Welded Plate Girders for Buildings

1. DIMENSIONING THE GIRDER

Plate girders are fabricated for requirements which exceed those of a rolled beam, or a rolled beam with added cover plate. The usual welded plate girder is made of two flange plates fillet welded to a single web plate. Where needed, web stiffeners are attached to one or both sides of the web. Box girders are made of two flange plates fillet welded to two web plates. Internal stiffening of these is accomplished with diaphragm plates.

The flange-area method is used to get an approximate dimension of the girder. This assumes that the flanges will carry all the bending moment and the web will carry all the shear forces.

The required web area is—

$$\boxed{A_w = \frac{V}{\tau}} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (1)$$

where:

V = vertical shear applied to cross-section to be considered

τ = allowable shear stress on web section

The formula for required flange area is derived from properties of the girder:

$$I = 2\, A_f \frac{d^2}{2} + \frac{t_w\, d^3}{12} \text{ or}$$

$$= \frac{A_f\, d^2}{2} + \frac{A_w\, d^2}{12} \qquad \text{since } A_w = t_w\, d$$

For simplicity, this assumes web depth is equal to (d), the distance between the centers of gravity of the two flange plates.

$$S = \frac{I}{d/2} \text{ or}$$

$$= A_f\, d + \frac{A_w\, d}{6}$$

Also,

$$S = \frac{M}{\sigma}$$

Therefore, the required flange area is—

$$\boxed{A_f = \frac{M}{\sigma\, d} - \frac{A_w}{6}} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (2)$$

where:

M = bending moment applied to section

σ = allowable bending stress

d = distance between centers of gravity of flange plates

This method will require some approximate knowledge of what the girder depth should be and some adjustment of the resulting figures before the design is finalized.

Guides to Girder Depth

The previous AISC specification held the depth of girders to a minimum value of 1/24 of the span. The Commentary on the new AISC specifications suggests, as a guide, that the girder depth should not exceed the following:

Floors: σ_y / 800,000 times the span
Roof purlins: σ_y / 1,000,000 times the span

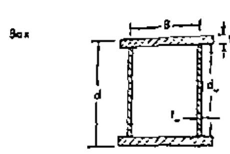
This translates into the Table 1 limiting values of depth-to-length for girders used in floors. These values are for general guidance only.

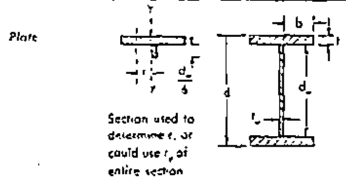
TABLE 1—Suggested Girder Depth Limits (AISC)

AISC Steels			Others	
	σ_y	d/L	σ	d/L
A7, A373	33,000	1/24.2	45,000	1/17.8
A36	36,000	1/22.2	50,000	1/16.0
A441	42,000	1/19.0	55,000	1/14.6
	46,000	1/17.4	60,000	1/13.3
	50,000	1/16.0	65,000	1/12.3
			90,000*	1/8.8
			95,000*	1/8.4
			100,000*	1/8.0

* Quenched & tempered steels: Yield strength at 0.2% offset.

TABLE 2—Allowable Bending Stresses For Plate and Box Girders





Compression elements which are not "compact" but meet the following AISC Sec 1.9 requirements—

$$* \quad \frac{b}{t} \leqq \frac{3{,}000}{\sqrt{\sigma_y}} \qquad (1.9.1)$$

$$* \quad \frac{B}{t} \leqq \frac{8{,}000}{\sqrt{\sigma_y}} \qquad (1.9.2)$$

$$\frac{d_w}{t_w} \leqq \frac{14{,}000{,}000}{\sqrt{\sigma_y(\sigma_y + 16{,}500)}} \qquad (1.10.2)$$

box girder	plate girder
tension (1.5.1.4.3) $\sigma = .60\ \sigma_y$	tension (1.5.1.4.4.) $\sigma = .60\ \sigma_y$
compression (1.5.1.4.3) $\sigma = .60\ \sigma_y$	compression (1.5.1.4.5) $\sigma = \left[1.0 - \frac{\left(\frac{L}{r}\right)^2}{2\ C_c^2\ C_b}\right] .60\ \sigma_y$ (AISC Formula 4); $\sigma = \frac{12{,}000{,}000}{\frac{Ld}{A_f}}$ (AISC Formula 5). Use the larger of (4) or (5) but not to exceed $.60\ \sigma_y$. If $\frac{L}{r} < 40$, don't need to use (4)

reduction in allowable compressive bending stress due to possible lateral displacement of web. (1.10.6)

when $\frac{d_w}{t_w} < \frac{24{,}000}{\sqrt{\sigma_b}}$ $\quad \sigma_b$ = allowable compression stress from above

$$\text{use } \sigma = \sigma_b\left[1.0 - .0005\ \frac{A_w}{A_f}\left(\frac{d_w}{t_w} - \frac{24{,}000}{\sqrt{\sigma_b}}\right)\right]$$

(AISC Formula 11)

* This ratio may be exceeded if the compressive bending stress, using a width not exceeding this limit, is within the allowable stress. The above table does not include the higher bending stress ($\sigma = .66\ \sigma_y$) for "compact" sections because most fabricated plate and box girders will exceed the width-thickness ratio of "compact" sections.

Allowable Bending Stresses

Table 2 summarizes the AISC allowable bending stresses for plate and box girders.

In Table 2:

L = span or unbraced length of compression flange

r = radius of gyration of a Tee section comprising the compression flange plus 1/6 of the web area, about the y-y axis (in the plane of the web). For girders symmetrical about their x-x axis of bending, substitution of r_y of the entire section is conservative

A_f = area of the compression flange

$$\boxed{C_b = 1.75 - 1.05\ \frac{M_1}{M_2} + .3\ \left(\frac{M_1}{M_2}\right)^2}$$

(but not more than 2.3; can conservatively be taken as 1.0)

$$\boxed{C_c = \sqrt{\frac{2\ \pi^2\ E}{\sigma_y}}}$$

σ_b = allowable compressive bending stress from above

M_1 is the smaller, and M_2 is the larger bending moment at the ends of the unbraced length (L), taken about the strong axis of the member. M_1/M_2 is the ratio of these end moments. When M_1 and M_2 have

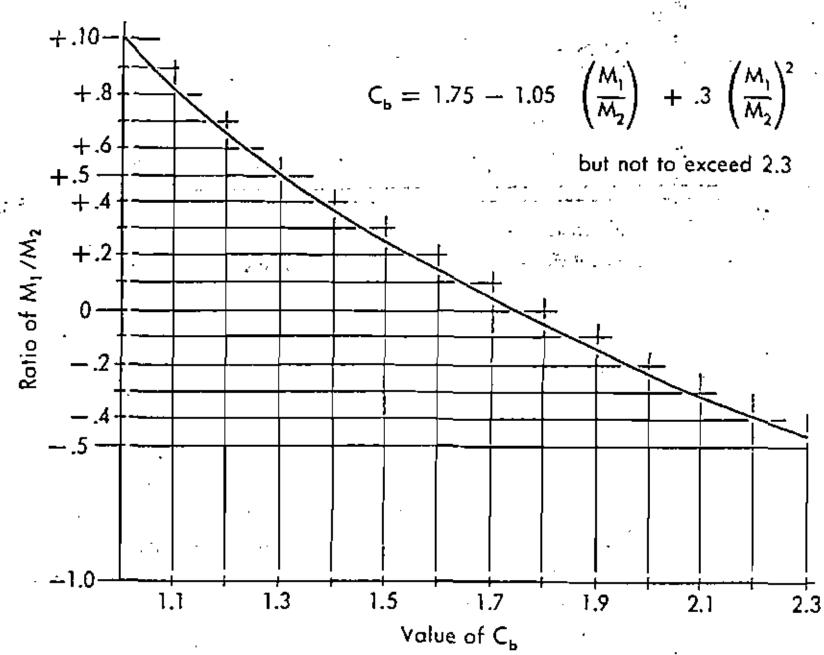


FIGURE 1

the same signs, this ratio is positive; when they have different signs, it is negative. When the bending moment within an unbraced length is larger than that at both ends of this length, the ratio is taken as unity.

Figure 1 is a graph showing the value of C_b for any given ratio of M_1/M_2.

When the bending moment within an unbraced length is larger than that at both ends of this length, *the ratio shall be taken as unity, and C_b becomes 1.0.*

2. TRANSVERSE INTERMEDIATE STIFFENERS

Loads applied to beams and girders cause bending moments along the length of the member. When these moments are non-uniform along the length of the member, both horizontal and vertical shear stresses are set up because shear is equal to the rate of change of moment.

The horizontal shear forces would cause the flange of a plate girder to slide past the web if it were not for the fillet welds joining them.

These horizontal and vertical shear stresses combine and produce both diagonal tension and compression, each at 45° to the shear stresses. In steel structures, tension is not the problem; however, the diagonal compression could be high enough to cause the web to buckle. Stiffeners are used to prevent the web from buckling in regions of high shear stress.

The ratio of web thickness to clear depth of web in the older specifications was based on predications of the plate buckling theory: the web being subjected to shear throughout its depth, and to compressive bending stresses over a portion of its depth. See Figure 2.

The plate buckling theory assumes the portion of the web between stiffeners to be an isolated plate; however, in the plate girder, the web is part of a built-up member. When the critical buckling stress in the web is reached, the girder does not collapse. This *is because the flanges carry all of the bending moment,*

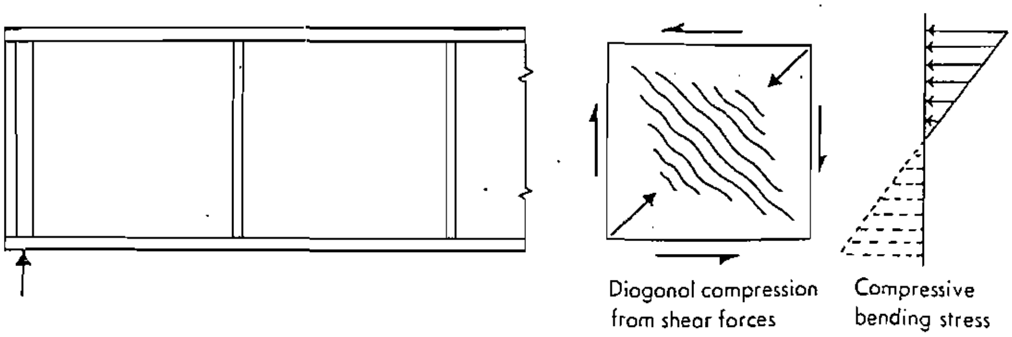


FIGURE 2

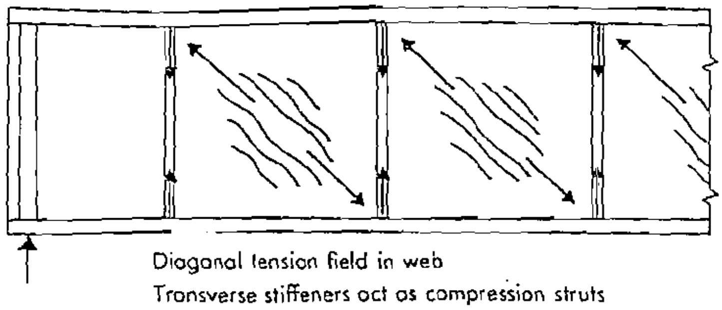


FIGURE 3

the buckled web then serves as a tension diagonal, and the transverse stiffeners become the vertical compression struts. This in effect makes the plate girder act as a truss. See Figure 3.

The carrying capacity of the plate girder is greater under this analysis, being equal to that supported by the beam action shear (Fig. 2) and that supported by the diagonal tension field in the web (Fig. 3). AISC Formulas 8 and 9 will meet this requirement. These formulas appear further along on this page.

AISC Specifications

Intermediate stiffeners are not required when the ratio (d_w/t_w) is less than 260 and the maximum web shear stress is less than that permitted by AISC Formula 9 (AISC 1.10.5.3).

Figure 4 partially summarizes the AISC specifications for intermediate stiffeners.

These requirements apply:

1. If single stiffeners are used, they must be welded to compression flange (AISC 1.10.5.4).

2. Intermediate stiffeners may be cut short of tension flange for a distance less than 4 t_w when not needed for bearing (AISC 1.10.5.4).

3. For intermittent fillet welds, clear spacing (s) between lengths of weld must $\leqq$ 16 t_w and $\leqq$ 10″ (AISC 1.10.5.4).

4. Welds joining stiffeners to web must be sufficient to transfer a total unit shear force of—

$$f_s = d_w \sqrt{\left(\frac{\sigma_y}{3400}\right)^3} \qquad \text{(AISC 1.10.5.4)}$$

This shear force to be transferred may be reduced in same proportion that the largest computed shear stress (τ) in the adjacent panel is less than that allowed by AISC Formula 8 (AISC 1.10.5.4).

5. If lateral bracing is attached to stiffener, welds connecting stiffener to compression flange must be sufficient to transfer a horizontal force (F) = 1% of flange force (AISC 1.10.5.4).

When intermediate stiffeners are required, their maximum spacing (a) depends on three items: a/d_w, d_w/t_w, and shear stress (τ).

The largest average web shear stress ($\tau_{av} = V/A_w$) in any panel between transverse intermediate stiffeners shall not exceed the following (AISC 1.10.5.2):

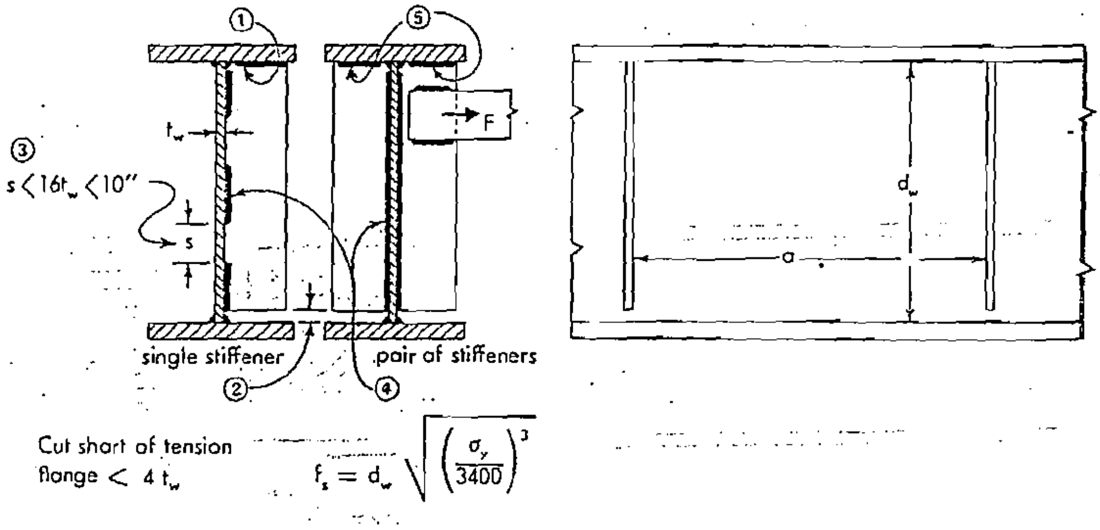


FIGURE 4

when $C_v < 1.0$

$$\boxed{\tau \leqq \frac{\sigma_y}{2.89}\left[C_v + \frac{1 - C_v}{1.15\sqrt{1 + \left(\frac{a}{d_w}\right)^2}}\right] \text{ (AISC Formula 8)}} \quad \text{..(3a)}$$

This provides an allowable shear stress (τ) up to about .35 σ_y and takes advantage of tension field action.

when $C_v > 1.0$ *or when no stiffeners are used*

$$\boxed{\tau \leqq \frac{\sigma_y\, C_v}{2.89} < .40\ \sigma_y \text{ (AISC Formula 9)}} \quad \text{..........(3b)}$$

This provides an allowable shear stress (τ) within the range of .347 σ_y to .40 σ_y and does not take advantage of tension field action.

where:

a = clear distance between transverse stiffeners, in.

d_w = clear distance between flanges, in.

t_w = thickness of web, in.

