being considered for a beam of web thickness .305", ASTM A36, the connection capacity must be multiplied by .305/.48, giving 31.9 kips.

[b]Should the thickness of material to which connection angles are welded exceed the limits set by AISC Specification, Sect. 1.17.4, for weld sizes specified, increase the weld size as required, but not to exceed the angle thickness.

[c]For welds on outstanding legs, connection capacity may be limited by the shear capacity of the supporting members as stipulated by AISC Specification, Sect. 1.17.5. See examples (d) and (e), pages 4-26, 4-27.

Note 1: Capacities shown in this table apply only when material welded is ASTM A36, A242 or A441. Use appropriate capacities from Table V when beam or supporting material is ASTM A7 or A373.

of 38.4 kips for a weld size of $\omega = 3/16''$ and angle length of $L_v = 10''$ slightly exceeds the reaction. The corresponding (Field) Weld B, using $\omega = 1/4''$, also is satisfactory. Since the beam's required web thickness is 0.21″ while the actual web thickness is 0.25″, the indicated 3″ x 3″ x $5/16''$ is all right.

If the beam is made of A36 steel, this connection's capacity will be reduced in the ratio of 0.25/0.29 of actual to required web thickness. The resulting capacity of 33.1 kips is less than the reaction. The next larger connection with apparently sufficient capacity shows that (Shop) Weld A's capacity is 47 kips, using same angle section but an angle length of $L_v = 12''$. Applying the multiplier of 0.25/0.29 reduces the capacity of the connection to 40.5 kips, which exceeds the end reaction.

5. SINGLE-PLATE OR TEE CONNECTION ON BEAM WEB

In the previous design of the field weld, connecting a pair of web framing angles to the supporting column or girder, it was assumed that the reaction (R) applied eccentric to each angle, resulted in a tendency for the angles to twist or rotate. In doing so, they would press together at the top and swing away from each other at the bottom, this being resisted by the welds. These forces are in addition to the vertical forces caused by the reaction (R); see Figure 10.

However, in both the single-plate web connection and the Tee-section type, this portion of the connection welded to the column is solid. Thus, there is no tendency for this spreading action which must be resisted by the welds. These vertical field welds to the column would be designed then for just the vertical reaction (R); see Figure 11.

In the shop weld of the single plate to the web of the beam, Figure 12, this double vertical weld would be designed for just the vertical reaction (R). There is not enough eccentricity to consider any bending action.

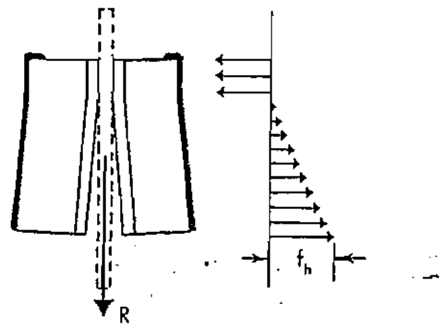


FIG. 10—Double-web framing angle.

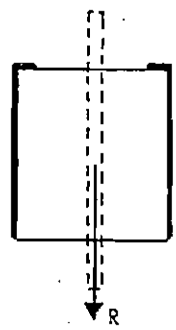


FIG. 11—Single plate or Tee.

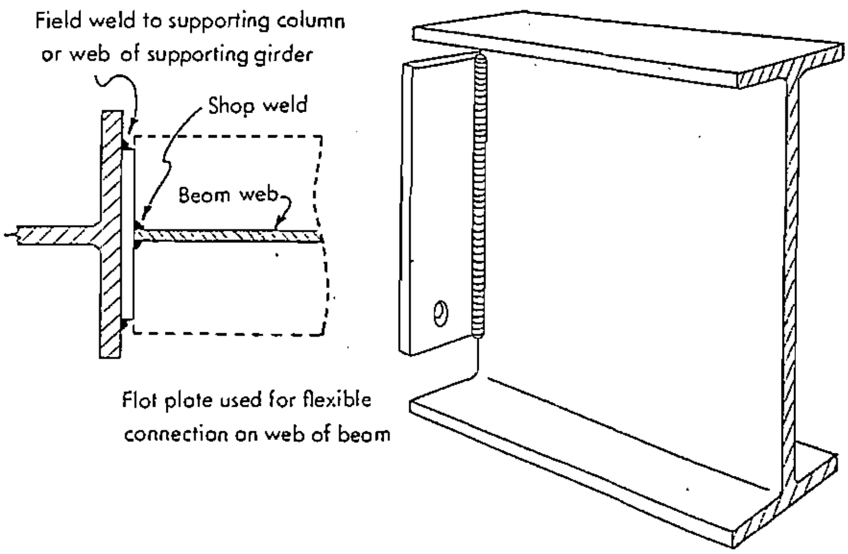


FIG. 12—Flat plate used for flexible connection on web of beam.