σ_y = yield strength of girder steel, psi

when $C_v < .8$

$$\boxed{C_v = \frac{45{,}000{,}000\ k}{\sigma_y\ (d_w/t_w)^2}}$$

when $C_v > .8$

$$\boxed{C_v = \frac{6{,}000}{(d_w/t_w)}\sqrt{\frac{k}{\sigma_y}}}$$

when $a/d_w < 1.0$

$$\boxed{k = 4.00 + \frac{5.34}{(a/d_w)^2}}$$

when $a/d_w > 1.0$

$$\boxed{k = 5.34 + \frac{4.00}{(a/d_w)^2}}$$

Above, the one C_v formula picks up exactly where the other leaves off. The value of C_v may be read directly from the nomograph, Figure 5, without separately computing the value of k.

Both ASIC Formulas 8 and 9 contain a basic factor $\left(\frac{\sigma_y\, C_v}{2.89}\right)$ which you will notice is the same as $\left(\frac{.60\ \sigma_y\, C_v}{\sqrt{3}}\right)$ or (.347 $\sigma_y\, C_v$). The expression (.60 σ_y) is recognized as the basic allowable tensile stress and $\left(\frac{\sigma_y}{\sqrt{3}}\right)$ as (τ_y).

For greater depth to thickness of web (d_w/t_w) and greater stiffener spacing (a/d_w), the values of (C_v) will become lower. This will result in lower values for the allowable shear stress in the web. For these conditions, AISC Formula 8 has an additional factor which takes advantage of the increased carrying capacity provided by the diagonal tension field and results in a higher shear allowable. When $C_v = 1$, this factor becomes zero and AISC Formula 8 becomes Formula 9.

The ratio a/d_w shall not exceed (AISC 1.10.5.3):

$$\boxed{a/d_w \leqq \frac{260}{d_w/t_w}} \quad \text{..........(4)}$$

nor

$$\boxed{a/d_w \leqq 3.0} \quad \text{..........(5)}$$

These arbitrary values provide a girder which will facilitate handling during fabrication and erection.

When a/d_w exceeds 3.0, its value is taken as infinity. Then AISC Formula 8 reduces to AISC Formula 9 and k = 5.34 (AISC 1.10.5.2).

This work can be greatly simplified by using the appropriate AISC Table 3 for the specific yield point of steel. See AISC's "Specification for the Design, Fabrication and Erection of Structural Steel for Buildings" and Bethlehem Steel Corp's *Steel Design File* on "V Steels—Recommended Allowable Stresses for Building Design."

In end panels and panels containing large holes, the smaller dimension (a or d_w) shall not exceed (AISC 1.10.5.3)—

$$\boxed{a \text{ or } d_w \leqq \frac{11{,}000\ t_w}{\sqrt{\tau}}} \quad \text{..........(6)}$$

where τ is the computed average shear stress in the web:

$$\tau = \frac{V}{A_w}$$

It is necessary that the stiffeners have sufficient cross-sectional area for them to act as compressive struts to resist the vertical component of the tension field in the web.

This cross-sectional area, in square inches, of intermediate stiffeners when spaced in accordance with

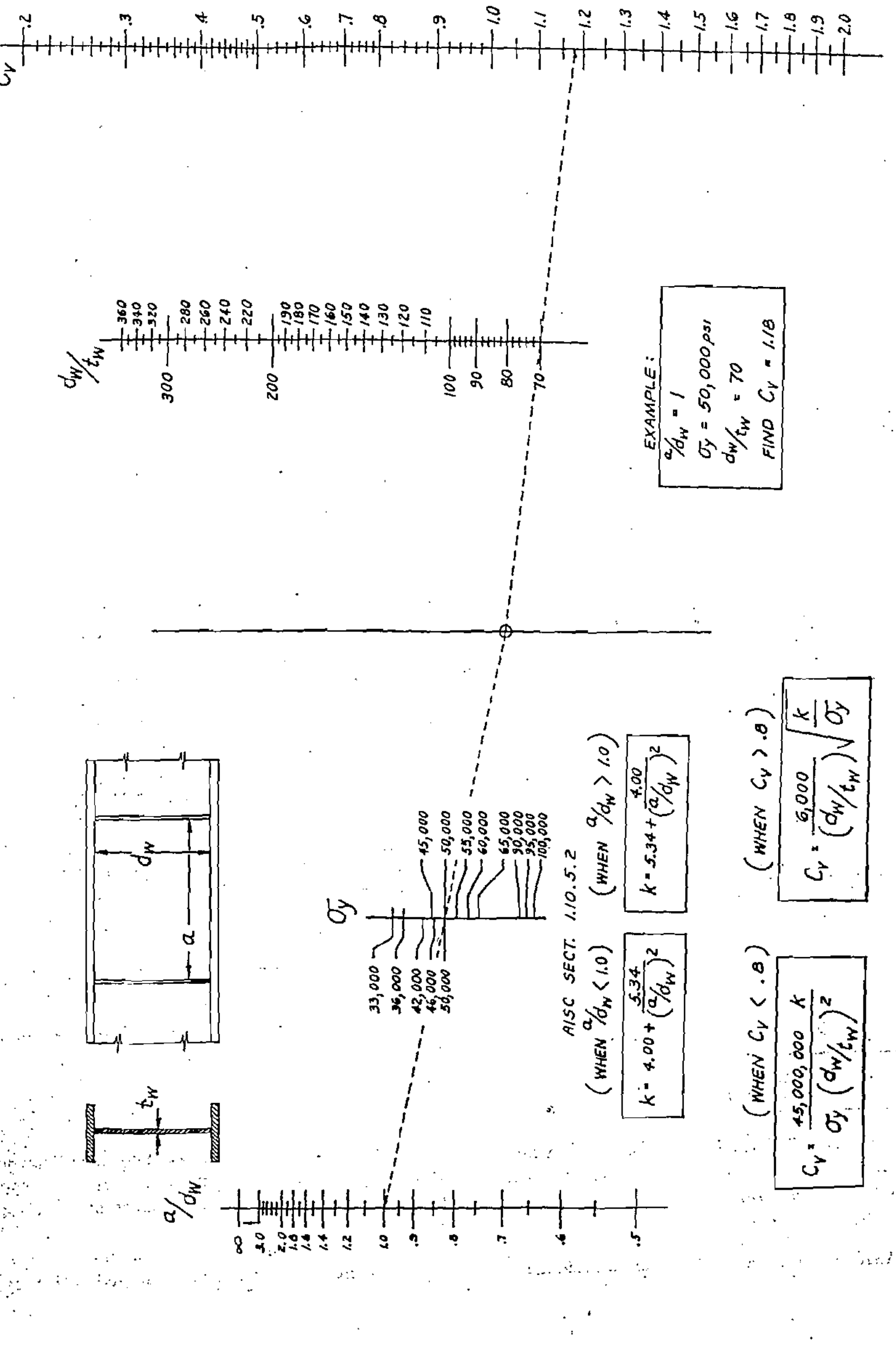


FIG. 5 Value of C_v to Determine Allowable Web Shear Stress (1.10.5.2)

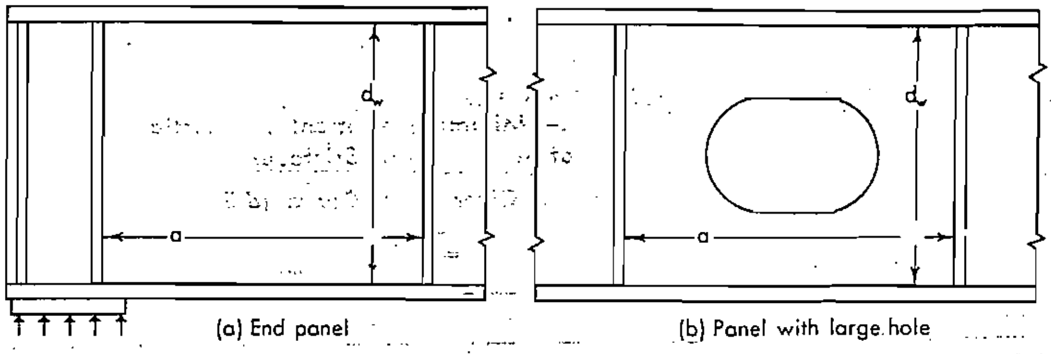


FIGURE 6

AISC Formula 8 (total area when in pairs) must not be less than (AISC 1.10.5.4)—

$$A_s \geqq \frac{1 - C_v}{2}\left[\frac{a}{d_w} - \frac{(a/d_w)^2}{\sqrt{1 + (a/d_w)^2}}\right] Y\, D\, d_w\, t \qquad (7)$$

(AISC Formula 10)

(See the appropriate AISC Table 3)

where:

$$Y = \frac{\text{yield point of web steel}}{\text{yield point of stiffener steel}}$$

D = 1.0 for a pair of stiffeners
1.8 for a single angle stiffener
2.4 for a single plate stiffener

When the greatest shear stress (τ) in a panel is less than that permitted by AISC Formula 8, this area (A_s) requirement may be reduced in like proportion (AISC 1.10.5.4).

The moment of inertia of a pair of stiffeners or a single stiffener, with reference to an axis in the plane of the web, shall not be less than (AISC 1.10.5.4)—

$$I_s \geqq \left(\frac{d_w}{50}\right)^4 \qquad \text{.......................(8)}$$

See Tables 3, 4, and 5.

Plate girder webs, subjected to a combination of bending tensile stress and shear stress shall be checked according to the following interaction formula:

$$\underline{\sigma_b} \leqq \left(0.825 - 0.375\, \frac{\tau}{\underline{\tau}}\right) \sigma_y \text{ or } .60\, \sigma_y \qquad \text{....(9)}$$

(AISC Formula 12)

where:

τ = computed average web shear stress $= \dfrac{V}{A_w}$

$\underline{\tau}$ = allowable web shear stress from AISC Formulas 8 or 9

$\underline{\sigma_b}$ = allowable bending tensile stress

It can be shown that this formula will result in—

a) full bending tensile stress allowable, if the concurrent shear stress is not greater than 60% of the full allowable value, or
b) full shear stress allowable, if the concurrent bending tensile stress is not greater than 75% of the full allowable value.

See Table 6B for abbreviated Formula 12 to use for a specific yield strength of steel.

3. BEARING STIFFENERS

Concentrated loads cause high compressive stress at the web toe of the fillet along a distance of N + K for end reactions, and N + 2K for interior loads.

If there are no bearing stiffeners, this compressive stress shall not exceed (AISC 1.10.10.1)—

for end reactions

$$\sigma = \frac{R}{t_w(N + K)} \leqq .75\, \sigma_y \qquad \text{...........(10a)}$$

(AISC Formula 14)

for interior loads

$$\sigma = \frac{R}{t_w(N + 2K)} \leqq .75\, \sigma_y \qquad \text{...........(10b)}$$

(AISC Formula 13)

Also, the sum of the compressive stresses from concentrated and distributed loads on the compression edge of the web plate not supported directly by bearing stiffeners shall not exceed (AISC 1.10.10.2)—

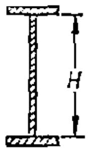


TABLE 3—Minimum Moment of Inertia of Intermediate Stiffener

For a Given Web Depth (d_w)

$$I_s \geq \left(\frac{d_w}{50}\right)^4$$

d_w	I_s	d_w	I_s	d_w	I_s	d_w	I_s	d_w	I_s
10	.00160	45	.656	80	6.56	115	28.0	150	81.0
11	.00234	46	.717	81	6.89	116	29.0	151	83.2
12	.00332	47	.781	82	7.24	117	30.0	152	85.3
13	.00457	48	.849	83	7.60	118	31.0	153	87.7
14	.00615	49	.924	84	7.96	119	32.0	154	90.0
15	.00810	50	1.00	85	8.36	120	33.1	155	92.3
16	.0105	51	1.08	86	8.76	121	34.3	156	94.7
17	.0134	52	1.17	87	9.17	122	35.4	157	97.2
18	.0168	53	1.26	88	9.60	123	36.6	158	99.7
19	.0208	54	1.36	89	10.0	124	37.8	159	102.0
20	.0256	55	1.46	90	10.5	125	39.1	160	104.5
21	.0311	56	1.57	91	11.0	126	40.3	161	107.3
22	.0375	57	1.69	92	11.5	127	41.6	162	110.0
23	.0448	58	1.81	93	12.0	128	42.9	163	112.7
24	.0531	59	1.94	94	12.5	129	44.3	164	115.7
25	.0625	60	2.07	95	13.0	130	45.7	165	118.3
26	.0731	61	2.22	96	13.6	131	47.1	166	121.5
27	.0850	62	2.37	97	14.2	132	48.6	167	124.3
28	.0984	63	2.52	98	14.8	133	50.1	168	127.5
29	.113	64	2.68	99	15.4	134	51.6	169	130.2
30	.130	65	2.86	100	16.0	135	53.2	170	133.5
31	.148	66	3.04	101	16.7	136	54.8	171	136.5
32	.168	67	3.22	102	17.3	137	56.3	172	140.0
33	.190	68	3.42	103	18.0	138	58.1	173	143.3
34	.214	69	3.63	104	18.7	139	59.8	174	146.4
35	.240	70	3.84	105	19.4	140	61.5	175	150.0
36	.269	71	4.06	106	20.2	141	63.2	176	153.0
37	.300	72	4.30	107	21.0	142	65.1	177	156.7
38	.334	73	4.54	108	21.8	143	66.8	178	160.0
39	.370	74	4.80	109	22.6	144	68.7	179	164.0
40	.410	75	5.06	110	23.4	145	70.8	180	168.0
41	.451	76	5.34	111	24.3	146	72.7		
42	.499	77	5.62	112	25.2	147	74.7		
43	.547	78	5.82	113	26.1	148	76.7		
44	.600	79	6.23	114	27.0	149	78.8		

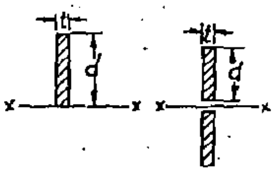


TABLE 4—Moment of Inertia of Single Flat Bar Stiffener

Double These Values for Stiffeners on Both Sides of Girder Web

$$I_s = \frac{t\,d^3}{3}$$

Thickness of bar (t)

		1/4"	5/16"	3/8"	7/16"	1/2"	5/8"	3/4"	7/8"	1"
Width of bar (d)	2"	.66	.82	.99	1.16	1.32	1.65	1.98	2.31	2.66
	2 1/2"	1.46	1.83	2.19	2.56	2.93	3.66	4.39	5.21	5.85
	3"	2.25	2.81	3.37	3.94	4.49	5.62	6.75	7.87	8.99
	3 1/2"	3.55	4.44	5.32	6.21	7.10	8.88	10.6	12.4	14.2
	4"	5.33	6.66	8.00	9.32	10.7	13.3	16.0	18.6	21.2
	4 1/2"	7.59	9.49	11.4	13.3	15.2	19.0	22.8	26.6	30.4
	5"	10.4	13.0	15.6	18.2	20.9	26.1	31.3	36.5	41.7
	5 1/2"		17.3	20.8	24.2	27.7	34.7	41.6	48.5	55.5
	6"		22.5	27.0	31.5	36.0	44.9	54.0	63.0	72.0
	6 1/2"			34.3	40.0	45.7	57.1	68.5	80.0	91.4
	7"			46.9	50.0	57.2	71.4	85.7	100.	114.
	7 1/2"			52.8	61.6	70.4	88.0	105.	123.	141.
	8"				74.8	85.5	107.	128.	150.	171.
	8 1/2"				89.6	102.	128.	154.	179.	204.
	9"				106.	121.	152.	182.	212.	243.
	9 1/2"					142.	178.	214.	250.	285.
	10"					166.	208.	249.	291.	333.

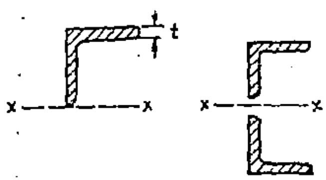


TABLE 5—Moment of Inertia of Single Angle Stiffener

Double These Values for Stiffeners on Both Sides of Girder Web

Thickness of angle stiffener (t)

		1"	7/8"	3/4"	5/8"	9/16"	1/2"	7/16"	3/8"	5/16"	1/4"
Angle size	8" × 8"	564.5	506.4	444.0	379.3	354.1	310.2				
	6" × 6"	224.0	201.9	178.5	153.8	140.4	127.2	113.0	98.3	79.9	
	5" × 5"		111.7	99.8	86.2		71.8	63.8	55.8	47.3	
	4" × 4"			48.2	42.1		35.4	31.7	27.8	23.6	19.4
	3 1/2" × 3 1/2"						23.0	20.7	18.3	15.7	12.8
	3" × 3'						14.0	12.6	11.2	9.6	7.9
	2 1/2" × 2 1/2"						7.6		6.2	5.4	4.5
	2" × 2"								3.0	2.6	2.2

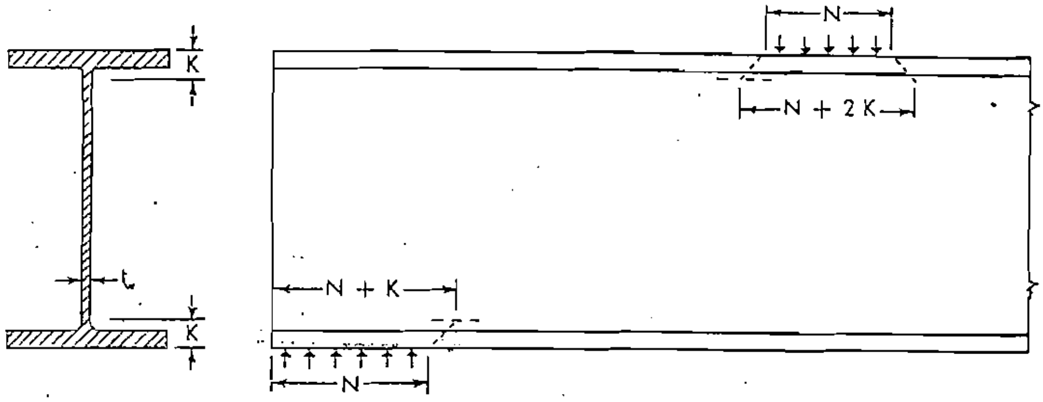


FIGURE 7

if flange restrained against rotation

$$\boxed{\sigma \leqq \left[5.5 + \frac{4}{(a/d_w)^2}\right]\frac{10{,}000{,}000}{(d_w/t_w)^2}} \quad \text{(AISC Formula 15)} \quad \ldots\ldots (11a)$$

if flange not restrained against rotation

$$\boxed{\sigma \leqq \left[2 + \frac{4}{(a/d_w)^2}\right]\frac{10{,}000{,}000}{(d_w/t_w)^2}} \quad \text{(AISC Formula 16)} \quad \ldots\ldots (11b)$$

Concentrated loads and loads distributed over a partial length of panel shall be divided by either the product of the web thickness and the girder depth or the length of panel in which the load is placed, whichever is the smaller panel dimension. Any other distributed loading, in lbs/linear in. of length, shall be divided by the web thickness.

If the above stress limits are exceeded, bearing stiffeners shall be placed in pairs at unframed ends and at points of concentrated loads, Figure 8.

Bearing stiffeners with the above sections of web are designed as columns (AISC 1.10.5.1).

These requirements apply:

1. Bearing stiffeners shall extend almost to edge of flange (AISC 1.10.5.1).

2. Bearing stiffeners shall have close bearing against flange or flanges to which load is applied (AISC 1.10.5.1).

3. Clear spacing of intermittent fillet welds $< 16\ t_w < 10''$ (AISC 1.10.5.4.).

4. Deduct leg of fillet weld or corner snipe for width of stiffener (b_s) effective in bearing at 90% σ_y (AISC 1.5.1.5.1). If parts have different yield strengths, use the lower value.

5. The limiting ratio of stiffener width to thickness shall be—

$$\frac{b_s}{t_s} \leqq \frac{3000}{\sqrt{\sigma_y}} \text{ (AISC 1.9.1)}$$

6. Use $L_e \geqq ¾\ d_w$ for slenderness ratio (L_e/r) of column section to determine allowable compressive stress (AISC 1.10.5.1); r is figured about an axis in the plane of the web.

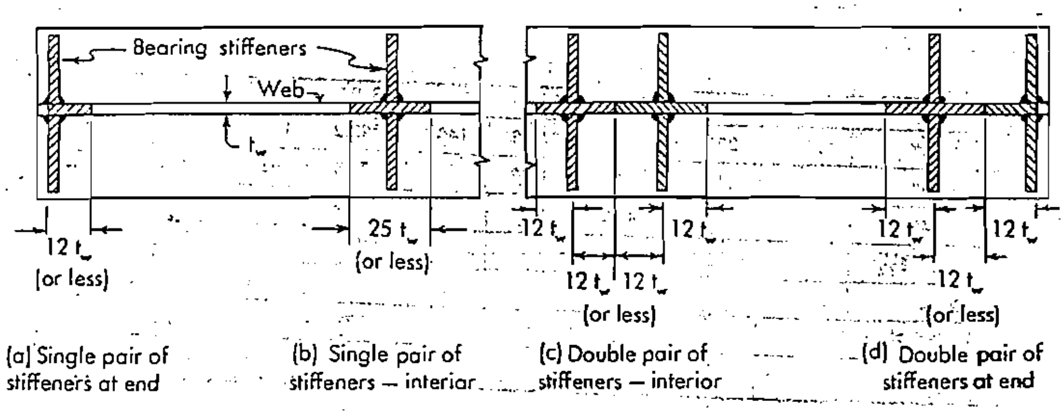


FIGURE 8

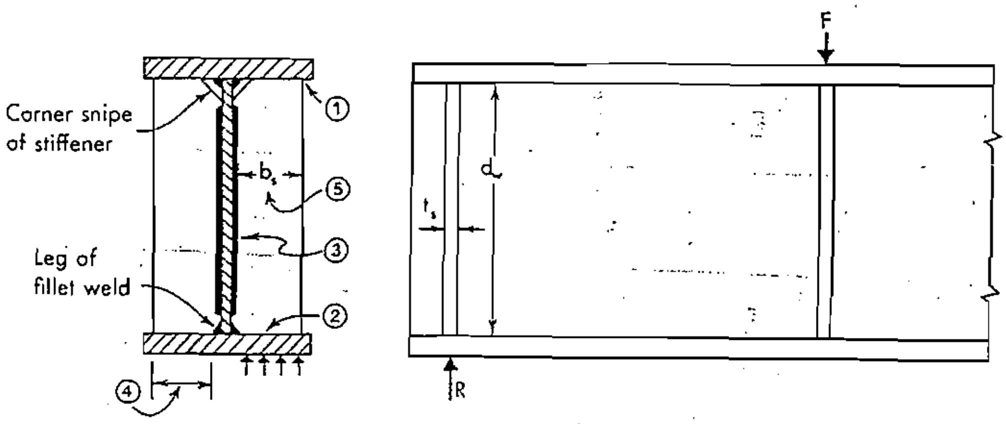


FIGURE 9

4. LONGITUDINAL FILLET WELDS

If intermittent fillet welds are used in plate or box girders, their longitudinal clear spacing shall not exceed—

tension flange (AISC 1.18.3.1)

$$\boxed{s \leqq 24 \times \text{thickness of thinner plate} \leqq 12''} \quad (12)$$

compression flange (AISC 1.18.2.3)

$$\boxed{s \leqq \frac{4000\ t_w}{\sqrt{\sigma_y}} \leqq 12''} \quad \ldots\ldots\ldots\ldots\ldots\ldots (13)$$

The longitudinal shear force on fillet weld between flange and web is—

$$f = \frac{V\ a\ y}{I\ n} \text{ lbs/linear in.}$$

where:

V = external shear on section

a = area of flange held by welds

y = distance between center of gravity of flange area held by welds, and neutral axis of entire section

I = moment of inertia of entire section

n = number of fillet welds holding flange area, usually 2 welds

5. SUMMARY OF SPECIFICATIONS

Table 6 summarizes the principal AISC specifications in easy to use form, permitting direct readout of the limiting value for the specific yield strength steel being used.

Problem 1

Design a welded plate girder to support a 120-kip uniformly distributed load, and a 125-kip concentrated load at midspan; Figure 11. Girder is to be simply supported, have a span of 50′, and have sufficient lateral support for its compressive flange. Use A36 steel and E70 or SA-2 weld metal.

FIGURE 10

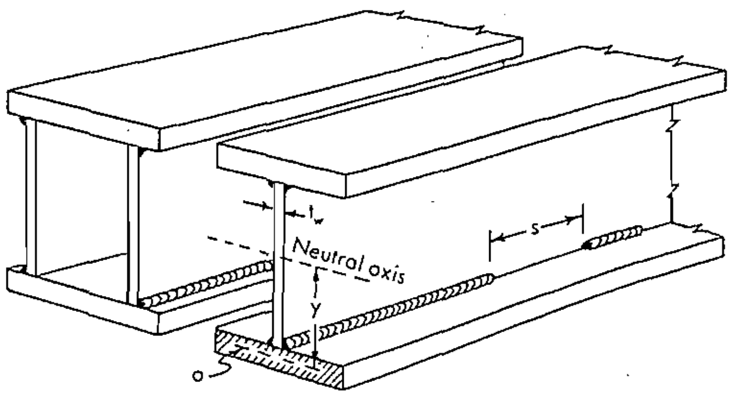


TABLE 6A—Summary of AISC Allowables and Limiting Ratios
(Expanded to Include Some Proprietary Steels)

σ_y	33,000	36,000	42,000	45,000	46,000	50,000	55,000	60,000	65,000	90,000*	95,000*	100,000*
Max depth to span of girder (suggested) $\frac{d}{L} \geq \frac{800,000}{\sigma_y}$ (AISC Commentary, p. 26)	1/24.2	1/22.2	1/19.0	1/17.8	1/17.4	1/16.0	1/14.6	1/13.3	1/12.2	1/8.8	1/8.4	1/8.0
Max width to thickness ratio of compression element $\frac{3,000}{\sqrt{\sigma_y}}$ (1.9.1)	16.5	15.8	14.6	14.1	14.0	13.4	12.8	12.2	11.8	10.0	9.7	9.5
Max width to thickness ratio of compression element $\frac{8,000}{\sqrt{\sigma_y}}$ (1.9.2)	44.0	42.1	39.0	37.7	37.3	35.8	34.1	32.6	31.4	26.6	25.9	25.3
Max width to thickness ratio of web $\frac{d_w}{t_w} \leq \frac{14,000,000}{\sqrt{\sigma_y(\sigma_y + 16,500)}}$ (1.10.2) Up to this limit the web is capable of providing vertical support for the compression flange	345	320	282	266	260	243	223	207	192	143	136	130
Bending stress allowable (tensile) $\sigma \leq .60\ \sigma_y$ (1.5.1.4.4) (1.5.1.4.3)	20,000	22,000	25,000	27,000	27,500	30,000	33,000	36,000	39,000	54,000	57,000	60,000
Max width to thickness ratio of web for no reduction in allowable compressive bending stress due to possible lateral displacement of web (1.10.6) $\frac{d_w}{t_w} \leq \frac{24,000}{\sqrt{\sigma_b}}$ (if $\sigma_b = .60\ \sigma_y$)	171	163	151	146	145	139	132	127	122	103.2	100.6	98.0
$C_c = \sqrt{\frac{2\pi^2 E}{\sigma_y}}$ (1.5.1.3)	131.7	126.1	116.7	112.8	111.6	107.0	102.0	97.7	93.8	79.8	77.6	75.7
Shear force on fillet welds between stiffener and web (1.10.5.4) $f_s = d_w \sqrt{\left(\frac{\sigma_y}{3400}\right)^3}$	30 d_w	35 d_w	43 d_w	48 d_w	50 d_w	56 d_w	65 d_w	74 d_w	84 d_w	136 d_w	153 d_w	159.5 d_w
Web crippling allowable $\sigma \leq .75\ \sigma_y$ for use in formulas 13 & 14 (1.10.10)	25,000	27,000	31,500	34,000	34,500	37,500	41,500	45,000	48,500	67,500	71,250	75,000
Max longitudinal spacing between intermittent fillet welds attaching compression flange to girder $s \leq \frac{4,000}{\sqrt{\sigma_y}} t \leq 12''$ (1.18.2.3)	22.0 t	21.0 t	19.5 t	18.9 t	18.7 t	17.9 t	17.1 t	16.3 t	15.7 t	13.3 t	13.0 t	12.6 t

* Quenched & tempered steels: yield strength at 0.2% offset.