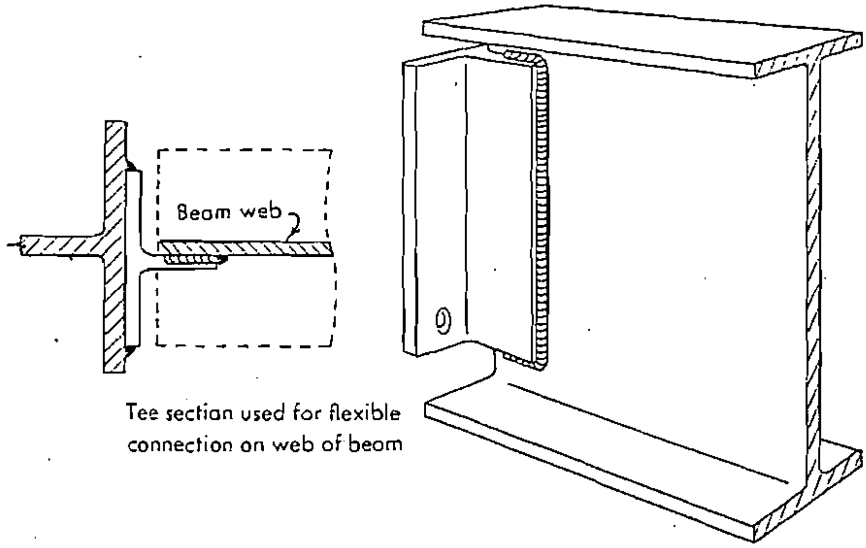


FIG. 13—Tee section used for flexible connection on web of beam.

In the shop weld of the Tee connection to the web of the beam, Figure 13, the size and length of the fillet weld would be determined just as in the case of the double-web framing angles, except there is just a single fillet weld in this case rather than two; so, for a given connection, this would carry just half of the reaction of the corresponding double-angle connection.

6. DIRECTLY-WELDED WEB CONNECTION

To see how this type of connection behaves, consider the following 18″ WF 85# beam, simply supported, 15′ span, with a uniformly distributed load of 139 kips, the same beam and load used in the general discussion on behavior of connections in Sect. 5.1, Topic 6.

If only the web is to be welded to the column, the weld must have sufficient length (L_v) so that the adjacent web of the beam will not be overstressed in shear.

For A373 steel

$$\tau = .40\ \sigma_y = 13{,}000 \text{ psi}$$

$$t_w = .526''$$

$$L_v = \frac{R}{t_w\ \tau}$$

$$= \frac{(69.5^k)}{(.526'')(13 \text{ ksi})}$$

$$= 10.2'', \text{ or } \textit{use } 11''$$

The leg size of this fillet weld must be equal to the web thickness, based upon standard allowables, if it is to match the allowable strength of this web section in shear as well as tension.

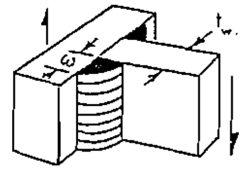


FIGURE 14

fillet weld in shear; parallel load

$$2(9600\omega)L = t_w\ 13{,}000\ L$$

$$\boxed{\omega = ⅔\ t_w}$$

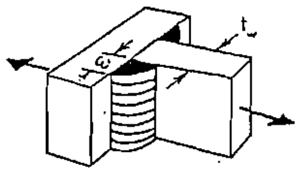


FIGURE 15

fillet weld in tension; transverse load

$$2(9600\omega)L = t_w\ 20{,}000\ L$$

$$* \boxed{\omega = t_w}$$

* Actually, transverse fillet welds are about ⅓ stronger than parallel fillet welds; this can be proved by theory as well as testing. This means for transverse loads, the leg size would be ⅔ of the plate thickness, just as in parallel loads. However, welding codes do not as yet recognize this; and for code work, fillet welds for transverse loads would be made equal to the plate thickness.

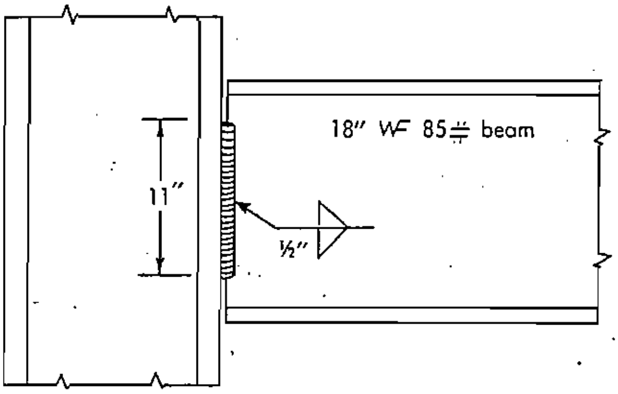


FIGURE 16

If there is a gap between the beam and the column, the leg size of this fillet weld is increased by this amount.

The moment-rotation chart, Figure 17, shows the beam line for this particular beam length and load, and the actual connection curve taken from test data at Lehigh University.

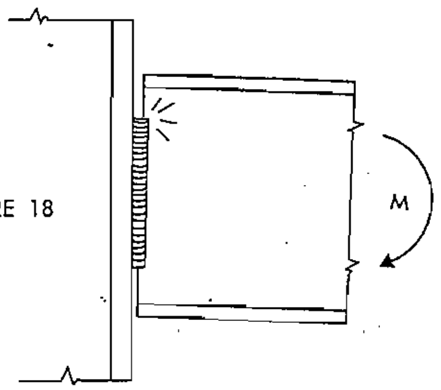
In testing this connection, the beam web showed initial signs of yielding adjacent to the lower ends of the weld at a moment of 360 in.-kips. At a moment of 660 in.-kips, point (a), there were indications that the beam web along the full length of the weld had yielded. At a moment of 870 in.-kips, both welds cracked slightly at the top; this point is marked with an "X" on the curve. With further cracking of the weld and yielding in the beam web, the lower flange of the beam contacted the column, point (b), and this resulted in increased stiffness. The moment built up to a maximum of 1918 in.-kips, and then gradually fell off as the weld continued to tear.

Notice in this particular example, the web would have yielded the full length of the weld at design load.

The weld started to crack when the connection had rotated about .011 radians; this would correspond to a horizontal movement of .06″ at the top portion of the weld. Compare this small amount of movement with that obtained in the top connecting plate example of Figure 4 which had the ability to pull out 1.6″ before failing.

This directly welded web connection (Fig. 18)