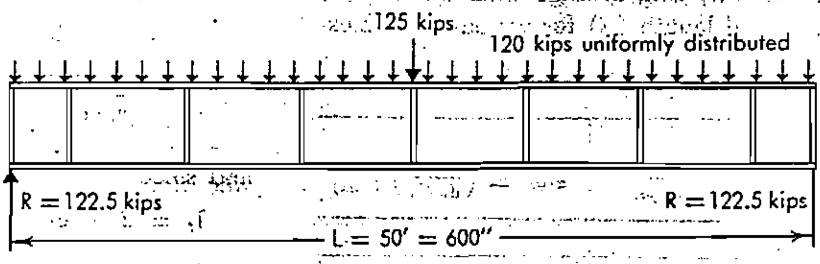


FIGURE 11

bending moment
for the uniform load,

$$M_{\mathfrak{C}} = \frac{W\,L}{8} = \frac{(120)(600)}{8} = 9{,}000 \text{ in.-kips}$$

for the concentrated load,

$$M_{\mathfrak{C}} = \frac{F\,L}{4} = \frac{(125)(600)}{4} = 18{,}750 \text{ in.-kips}$$

$$\text{Total } M_{\mathfrak{C}} = 27{,}750 \text{ in.-kips}$$

shear

$$V = 122.5 \text{ kips}$$

Design Procedure

1. Design the girder web for the shear requirements, assuming it held to a depth of 66″.

$$A_w = \frac{V}{\tau_{av}} = t_w\, d_w$$

TABLE 6B—AISC Allowable Bending Strengths
(Expanded to Include Some Important Proprietary Steels)

σ_y	AISC Formula 4 (1.5.1.4.5)	AISC Formula 12 (1.10.7)
33,000	$\sigma_b = 20{,}000 - \frac{.576}{C_b}\left(\frac{L}{r}\right)^2$	$\sigma_b = 27{,}000 - 12{,}500\,\frac{\tau}{\tau_a} \leq 20{,}000$
36,000	$\sigma_b = 22{,}000 - \frac{.692}{C_b}\left(\frac{L}{r}\right)^2$	$\sigma_b = 29{,}500 - 13{,}500\,\frac{\tau}{\tau_a} \leq 22{,}000$
42,000	$\sigma_b = 25{,}000 - \frac{.925}{C_b}\left(\frac{L}{r}\right)^2$	$\sigma_b = 34{,}500 - 15{,}500\,\frac{\tau}{\tau_a} \leq 25{,}000$
45,000	$\sigma_b = 27{,}000 - \frac{1.06}{C_b}\left(\frac{L}{r}\right)^2$	$\sigma_b = 37{,}000 - 17{,}000\,\frac{\tau}{\tau_a} \leq 27{,}000$
46,000	$\sigma_b = 27{,}500 - \frac{1.110}{C_b}\left(\frac{L}{r}\right)^2$	$\sigma_b = 38{,}000 - 17{,}000\,\frac{\tau}{\tau_a} \leq 27{,}500$
50,000	$\sigma_b = 30{,}000 - \frac{1.310}{C_b}\left(\frac{L}{r}\right)^2$	$\sigma_b = 41{,}000 - 18{,}500\,\frac{\tau}{\tau_a} \leq 30{,}000$
55,000	$\sigma_b = 33{,}000 - \frac{1.59}{C_b}\left(\frac{L}{r}\right)^2$	$\sigma_b = 45{,}500 - 20{,}500\,\frac{\tau}{\tau_a} \leq 33{,}000$
60,000	$\sigma_b = 36{,}000 - \frac{1.89}{C_b}\left(\frac{L}{r}\right)^2$	$\sigma_b = 49{,}500 - 22{,}500\,\frac{\tau}{\tau_a} \leq 36{,}000$
65,000	$\sigma_b = 39{,}000 - \frac{2.22}{C_b}\left(\frac{L}{r}\right)^2$	$\sigma_b = 53{,}500 - 24{,}500\,\frac{\tau}{\tau_a} \leq 39{,}000$
90,000*	$\sigma_b = 54{,}000 - \frac{4.24}{C_b}\left(\frac{L}{r}\right)^2$	$\sigma_b = 74{,}250 - 33{,}750\,\frac{\tau}{\tau_a} \leq 54{,}000$
95,000*	$\sigma_b = 57{,}000 - \frac{4.73}{C_b}\left(\frac{L}{r}\right)^2$	$\sigma_b = 78{,}375 - 35{,}625\,\frac{\tau}{\tau_a} \leq 57{,}000$
100,000*	$\sigma_b = 60{,}000 - \frac{5.23}{C_b}\left(\frac{L}{r}\right)^2$	$\sigma_b = 82{,}500 - 37{,}500\,\frac{\tau}{\tau_a} \leq 60{,}000$

* Quenched & tempered steels; yield strength at 0.2% offset.

τ = average shear stress in web $= \frac{V}{A_w}$

τ_a = allowable shear stress in web from formulas (8) or (9) or table 3

Consider the following average shear stress (τ_{av}) and maximum panel length (a) for various web thicknesses (t_w):

t_w	Actual $\tau_{av} = \frac{V}{A_w}$ (max)	Actual d_w/t_w	$a/d_w = \left(\frac{260}{d_w/t_w}\right)^2$ (max)
¼″	7430 psi	264	.97
5/16″	5950	211	1.52
⅜″	4950	176	2.18

Although the ¼″ thick web would result in a reasonable shear stress of 7430 psi, the greatest stiffener spacing (a) allowed would be 97% of the web depth (d_w); this would require more intermediate stiffeners. It would be more practical, in this example, to increase the web thickness to 5/16″, thus allowing a greater distance between stiffeners.

$$A_w = (66)(5/16) = 20.6 \text{ in.}^2$$

$$I_w = \frac{(5/16)(66)^3}{12} = 7487 \text{ in.}^4$$

$$d_w/t_w = 211$$

2. Design the flange to make up the remainder of the moment requirements. Assume a bending stress of about $\sigma = 21{,}000$ psi.

section modulus required of girder

$$S = \frac{M}{\sigma} = \frac{(27{,}750 \text{ in.-kips})}{(21{,}000 \text{ psi})} = 1320 \text{ in.}^3$$

distance from neutral axis of girder to outer fiber assuming a flange thickness of about 1″

$$c = \tfrac{1}{2} d_w + t_f = (33'') + (1'') = 34''$$

total moment of inertia required of girder

$$I_t = S\,c = (1320)(34) = 44{,}880 \text{ in.}^4$$

remaining moment of inertia required of flanges

$$I_f = I_t - I_w = (44{,}880) - (7487) = 37{,}393 \text{ in.}^4$$

and since

$$I_f = 2\,A_f\,c_f^2$$

here:

$$c_f = 33'' + \tfrac{1}{2}'' = 33.5''$$

area of flange required

$$A_f = \frac{I_f}{2\,c_f^2} = \frac{(37{,}393)}{2(33.5)^2} = 16.67 \text{ in.}^2$$

or *use two 17″ × 1″ flange plates.*

final properties of girder

$$I = 2\,(17 \text{ in.}^2)(33.5'')^2 + \frac{(5/16'')(66'')^3}{12} = 46{,}766 \text{ in.}^4 > 44{,}880 \text{ in.}^4 \quad \underline{OK}$$

$$S = \frac{I}{c} = \frac{(46{,}766 \text{ in.}^4)}{(34'')} = 1375 \text{ in.}^3 > 1320 \text{ in.}^3 \quad \underline{OK}$$

actual bending stress in girder

$$\sigma = \frac{M}{S} = \frac{(27{,}750 \text{ in.-kips})}{(1375 \text{ in.}^3)} = 20{,}200 \text{ psi}$$

reduced allowable compressive bending stress in flange due to possible lateral displacement of the web in the compression region (AISC 1.10.6)

$$\sigma_b \leqq \sigma_b\left[1.0 - .005\,\frac{A_w}{A_f}\left(\frac{d_w}{t_w} - \frac{24{,}000}{\sqrt{\sigma_b}}\right)\right]$$

$$= (22{,}000)\left[1.0 - .0005\,\frac{(20.6)}{(17)}(211 - 162)\right]$$

$$= 21{,}347 \text{ psi} > 20{,}200 \text{ psi actual} \quad \underline{OK}$$

where:

σ_b = allowable bending stress
$= .60\,\sigma_y$
$= 22{,}000$ psi

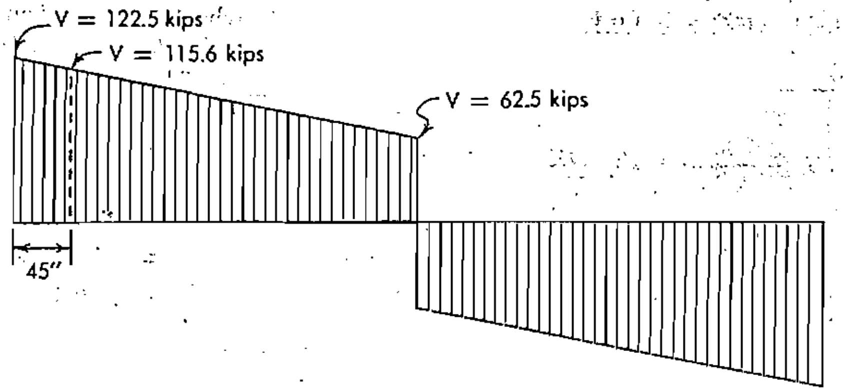


FIGURE 12

Shear diagram

3. Design the transverse intermediate stiffeners. Figure 12 is a shear diagram of the girder.

end panel distance between intermediate stiffeners (AISC 1.10.5.3)

$$a \leqq \frac{11{,}000\ t_w}{\sqrt{\tau}}$$

$$\leqq \frac{11{,}000\ (5/16)}{\sqrt{5950}}$$

$$\leqq 45.6'' \text{ or use } 45''$$

maximum shear just inside of this stiffener

$$V = (122.5 \text{ kips} - 62.5 \text{ kips})\left(\frac{255''}{300''}\right) + 62.5 \text{ kips}$$

$$= 155.6 \text{ kips}$$

maximum spacing between remaining intermediate stiffeners (AISC 1.10.5.3)

$$a/d_w \leqq 3.0 \text{ or}$$

$$\leqq \left(\frac{260}{d_w/t_w}\right)^2$$

$$\leqq \left(\frac{260}{211}\right)^2$$

$$\leqq 1.52$$

$$\text{or } a \leqq 1.52\ d_w$$

$$\leqq 1.52\ (66'')$$

$$\leqq 100''$$

required number of panels

$$600'' - 2(45'') = 510''$$

$$\frac{510''}{100''} = 5.1$$

so use 6 panels of a = 85″ each.

check the allowable shear stress in the web and determine required area of stiffener

Since the girder web's ratio is—

$$d_w/t_w = 211$$

and the ratio of panel width to web thickness is—

$$a/d_w = \frac{85''}{66''} = 1.29$$

the maximum allowable shear stress (τ) to be carried by the girder, web and the total area of stiffener (A_s) to resist this shear are found from Table 3-36 in the following manner:

	$a/d_w = 1.2$	$a/d_w = 1.3$	$a/d_w = 1.4$
$d_w/t_w = 200$	8.4 10.4		7.8 10.0
$d_w/t_w = 210$		$\tau = 8000$ psi $A_s = 10.5\%\ A_w$	
$d_w/t_w = 220$	8.2 11.0		7.5 10.6

Within the above limited area of the larger AISC table, the values in the four corner cells are read directly from the AISC table. Then the required values obtained by interpolation are filled into the center cell. Within each cell, the upper value is the allowable shear stress (τ) and the lower value is the required area of stiffener (A_s).

Thus, for our problem:

$$\tau = 8.0 \text{ kips or } 8000 \text{ psi} > 5950 \text{ psi} \quad \textit{OK}$$

$$A_s = 10.5\%\ A_w$$

$$= 10.5\%\ (20.6) \qquad = 2.16 \text{ in.}^2$$

width of stiffener (if using $t_s = 3/8''$)

$$b_s = \frac{A_w}{2\ t_s}$$

$$= \frac{(2.16)}{2(3/8)}$$

$$= 2.88'' \text{ or } \textit{use } 3\tfrac{1}{2}''$$

Since:

$$A_w = 2 b_s\ t_s$$

also check AISC Sec 1.9.1:

$$\frac{b_s}{t_s} = \frac{3\frac{1}{2}}{\frac{3}{8}}$$

$$= 9.3 \leqq \frac{3000}{\sqrt{\sigma_y}} \text{ or } 16 \quad \underline{OK}$$

required moment of inertia

$$I_s = \left(\frac{d_w}{50}\right)^4$$

$$= \left(\frac{66}{50}\right)^4$$

$$= 3.04 \text{ in.}^4$$

actual moment of inertia

$$I_s = \frac{(2 \times 3\frac{1}{2}'' + \frac{5}{16}'')^3 \frac{3}{8}''}{12}$$

$$= 12.2 \text{ in.}^4 > 3.04 \text{ in.}^4 \quad \underline{OK}$$

4. Determine the size of fillet weld joining intermediate stiffeners to the girder web.

unit shear force per linear inch of stiffener

$$f_s = d_w \sqrt{\left(\frac{\sigma_y}{3400}\right)^3}$$

$$= (66)\sqrt{\left(\frac{36{,}000}{3400}\right)^3}$$

$$= 2280 \text{ lbs/in.}$$

or $f_s = 1140$ lbs/in. for a single fillet weld (one on each side).

leg size of fillet weld

$$\omega = \frac{1140}{11{,}200}$$

$= .102''$ or use $\frac{3}{16}''$ continuous fillet

or, for a $\frac{3}{16}''$ intermittent fillet weld

$$\% = \frac{.102''}{\frac{3}{16}''}$$

$= 58.6\%$ or use $\frac{3}{16}''$ 3-5 or $\frac{3}{16}''$ 3-5

or, for a $\frac{1}{4}''$ intermittent fillet weld

$$\% = \frac{.102''}{\frac{1}{4}''}$$

$= 44\%$ or use $\frac{1}{4}''$ 3-7 or $\frac{1}{4}''$ 3-7

5. Check the combined bending tensile stress and shear stress in the girder web according to

$$\sigma_b \leqq \left(0.825 - 0.375 \frac{\tau}{\mathrm{T}}\right) \sigma_y \text{ or } .60\ \sigma_y$$

(AISC Formula 12)

wherever the calculated shear stress exceeds 60% of that allowed according to AISC Formulas 8 and 9.

The allowable shear stress was found to be $\underline{\tau} = 8000$ psi and 60% of this would be 4800 psi.

This would correspond to a shear force of

$$V = \underline{\tau}\, A_w$$

$$= (4800 \text{ psi})(\tfrac{5}{16} \times 66)$$

$$= 99.0 \text{ kips}$$

and would occur at $x = 125''$.