FIGURE 18



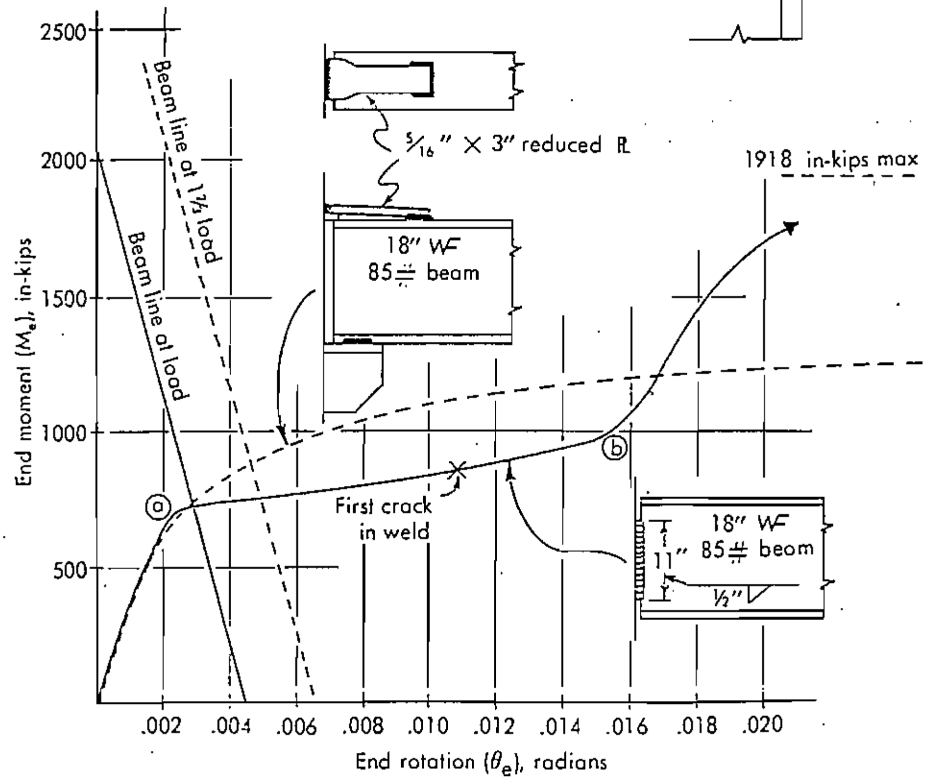


FIGURE 17

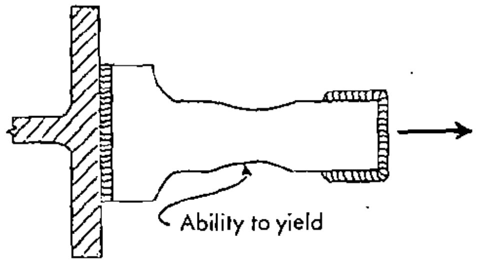


FIGURE 19

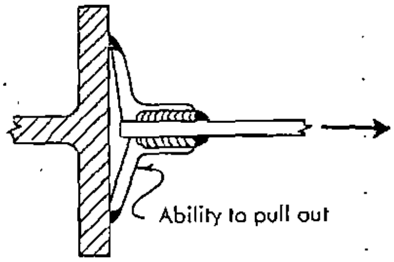


FIGURE 20

is not as dependable as a top connecting plate designed to yield at working load (Fig. 19) or either flexible web framing angles (Fig. 20) or flexible top angle.

Also remember this highly yielded web section, in the case of the directly welded web connection, must still support or carry the vertical reaction (R) of the beam, whereas in the top plate connection, the support of the beam at the bottom seat is still sound no matter what happens to the top plate.

Figure 17 would indicate the directly welded web connection results in an end moment of M_e = 720 in.-kips, or an end restraint of—

$$R = \frac{720 \text{ in.-kips}}{2016 \text{ in.-kips}}$$

$$= 35.8\%$$

This restraint is a little high to be classed as simply supported.

The same top plate connection is shown in dotted lines on Figure 17; it has about the same stiffness, but many times the rotational ability.

The use of side plates, Figure 21, would allow a wide variation in fit-up, but in general they are no better than the directly welded web connection. Unless the plates are as thick as the beam web, the resulting connecting fillet welds will be smaller and will reduce the strength of the connection.