The bending moment at this point is—

$$M = 122.5^k\ (125'') - \left(\frac{120^k}{600''}\right)\frac{(125'')^2}{2}$$

$$= 13{,}750 \text{ in.-kips}$$

and the bending stress is—

$$\sigma_b = \frac{M}{S}$$

$$= \frac{13{,}750 \text{ in.-kips}}{1375 \text{ in.}^3}$$

$$= 10{,}000 \text{ psi}$$

It is only when the shear stress exceeds 60% of the allowable that the allowable bending stress must be reduced according to AISC Formula 12.

Since the calculated bending stress at this point ($x = 125''$) is only 10,000 psi or 45% of the allowable, and it rapidly decreases as we approach the ends, there will be no problem of the combined bending tensile stress and shear stress exceeding the allowable values of AISC Formula 12.

6. Determine the size of fillet weld joining flanges to the girder web, Figure 13.

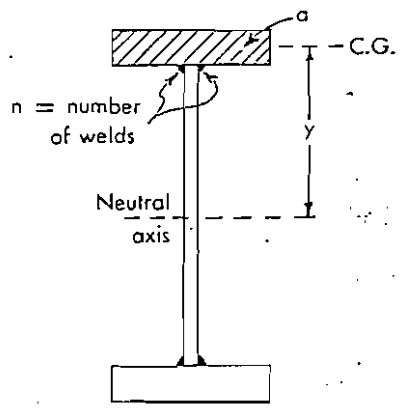


FIGURE 13

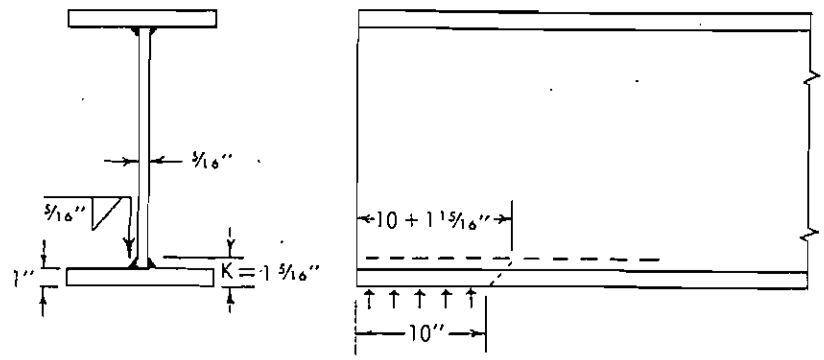


FIGURE 14

force on weld

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

$$= \frac{(122.5 \text{ kips})(17 \text{ in.}^2)(33.5'')}{(46{,}776 \text{ in.}^4)(2 \text{ welds})}$$

$$= 746 \text{ lbs/in.}$$

leg size of fillet weld

$$\omega = \frac{746}{11{,}200}$$

$$= .066''$$

but because of 1" thick flange plates, use $\frac{5}{16}''$ ◺

Bearing Stiffeners

6. Check to see if bearing stiffeners are needed at the girder ends (AISC 1.10.10.1); Figure 14.

compressive stress at web toe of girder fillet

$$\sigma = \frac{R}{t_w(N + K)}$$

$$= \frac{(122.5 \text{ kips})}{\frac{5}{16}(10'' + 1\frac{5}{16}'')}$$

$$= 34{,}700 \text{ psi} > 27{,}000 \text{ psi, or } .75\ \sigma_y$$

This stress is too high; bearing stiffeners are needed. Try a single pair and treat the stiffeners along with a portion of the web as a column. Assume an acceptable compressive stress of about 20,000 psi.

7. Determine size of bearing stiffeners.

sectional area required to carry this stress

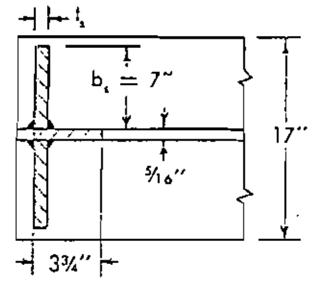
$$A = \frac{R}{\sigma}$$

$$= \frac{(122.5 \text{ kips})}{(20{,}000 \text{ psi})}$$

$$= 6.1 \text{ in.}^2$$

portion of web acting with stiffeners to form column

FIGURE 15



$$= 12\ t_w$$

$$= 12\ (\tfrac{5}{16}'')$$

$$= 3\tfrac{3}{4}''$$

area of this web portion

$$= (3\tfrac{3}{4}'')(\tfrac{5}{16}'')$$

$$= 1.17 \text{ in.}^2$$

required area of bearing stiffeners

$$6.10 - 1.17 = 4.93 \text{ in.}^2$$

If stiffeners extend almost the full width of the flange, a width of 7" will be needed on each side.

$$A_s = 2\ (7'')\ t_s$$

$$= 4.93 \text{ in.}^2$$

$$\therefore t_s = \frac{4.93}{2(7'')}$$

$$= .352 \text{ or } \textit{use } \tfrac{3}{8}'' \textit{ thickness}$$

8. Check stiffener profile for resistance to compression (AISC 1.9.1).

$$\frac{b_s}{t_s} = \frac{7}{\frac{3}{8}}$$

$$= 18.7 > \frac{3000}{\sqrt{\sigma_y}} \text{ or } 16$$

This ratio is too high, so *use a pair of 7″ x 7⁄16″ bearing stiffeners.*

9. Check this bearing stiffener area as a column; Figure 16.

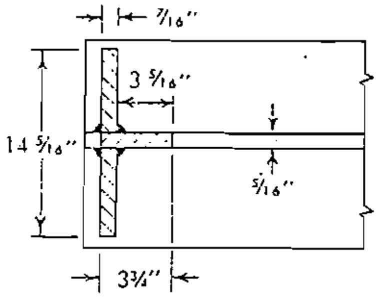


FIGURE 16

$$I_x = \frac{(7/16)(14\,5/16)^3}{12} + \frac{(3\,5/16)(5/16)^3}{12}$$

$$= 106.8 \text{ in.}^4$$

$$A = (7/16)(14\,5/16) + (3\,5/16)(5/16)$$

$$= 7.3 \text{ in.}^2$$

$$r_x = \sqrt{\frac{I_x}{A}}$$

$$= \sqrt{\frac{(106.8)}{(7.3)}}$$

$$= 4.6''$$

slenderness ratio

$$\frac{L_e}{r} = \frac{3/4(66'')}{(4.6'')}$$

$$= 10.6$$

allowable compressive stress

σ = 21,100 psi, from Table 6 in Section 3.1

and

$$R = \sigma\, A$$

$$= (21{,}100)(7.3)$$

$$= 154.0 \text{ kips} > 122.5 \text{ kips actual} \quad \underline{OK}$$

10. Determine the size of fillet weld joining bearing stiffeners to the girder web.

length of weld

$$L = 4\, d_w$$

$$= 4\ (66'')$$

$$= 264''$$

force on weld (treating weld as a line)

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

$$= \frac{(122.5 \text{ kips})}{(264'')}$$

$$= 464 \text{ lbs/in.}$$

leg size of fillet weld

$$\omega = \frac{(464)}{(11{,}200)}$$

$$= .042'' \text{ or } use\ 3/16''$$

11. Check bearing stress in these stiffeners.

bearing area of stiffener (less corner snipes)

$$(7'' - 1'')\ 7/16'' = 2.62 \text{ in.}^2 \text{ each}$$

bearing stress in stiffener

$$\sigma = \frac{R}{A}$$

$$= \frac{(122.5 \text{ kips})}{2(2.62)}$$

$$= 23{,}400 \text{ psi} < 27{,}000 \text{ psi or } .75\ \sigma_y \quad \underline{OK}$$

12. In a similar manner, check the bearing stiffener at centerline for resistance to 125-kip load. If using the same stiffener size as at ends, Figure 17:

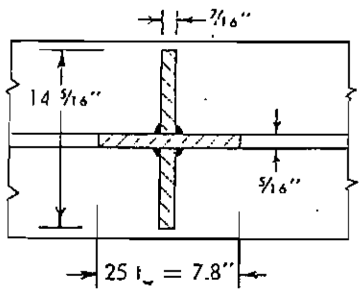


FIGURE 17

$$I_x = \frac{(7/16'')(14\,5/16'')^3}{12} + \frac{(7.8'' - 7/16'')(5/16'')^3}{12}$$

$$= 106.8 \text{ in.}^4$$

$$A = (7/16'')(14\,5/16'') + (7.8'' - 7/16'')(5/16'')$$

$$= 8.56 \text{ in.}^2$$

$$r_x = \sqrt{\frac{I_x}{A}}$$

$$= \sqrt{\frac{(106.8)}{(8.56)}}$$

$$= 3.92''$$

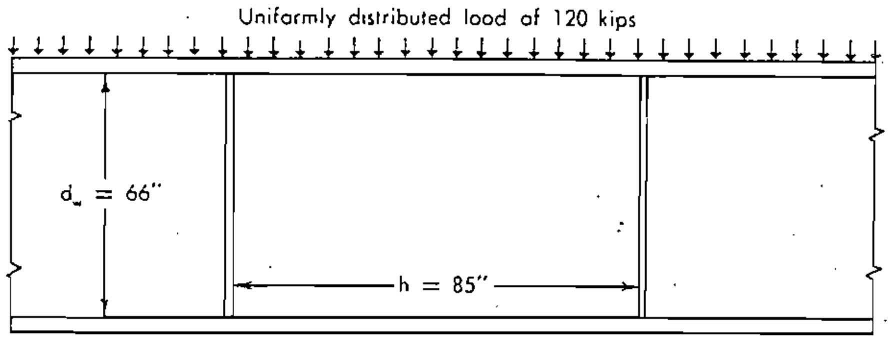


FIGURE 18

slenderness ratio

$$\frac{L_e}{r} = \frac{\frac{3}{4}(66'')}{(3.92'')}$$
$$= 12.5$$

allowable compressive stress

σ = 21,000 psi, from Table 6 in Section 3.1

and

$$F = \sigma A$$
$$= (21{,}000)(8.56)$$
$$= 179.5 \text{ kips} > 125.0 \text{ kips actual} \quad \underline{OK}$$

so use the same amount of fillet welding as before.

bearing stress in center stiffener

$$\sigma = \frac{F}{A}$$
$$= \frac{(125 \text{ kips})}{2(7'' - 1'')(\frac{7}{16}'')}$$
$$= 23{,}800 \text{ psi} < 27{,}000 \text{ psi or } .75\ \sigma_y \quad \underline{OK}$$

13. Check the compressive stresses from the uniformly distributed load of 120 kips on the compression edge of the web plate (AISC 1.10.10.2). See Figure 18.

allowable compressive stress against web edge assuming flange is not restrained against rotation

$$\sigma \leqq \left[2 + \frac{4}{(a/d_w)^2}\right]\frac{10{,}000{,}000}{(d_w/t_w)^2}$$
$$\leqq \left[2 + \frac{4}{(1.29)^2}\right]\frac{10{,}000{,}000}{(211)^2}$$
$$\leqq 990 \text{ psi}$$

actual pressure of uniform load against web edge

$$\sigma = \frac{(120 \text{ kips})}{(600'')(\frac{5}{16}'')}$$
$$= 640 \text{ psi} < 990 \text{ psi allowable} \quad \underline{OK}$$

14. Consolidate these findings into the final girder design, Figure 19.

As a matter of interest, reducing the web thickness to ¼" would have saved about 143 lbs in steel. However, this would have required 13 pairs of stiffeners instead of 9 pairs, Figure 20. The additional cost in fitting and welding the extra 4 pairs of stiffeners probably would exceed any savings in steel.

Increasing the web thickness to ⅜" would only reduce the number of stiffeners by 2 pair, Figure 21. However, this would increase the weight by 287 lbs.

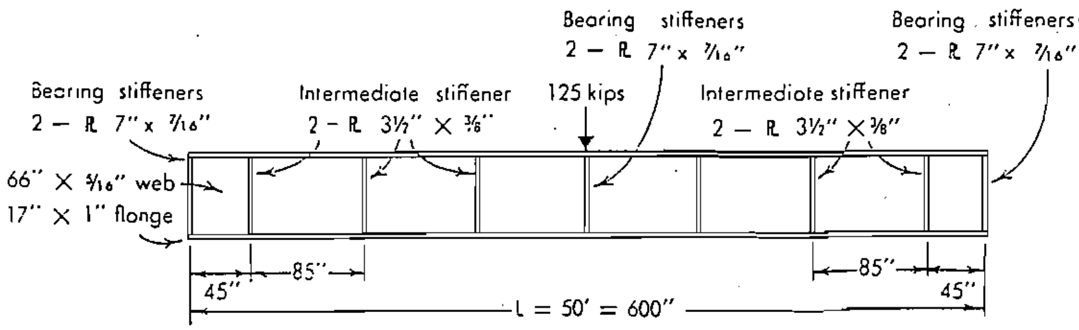


FIGURE 19

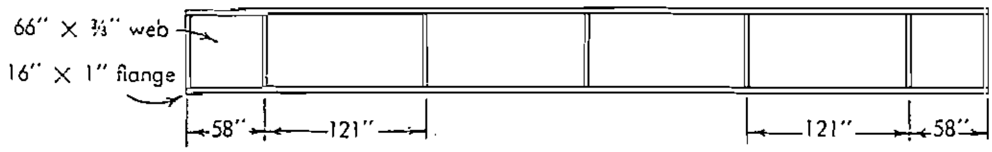


FIGURE 20

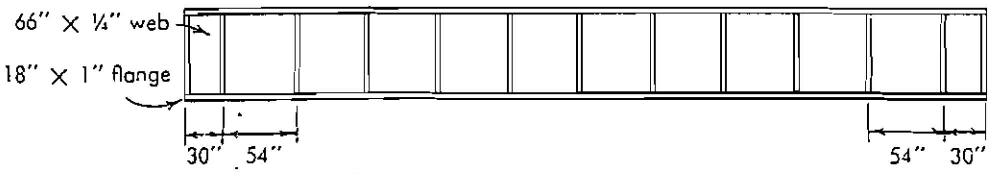


FIGURE 21

6. HOLES CUT INTO GIRDER WEB

Many times access holes must be cut into the webs of beams and girders for duct work, etc. If sufficiently large, they must be reinforced in some manner.

Since the flanges carry most of the bending forces, the loss of web area does not present much of a problem. However, since the shear (V) is carried for the most part by the web, any reduction of web area must be checked. See Figure 22.

If the hole is located at midspan (b), the shear is minimum and may have little effect on the strength of the girder. If the hole is located near the support in a region of high shear, the additional bending stresses produced by this shear must be added to the conventional bending stresses from the applied beam load. See Figure 23.

An inside horizontal flange may be added to the Tee section in order to give it sufficient bending strength, or sufficient compressive buckling strength.

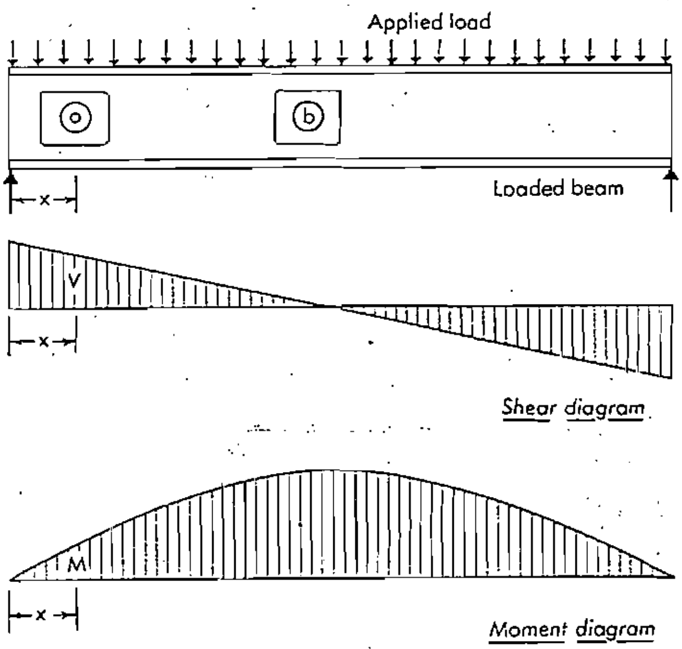


FIGURE 22

When this is done, it must be remembered that this flange becomes a part of the Tee area and is subjected to the same axial tension (F_b) and compression (F_t) force caused by the bending moment (M_x) from the external loading. Therefore, this flange must extend far enough beyond the web opening to effectively transfer this portion of the axial force back into the main web of the girder; see Figure 24. Of course in the region of low moment (M_x), this axial force may be low and not require this extra length of flange.

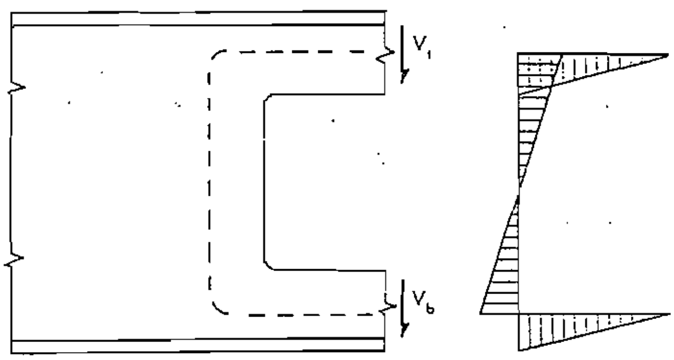


FIGURE 23

If these access holes in the web are close enough together, the portion of the web between the holes behaves in the same manner as the vertical members of a Vierendeel truss. See Figure 25.

Unless the bending stress at the corner of the access hole is rather low, reinforcement of this corner should be considered:

1. Because of the abrupt change in section, there is a stress concentration several times the average stress value. See Figure 26.

2. The Tee section at this inside corner behaves similar to a curved beam in that the neutral axis shifts in toward this inner corner, greatly increasing the bending stresses on this inward face. This increase is greater with a smaller radius of corner.

In the usual analysis of a Vierendeel truss, the horizontal shear (V_h) along the neutral axis of the

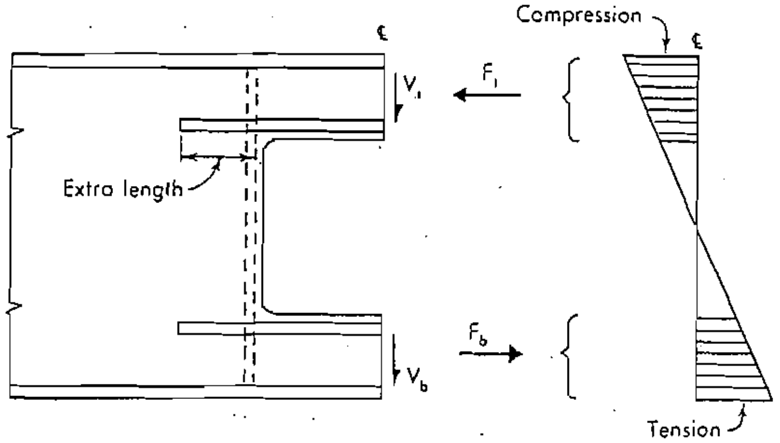


FIGURE 24

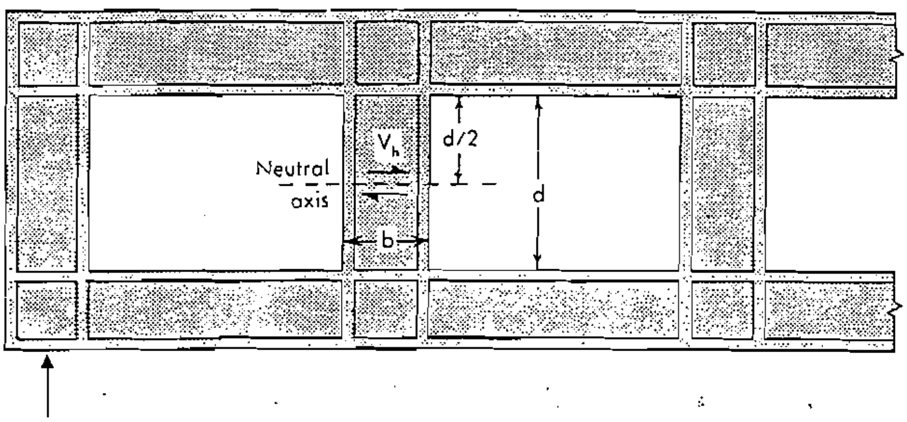


FIGURE 25

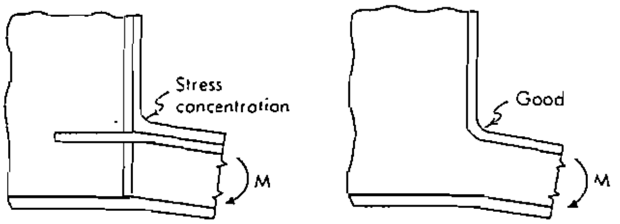


FIGURE 26

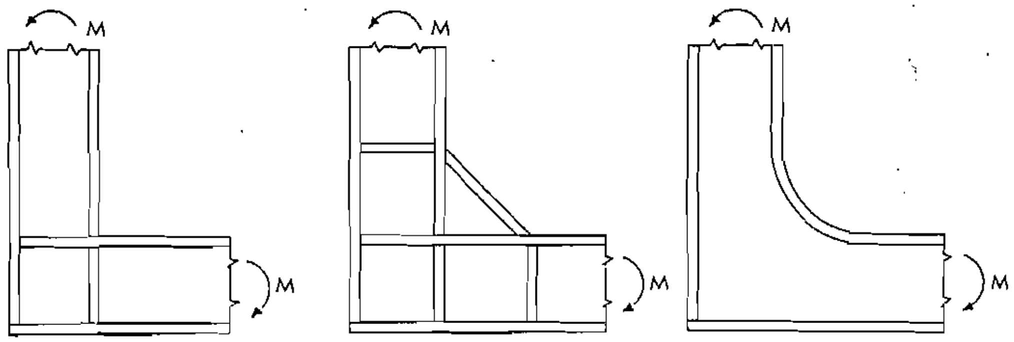
truss in the vertical member is assumed to cause a moment at the upper and lower ends of this vertical member. If the horizontal dimension of this member (b) is insufficient to resist this bending moment, it will be necessary to add vertical flange plates; see Figure 25.

It has been found in tests of various types of knees for rigid frames that the square corner without any gusset plate or bracket is the most flexible of those tested; adding a corner bracket increases the rigidity of the connection; and a curved knee has the greatest rigidity; see Figure 27.

If these holes in the web are a sufficient distance (b) apart, the bending resistance of this web portion may be developed without the additional vertical flange plates; see Figure 28.

The stem of the Tee section which is subject to

FIGURE 27



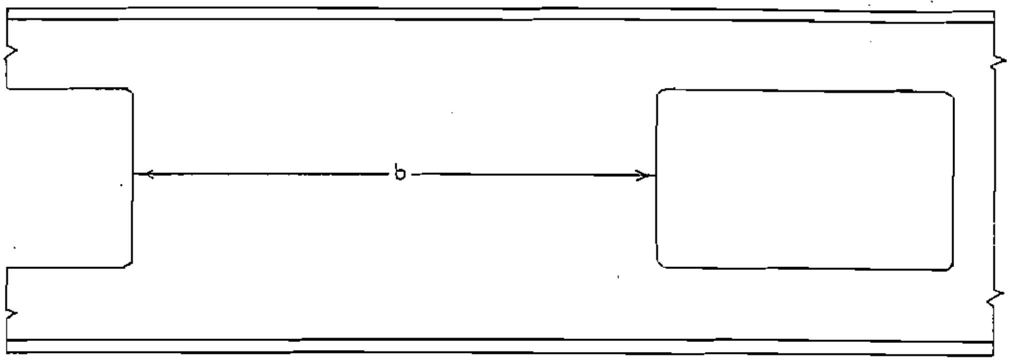


FIGURE 28

compression must be checked against buckling according to AISC 1.9.1:

$$\frac{b_f}{t_f} \geqq \frac{3000}{\sqrt{\sigma_y}}$$

$$\frac{b_s}{t_s} \geqq \frac{3000}{\sqrt{\sigma_y}}$$

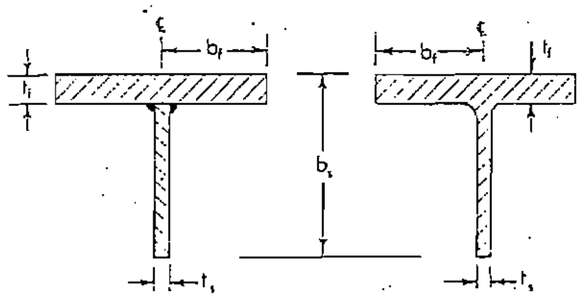


FIGURE 29

If the resulting bending stress in the stem is excessive, it must be reinforced by an inside flange or stiffener.

Corners of the hole should always be round and smooth. A minimum corner radius of 2″ is recommended when the hole is not stiffened.

Usually it is assumed the point of contraflexure of the moment in the top and bottom portions produced by the shear (V_T) and (V_b) is about midsection of the hole (℄). It is also assumed the total vertical shear is divided between these two sections in proportion to their depths. For Tees of equal depth, $V_t = V_b = \frac{1}{2} V_x$.

The top and bottom Tee sections must be capable of withstanding this combined bending stress, and the vertical shear.

A flange may be added around the edge of the web opening to give the Tee section sufficient strength for the bending moment. An additional plate may be added to the web of the Tee to give it sufficient strength for the vertical shear (V).

7. COVER PLATES

It may be advantageous in some cases to use partial-length cover plates in the bearing regions of a beam or girder, to reduce the required thickness of the flange plate extending from end-to-end of the member.

Related discussion will be found further along in this text under Section 4.3 on Welded Plate Girders for Bridges (see Topic 12) and under Section 6.1 on Design of Rigid Frames (see Topic 3).

The termination of partial-length cover plates for buildings is governed by AISC Sec. 1.10.4. The following paragraphs summarize these requirements.

Partial-length cover plates shall extend beyond the theoretical cut-off point for a distance (a′), defined below. This extended portion (a′) shall be attached to the beam or girder with sufficient fillet welds to develop the cover plate's portion of the bending force

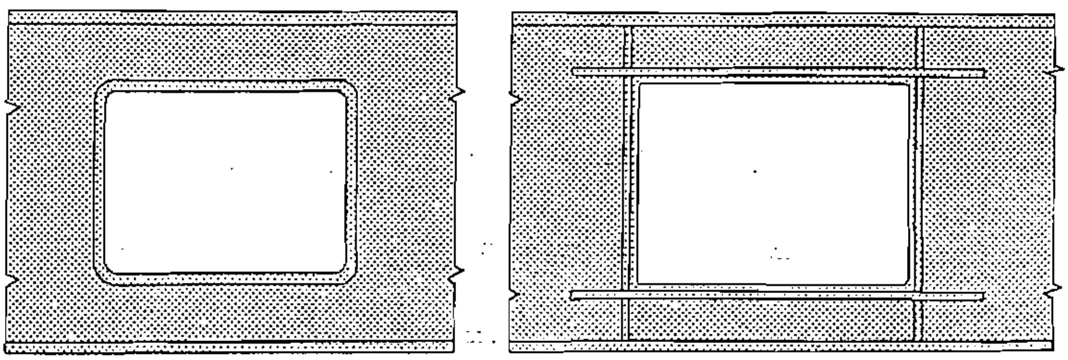


FIGURE 30

in the beam or girder at the theoretical cut-off point which is equal to—

$$\boxed{F_{weld} = \frac{M\ Q}{I}} \quad \text{.......................(14)}$$

where:

M = bending moment at section in question

Q = statical moment of cover plate area about neutral axis of cover-plated beam section

I = moment of inertia of cover-plated beam section

The moment, computed by equating $\frac{M\ Q}{I}$ to the capacity of the connecting fillet welds in this distance (a′) from the actual end of the cover plate, must equal or exceed the moment at the theoretical cut-off point. Otherwise, the size of the fillet welds in this terminal section (a′) must be increased, or the actual end of the cover plate must be extended to a point of lower moment.

The length (a′) measured from the actual end of the cover plate shall be:

1. A distance equal to the width of the cover plate when there is a continuous fillet weld equal to or larger than ¾ of the plate thickness across the end of the plate and continued welds along both edges of the cover plate in the length (a′).

2. A distance equal to 1½ times the width of the cover plate when there is a continuous fillet weld smaller than ¾ of the plate thickness across the end of the plate and continued welds along both edges of the cover plate in the length (a′).

3. A distance equal to 2 times the width of the cover plate when there is no weld across the end of the plate but continuous welds along both edges of the cover plate in the length (a′).