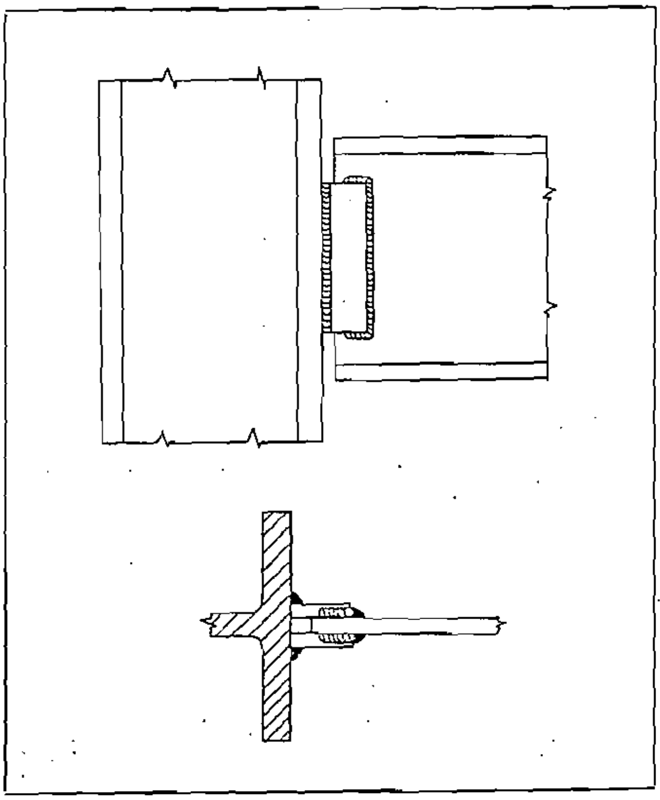


FIGURE 21

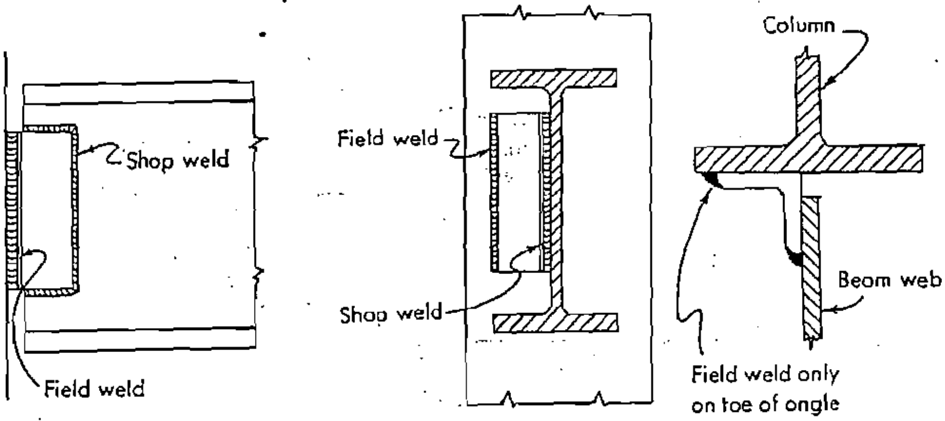


FIGURE 22

In the tests at Lehigh University, the corresponding connection on the 18" WF 85# beam (.526"-thick web) used 5/16" thick side plates with 5/16" fillet welds. They failed at a lower load.

If ½" thick side plates with ½" fillet welds had been used, they undoubtedly would have been as strong as the directly welded web connection.

7. ONE-SIDED WEB CONNECTIONS

A single web framing angle used by itself is not recommended; see Figure 22.

Use of only a single vertical fillet weld to join the angle to the supporting member imposes a greater eccentricity upon the connection. This results in a maximum force on the weld of about 4 times that of the double-angle connection; see Figures 23 and 24.

It might be argued that in the conventional double-angle connection, the field weld is subject only to vertical shear because the stiffness of the angles largely prevents any twisting action on the connection even though the analysis is based upon this twist as shown in Figure 23. However, there is no doubt that the single-angle connection has this twisting action which would greatly decrease its strength.