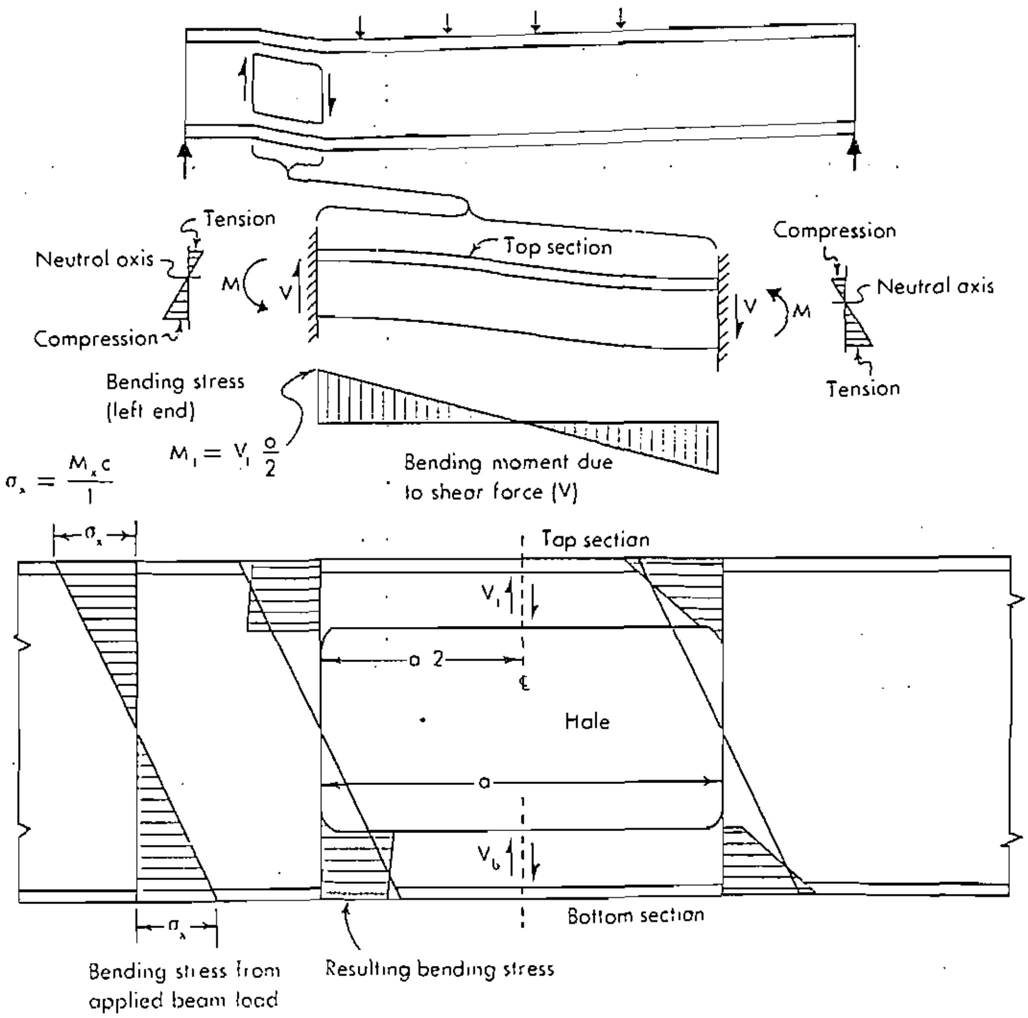
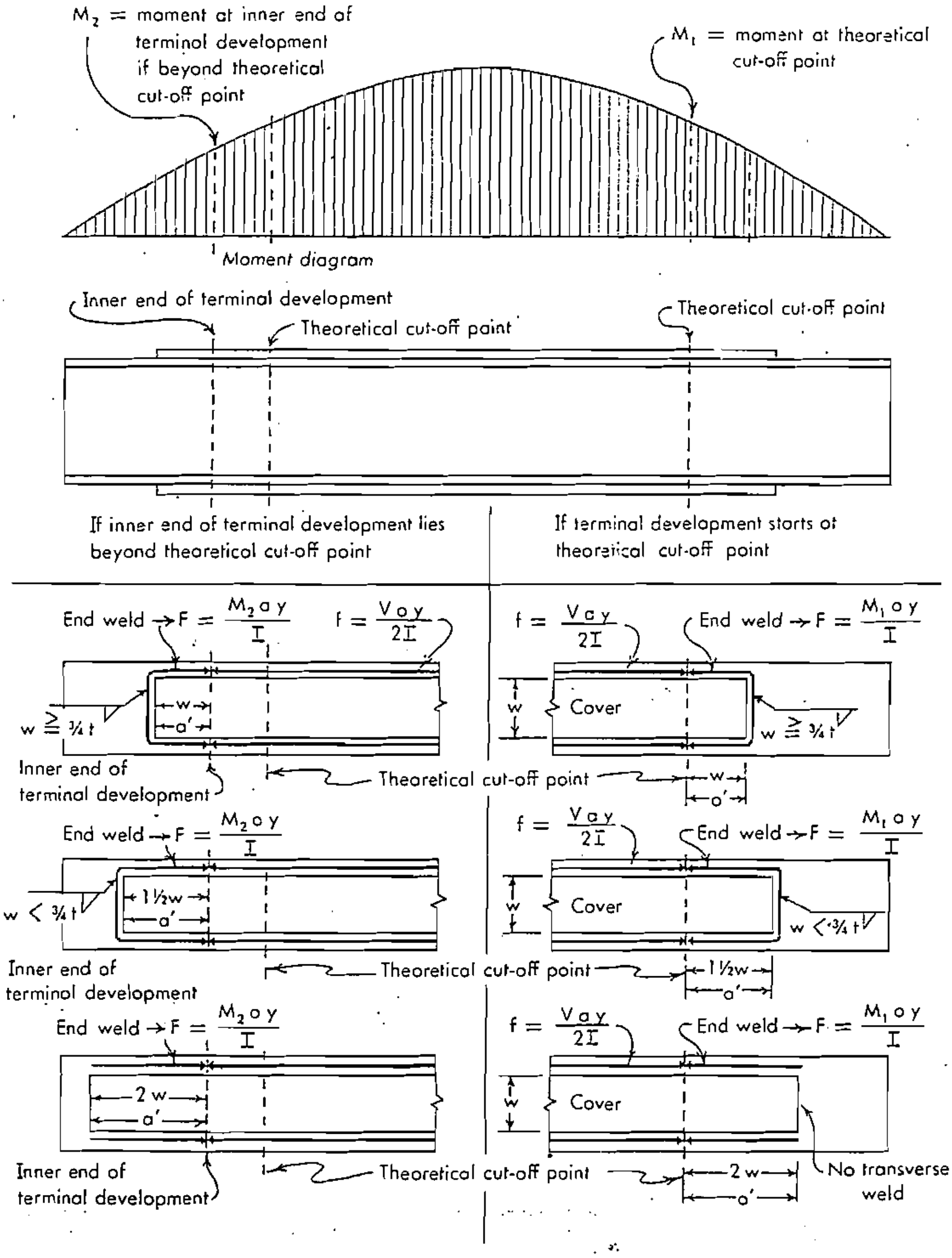


FIGURE 31



M_2 = moment at inner end of terminal development if beyond theoretical cut-off point
M_1 = moment at theoretical cut-off point
Moment diagram
Inner end of terminal development
Theoretical cut-off point
Theoretical cut-off point
If inner end of terminal development lies beyond theoretical cut-off point
If terminal development starts at theoretical cut-off point
End weld → $F = \frac{M_2 a y}{I}$
$f = \frac{V a y}{2I}$
$w \geqq \frac{3}{4} t$
w
a'
Inner end of terminal development
Theoretical cut-off point
$f = \frac{V a y}{2I}$
End weld → $F = \frac{M_1 a y}{I}$
Cover
$w \geqq \frac{3}{4} t$
w
a'
End weld → $F = \frac{M_2 a y}{I}$
$w < \frac{3}{4} t$
1½w
a'
Inner end of terminal development
Theoretical cut-off point
$f = \frac{V a y}{2I}$
End weld → $F = \frac{M_1 a y}{I}$
Cover
$w < \frac{3}{4} t$
1½w
a'
End weld → $F = \frac{M_2 a y}{I}$
2 w
a'
Inner end of terminal development
Theoretical cut-off point
$f = \frac{V a y}{2I}$
End weld → $F = \frac{M_1 a y}{I}$
Cover
2 w
a'
No transverse weld

FIGURE 32

ALLOWABLE SHEAR STRESSES IN PLATE GIRDERS, KSI

And Required Gross Area of Pairs of Intermediate Stiffeners

AISC TABLE 3-33—Steel of 33 ksi Yield Point

Shear Stress, ksi (Shown on 1st line)
Stiffener Area, % of Web Area (In italics, on 2nd line)

Slenderness ratios h/t: web depth to web thickness	Aspect ratios a/h: stiffener spacing to web depth													
	0.5	0.6	0.7	0.8	0.9	1.0	1.2	1.4	1.6	1.8	2.0	2.5	3.0	over 3
70								13.0	13.0	13.0	13.0	13.0	13.0	12.6
80						13.0	13.0	12.9	12.5	12.2	12.0	11.6	11.5	11.0
90					13.0	12.9	12.1	11.5	11.3 *0.4*	11.1 *0.7*	11.0 *0.8*	10.7 *0.9*	10.6 *0.9*	9.8
100				13.0	12.4	11.6	11.2 *0.8*	11.0 *1.3*	10.7 *1.6*	10.5 *1.7*	10.4 *1.8*	10.0 *1.7*	9.8 *1.6*	8.4
110			13.0	12.2	11.4 *0.3*	11.2 *1.2*	10.8 *2.0*	10.5 *2.4*	10.2 *2.7*	9.8 *2.9*	9.5 *3.0*	9.0 *2.9*	8.6 *2.7*	6.9
120		13.0	12.3	11.4 *0.5*	11.1 *1.6*	10.8 *2.3*	10.4 *3.2*	9.8 *3.9*	9.4 *4.2*	9.0 *4.3*	8.7 *4.2*	8.2 *3.9*	7.8 *3.5*	5.8
130	13.0	12.7	11.5 *0.3*	11.1 *1.6*	10.8 *2.6*	10.4 *3.6*	9.8 *4.8*	9.2 *5.3*	8.8 *5.4*	8.4 *5.3*	8.1 *5.2*	7.6 *4.6*	7.1 *4.1*	5.0
140	13.0	11.8	11.2 *1.3*	10.9 *2.6*	10.5 *3.9*	10.0 *5.1*	9.3 *6.1*	8.8 *6.3*	8.3 *6.3*	7.9 *6.2*	7.6 *5.9*	7.0 *5.2*	6.5 *4.6*	4.3
150	12.8	11.4 *0.7*	11.0 *2.2*	10.6 *3.8*	10.1 *5.3*	9.6 *6.4*	8.9 *7.1*	8.4 *7.2*	7.9 *7.1*	7.5 *6.8*	7.2 *6.5*	6.6 *5.7*	6.1 *5.0*	3.7
160	12.0	11.2 *1.5*	10.9 *3.1*	10.3 *5.1*	9.8 *6.5*	9.3 *7.4*	8.6 *7.9*	8.1 *7.9*	7.6 *7.7*	7.2 *7.4*	6.9 *7.0*	6.2 *6.1*		3.2
170	11.5 *0.3*	11.1 *2.3*	10.6 *4.4*	10.0 *6.3*	9.5 *7.5*	9.1 *8.2*	8.4 *8.6*	7.8 *8.5*	7.3 *8.2*	6.9 *7.8*	6.6 *7.4*			2.9
180	11.3 *1.1*	10.9 *3.0*	10.3 *5.6*	9.8 *7.2*	9.3 *8.3*	8.8 *8.9*	8.2 *9.2*	7.6 *9.0*	7.1 *8.6*	6.7 *8.2*	6.4 *7.7*			2.6
200	11.1 *2.3*	10.5 *5.2*	9.9 *7.4*	9.4 *8.7*	8.9 *9.5*	8.5 *10.0*	7.8 *10.1*	7.3 *9.8*	6.8 *9.3*					2.1
220	10.8 *4.0*	10.2 *6.8*	9.6 *8.7*	9.1 *9.8*	8.7 *10.5*	8.3 *10.8*	7.6 *10.7*	7.0 *10.3*						1.7
240	10.5 *5.5*	9.9 *8.1*	9.4 *9.7*	8.9 *10.6*	8.5 *11.2*	8.1 *11.4*								1.4
260	10.3 *6.8*	9.7 *9.0*	9.2 *10.4*	8.8 *11.3*	8.3 *11.7*	7.9 *11.9*								1.2
280	10.1 *7.7*	9.6 *9.8*	9.1 *11.1*	8.6 *11.8*										
300	10.0 *8.5*	9.4 *10.4*	9.0 *11.6*											
320	9.8 *9.2*	9.3 *10.9*												
340	9.7 *9.7*													

Girders so proportioned that the computed shear is less than that given in right-hand column do not require intermediate stiffeners.

* For single angle stiffeners, multiply by 1.8; for single plate stiffeners, multiply by 2.4.

AISC TABLE 3-36—Steel of 36 ksi Yield Point

Shear Stress, ksi (Shown on 1st line)
Stiffener Area, % of Web Area (In italics, on 2nd line)

Slenderness ratios h/t: web depth to web thickness	Aspect ratios a/h: stiffener spacing to web depth													
	0.5	0.6	0.7	0.8	0.9	1.0	1.2	1.4	1.6	1.8	2.0	2.5	3.0	over 3
70							14.5	14.5	14.5	14.5	14.3	14.0	13.7	13.1
80					14.5	14.5	14.2	13.5	13.1	12.8	12.6 *0.7*	12.3 *0.3*	12.1 *0.4*	11.5
90				14.5	14.4	13.9	12.6	12.3 *0.6*	12.1 *0.9*	11.9 *1.1*	11.7 *1.2*	11.4 *1.3*	11.2 *1.2*	10.2
100			14.5	14.0	13.0	12.4 *0.5*	12.0 *1.4*	11.7 *1.8*	11.4 *2.1*	11.2 *2.1*	11.0 *2.2*	10.4 *2.3*	10.1 *2.1*	8.4
110		14.5	14.0	12.7	12.3 *1.0*	12.0 *1.8*	11.6 *2.5*	11.1 *3.1*	10.6 *3.5*	10.3 *3.6*	9.9 *3.6*	9.3 *3.4*	8.9 *3.1*	6.9
120	14.5	14.4	12.8	12.3 *1.1*	12.0 *2.1*	11.6 *2.9*	10.9 *4.1*	10.4 *4.7*	9.9 *4.9*	9.5 *4.9*	9.1 *4.8*	8.5 *4.3*	8.0 *3.8*	5.8
130	14.5	13.3	12.4 *0.9*	12.0 *2.2*	11.6 *3.2*	11.1 *4.3*	10.4 *5.6*	9.8 *5.9*	9.3 *6.0*	8.9 *5.8*	8.5 *5.6*	7.8 *5.0*	7.4 *4.4*	5.0
140	14.3	12.5 *0.3*	12.1 *1.9*	11.7 *3.2*	11.1 *4.8*	10.6 *5.9*	9.9 *6.7*	9.3 *6.9*	8.8 *6.8*	8.4 *6.6*	8.0 *6.3*	7.3 *5.5*	6.8 *4.9*	4.3
150	13.4	12.3 *1.2*	11.9 *2.8*	11.3 *4.7*	10.8 *6.1*	10.3 *7.1*	9.5 *7.6*	8.9 *7.7*	8.4 *7.5*	8.0 *7.2*	7.6 *6.8*	6.9 *6.0*	6.4 *5.2*	3.7
160	12.6 *0.1*	12.1 *2.1*	11.6 *4.1*	11.0 *6.0*	10.4 *7.2*	10.0 *8.0*	9.2 *8.4*	8.6 *8.3*	8.1 *8.1*	7.7 *7.7*	7.3 *7.3*	6.6 *6.3*		3.2
170	12.4 *0.9*	12.0 *2.8*	11.3 *5.3*	10.7 *7.0*	10.2 *8.1*	9.7 *8.7*	9.0 *9.0*	8.3 *8.9*	7.8 *8.5*	7.4 *8.1*	7.0 *7.7*			2.9
180	12.3 *1.6*	11.7 *4.0*	11.0 *6.4*	10.5 *7.9*	10.0 *8.8*	9.5 *9.4*	8.8 *9.6*	8.1 *9.3*	7.6 *8.9*	7.2 *8.5*	6.8 *8.0*			2.6
200	12.0 *2.9*	11.3 *6.0*	10.7 *8.0*	10.1 *9.2*	9.6 *10.0*	9.2 *10.4*	8.4 *10.4*	7.8 *10.0*	7.3 *9.5*					2.1
220	11.6 *4.8*	10.9 *7.5*	10.4 *9.2*	9.8 *10.2*	9.4 *10.8*	8.9 *11.1*	8.2 *11.0*	7.5 *10.6*						1.7
240	11.3 *6.2*	10.7 *8.6*	10.1 *10.1*	9.6 *11.0*	9.2 *11.5*	8.7 *11.7*								1.4
260	11.1 *7.3*	10.5 *9.5*	10.0 *10.8*	9.5 *11.6*	9.0 *12.0*	8.6 *12.1*								1.2
280	10.9 *8.2*	10.3 *10.2*	9.8 *11.4*	9.3 *12.1*										
300	10.8 *9.0*	10.2 *10.8*	9.7 *11.8*											
320	10.7 *9.5*	10.1 *11.2*												

Girders so proportioned that the computed shear is less than that given in right-hand column do not require intermediate stiffeners.