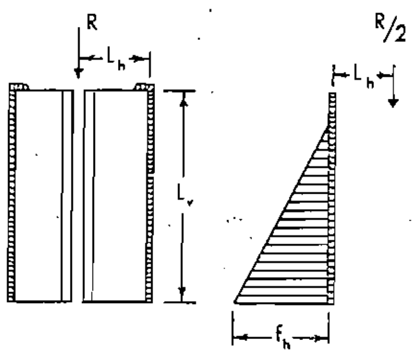


FIGURE 23

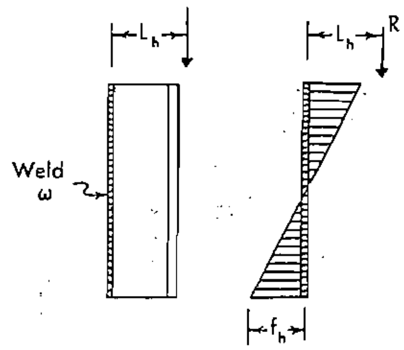
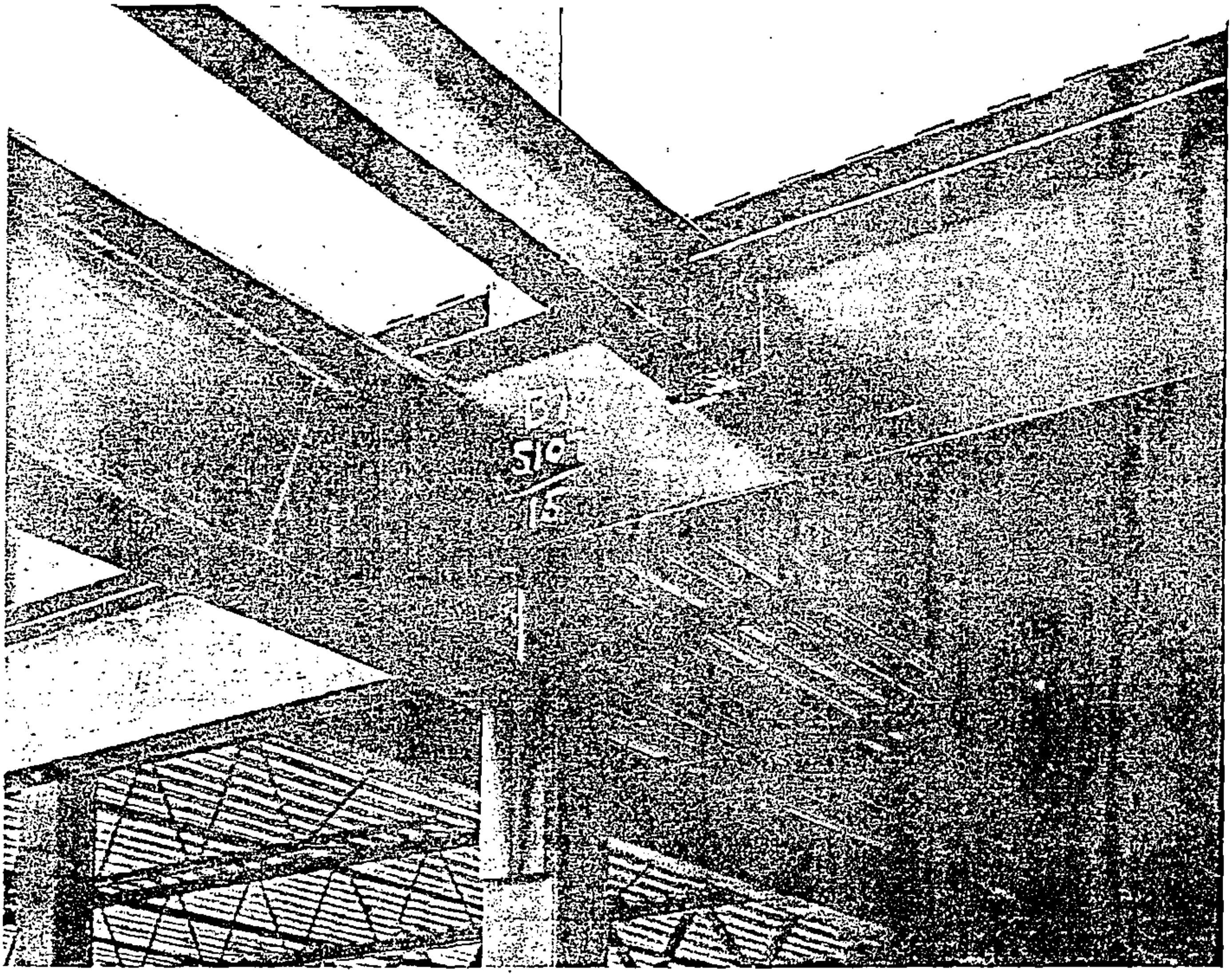


FIGURE 24

Any additional welding on the single angle, such as vertically along its heel or horizontally across the top and bottom edges, would make it rigid and prevent it from moving under load. This would cause the end moment to build up and greatly overstress the connection.

In the original research at Lehigh University on welded connections, this single-angle connection with a single vertical weld was never tested. Single angle connections welded both along the sides and along the ends were tested, but as already mentioned, they did not have enough flexibility, and the end moment built up above the strength of the connection.



Web framing angles are commonly shop welded to the supported beam. To facilitate erection, bolts are used in joining the other member until the web framing angle can be permanently welded to it. The erection bolts can be left in, or removed if there is any concern that they will offer restraint. Note the use of box section column, in this case it being hot rolled square structural tubing.