* For single angle stiffeners, multiply by 1.6; for single plate stiffeners, multiply by 2.4.

These tables simplify the design of intermediate stiffeners to AISC specifications, as discussed on pages 4.1–4, 5 and 7.

ALLOWABLE SHEAR STRESSES IN PLATE GIRDERS, KSI

And Required Gross Area of Pairs of Intermediate Stiffeners

AISC TABLE 3-42—Steel of 42 ksi Yield Point

Shear Stress, ksi (Shown on 1st line)
Stiffener Area, % of Web Area (In italics, on 2nd line)

Slenderness ratios h/t: web depth to web thickness	Aspect ratios a/h: stiffener spacing to web depth 0.5	0.6	0.7	0.8	0.9	1.0	1.2	1.4	1.6	1.8	2.0	2.5	3.0	over 3
70						17.0	17.0	16.7	16.1	15.7	15.5	15.0	14.8	14.2
80				17.0	17.0	16.4	15.3	14.6 *0.1*	14.4 *0.5*	14.2 *0.7*	14.0 *0.8*	13.6 *0.9*	13.4 *0.9*	12.4
90			17.0	16.8	15.6	14.7 *0.1*	14.2 *1.0*	13.8 *1.5*	13.5 *1.8*	13.3 *1.9*	13.1 *1.9*	12.6 *1.9*	12.2 *1.8*	10.4
100		17.0	16.6	15.1	14.4 *0.7*	14.1 *1.5*	13.6 *2.3*	13.2 *2.7*	12.6 *3.2*	12.2 *3.4*	11.8 *3.4*	11.1 *3.2*	10.6 *2.9*	8.4
110		17.0	15.1	14.4 *1.0*	14.0 *2.0*	13.6 *2.7*	12.9 *3.9*	12.2 *4.5*	11.6 *4.7*	11.1 *4.7*	10.7 *4.6*	10.0 *4.2*	9.5 *3.8*	6.9
120	17.0	15.5	14.4 *0.9*	14.0 *2.1*	13.6 *3.2*	13.0 *4.5*	12.1 *5.5*	11.4 *5.9*	10.8 *5.9*	10.4 *5.8*	9.9 *5.6*	9.2 *5.0*	8.6 *4.4*	5.8
130	16.6	14.6 *0.3*	14.1 *2.0*	13.6 *3.3*	13.0 *4.9*	12.4 *6.0*	11.5 *6.8*	10.8 *6.9*	10.2 *6.8*	9.7 *6.6*	9.3 *6.3*	8.5 *5.6*	7.9 *4.9*	5.0
140	15.5	14.3 *1.4*	13.9 *2.9*	13.2 *4.9*	12.5 *6.3*	11.9 *7.2*	11.1 *7.7*	10.4 *7.9*	9.8 *7.6*	9.3 *7.3*	8.8 *6.9*	8.0 *6.0*	7.4 *5.3*	4.3
150	14.6 *0.2*	14.1 *2.2*	13.5 *4.3*	12.7 *6.2*	12.1 *7.4*	11.5 *8.1*	10.7 *8.5*	10.0 *8.5*	9.4 *8.2*	8.9 *7.8*	8.4 *7.4*	7.6 *6.4*	7.0 *5.6*	3.7
160	14.4 *1.1*	13.9 *3.1*	13.1 *5.6*	12.4 *7.3*	11.8 *8.3*	11.2 *8.9*	10.4 *9.2*	9.7 *9.0*	9.1 *8.7*	8.5 *8.2*	8.1 *7.8*	7.2 *6.7*		3.2
170	14.3 *1.8*	13.6 *4.4*	12.8 *6.7*	12.1 *8.1*	11.5 *9.1*	11.0 *9.6*	10.3 *9.7*	9.4 *9.5*	8.8 *9.1*	8.3 *8.6*	7.8 *8.1*			2.9
180	14.1 *2.5*	13.3 *5.5*	12.5 *7.6*	11.9 *8.9*	11.3 *9.8*	10.8 *10.1*	9.9 *10.2*	9.2 *9.9*	8.6 *9.4*	8.0 *8.9*	7.6 *8.3*			2.6
200	13.6 *4.4*	12.8 *7.2*	12.1 *9.0*	11.5 *10.1*	11.0 *10.7*	10.4 *11.0*	9.6 *10.9*	8.9 *10.5*	8.2 *9.9*					2.1
220	13.3 *6.1*	12.5 *8.5*	11.9 *10.0*	11.3 *10.9*	10.7 *11.4*	10.2 *11.6*	9.3 *11.4*	8.6 *10.9*						1.7
240	13.0 *7.3*	12.3 *9.5*	11.6 *10.8*	11.1 *11.6*	10.5 *12.0*	10.0 *12.1*								1.4
260	12.7 *8.3*	12.1 *10.2*	11.5 *11.4*	10.9 *12.1*	10.4 *12.4*	9.9 *12.5*								1.2
280	12.6 *9.0*	11.9 *10.8*	11.3 *11.9*	10.8 *12.5*										

Girders so proportioned that the computed shear is less than that given in right-hand column do not require intermediate stiffeners.

* For single angle stiffeners, multiply by 1.8; for single plate stiffeners, multiply by 2.4.

AISC TABLE 3-46—Steel of 46 ksi Yield Point

Shear Stress, ksi (Shown on 1st line)
Stiffener Area, % of Web Area (In italics, on 2nd line)

Slenderness ratios h/t: web depth to web thickness	Aspect ratios a/h: stiffener spacing to web depth 0.5	0.6	0.7	0.8	0.9	1.0	1.2	1.4	1.6	1.8	2.0	2.5	3.0	over 3
60							18.5	18.5	18.5	18.5	18.5	18.3	18.0	17.3
70					18.5	18.5	18.3	17.5	16.9	16.5	16.2	15.8 *0.2*	15.3 *0.3*	14.9
80				18.5	18.3	17.2	16.1 *0.1*	15.7 *0.7*	15.4 *1.0*	15.1 *1.2*	14.9 *1.3*	14.5 *1.3*	14.2 *1.2*	13.0
90			18.5	17.6	16.3	15.8 *0.1*	15.3 *1.6*	14.8 *2.0*	14.5 *2.2*	14.2 *2.3*	13.8 *2.5*	13.1 *2.5*	12.6 *2.3*	10.4
100		18.5	17.4	16.0 *0.3*	15.6 *1.3*	15.2 *2.1*	14.6 *2.9*	13.9 *3.6*	13.3 *4.0*	12.8 *4.0*	12.4 *4.0*	11.6 *3.7*	11.0 *3.3*	8.4
110	18.5	17.8	16.0 *0.3*	15.5 *1.6*	15.1 *2.6*	14.6 *3.6*	13.6 *4.8*	12.9 *5.3*	12.3 *5.4*	11.7 *5.3*	11.3 *5.2*	10.5 *4.6*	9.9 *4.1*	6.9
120	18.5	16.3	15.6 *1.5*	15.1 *2.7*	14.5 *4.2*	13.8 *5.4*	12.9 *6.2*	12.1 *6.5*	11.5 *6.5*	10.9 *6.3*	10.5 *6.0*	9.6 *5.3*	9.0 *4.7*	5.8
130	17.4	15.8 *1.0*	15.3 *2.5*	14.6 *4.3*	13.9 *5.8*	13.2 *6.7*	12.3 *7.4*	11.5 *7.5*	10.9 *7.3*	10.3 *7.0*	9.9 *6.7*	9.0 *5.9*	8.3 *5.1*	5.0
140	16.2	15.5 *1.9*	14.9 *3.8*	14.1 *5.8*	13.4 *7.0*	12.8 *7.8*	11.8 *8.3*	11.1 *8.2*	10.4 *8.0*	9.8 *7.6*	9.4 *7.2*	8.4 *6.3*	7.8 *5.5*	4.3
150	15.9 *0.8*	15.3 *2.8*	14.5 *5.3*	13.7 *7.0*	13.0 *8.0*	12.4 *8.7*	11.5 *9.0*	10.7 *8.9*	10.0 *8.5*	9.4 *8.1*	9.0 *7.7*	8.0 *6.7*	7.4 *5.8*	3.7
160	15.7 *1.7*	14.9 *4.1*	14.1 *6.4*	13.4 *7.9*	12.7 *8.9*	12.1 *9.4*	11.2 *9.6*	10.4 *9.4*	9.7 *9.0*	9.1 *8.5*	8.6 *8.0*	7.7 *6.9*		3.2
170	15.5 *2.4*	14.6 *5.9*	13.8 *7.4*	13.1 *8.7*	12.4 *9.6*	11.8 *10.0*	10.9 *10.1*	10.1 *9.8*	9.4 *9.3*	8.9 *8.8*	8.4 *8.3*			2.9
180	15.2 *3.2*	14.3 *6.3*	13.5 *8.2*	12.8 *9.4*	12.2 *10.1*	11.6 *10.5*	10.7 *10.5*	9.9 *10.1*	9.2 *9.6*	8.6 *9.1*	8.1 *8.5*			2.6
200	14.7 *5.3*	13.9 *7.9*	13.1 *9.5*	12.5 *10.5*	11.9 *11.0*	11.3 *11.3*	10.4 *11.2*	9.6 *10.7*	8.9 *10.1*					2.1
220	14.3 *6.7*	13.6 *9.0*	12.9 *10.4*	12.2 *11.3*	11.6 *11.7*	11.1 *11.9*	10.1 *11.6*	9.3 *11.1*						1.7
240	14.0 *7.9*	13.3 *9.9*	12.6 *11.2*	12.0 *11.9*	11.4 *12.2*	10.9 *12.3*								1.4
260	13.8 *8.8*	13.1 *10.6*	12.5 *11.7*	11.8 *12.3*	11.3 *12.6*	10.7 *12.7*								1.2

Girders so proportioned that the computed shear is less than that given in right-hand column do not require intermediate stiffeners.

* For single angle stiffeners, multiply by 1.8; for single plate stiffeners, multiply by 2.4.

ALLOWABLE SHEAR STRESSES IN PLATE GIRDERS, KSI
And Required Gross Area of Pairs of Intermediate Stiffeners

AISC TABLE 3-50—Steel of 50 ksi Yield Point

Shear Stress, ksi (Shown on 1st line)
Stiffener Area, % of Web Area (In italics, on 2nd line)

Slenderness ratios h/t: web depth to web thickness; Aspect ratios a/h: stiffener spacing to web depth

h/t	0.5	0.6	0.7	0.8	0.9	1.0	1.2	1.4	1.6	1.8	2.0	2.5	3.0	over 3
60							20.0	20.0	20.0	20.0	19.7	19.1	18.8	18.1
70					20.0	20.0	19.1	18.2	17.6	17.3 *0.2*	17.1 *0.4*	16.7 *0.6*	16.5 *0.6*	15.5
80				20.0	19.1	17.9	17.1 *0.6*	16.7 *1.2*	16.4 *1.4*	16.1 *1.6*	15.8 *1.6*	15.4 *1.6*	15.0 *1.5*	13.1
90			20.0	18.3	17.3 *0.4*	16.9 *1.3*	16.3 *2.1*	15.8 *2.5*	15.3 *2.8*	14.8 *3.1*	14.3 *3.1*	13.6 *3.0*	13.0 *2.8*	10.4
100		20.0	18.1	17.2 *0.9*	16.7 *1.9*	16.3 *2.6*	15.4 *3.8*	14.6 *4.4*	13.9 *4.6*	13.4 *4.6*	12.9 *4.5*	12.0 *4.1*	11.4 *3.7*	8.4
110	20.0	18.5	17.2 *0.9*	16.7 *2.2*	16.2 *3.2*	15.4 *4.5*	14.4 *5.5*	13.6 *5.9*	12.9 *5.9*	12.3 *5.8*	11.8 *5.6*	10.9 *5.0*	10.3 *4.4*	6.9
120	19.7	17.4 *0.4*	16.8 *2.0*	16.2 *3.4*	15.4 *5.0*	14.7 *6.1*	13.7 *6.9*	12.8 *7.0*	12.1 *6.9*	11.5 *6.7*	11.0 *6.4*	10.1 *5.8*	9.4 *4.9*	5.8
130	18.2	17.0 *1.5*	16.4 *3.1*	15.6 *5.1*	14.8 *6.5*	14.1 *7.4*	13.1 *7.9*	12.2 *7.9*	11.5 *7.7*	10.9 *7.4*	10.4 *7.0*	9.4 *6.1*	8.7 *5.3*	5.0
140	17.3 *0.5*	16.7 *2.5*	15.9 *4.7*	15.1 *6.5*	14.3 *7.7*	13.6 *8.4*	12.6 *8.7*	11.8 *8.6*	11.0 *8.3*	10.4 *7.9*	9.9 *7.5*	8.9 *6.5*	8.2 *5.7*	4.3
150	17.1 *1.4*	16.4 *3.6*	15.5 *6.0*	14.6 *7.6*	13.9 *8.6*	13.3 *9.2*	12.2 *9.4*	11.4 *9.2*	10.7 *8.8*	10.0 *8.4*	9.5 *7.9*	8.5 *6.8*	7.7 *5.9*	3.7
160	16.9 *2.2*	16.0 *4.9*	15.1 *7.1*	14.3 *8.5*	13.6 *9.4*	13.0 *9.8*	11.9 *9.9*	11.1 *9.7*	10.3 *9.2*	9.7 *8.7*	9.2 *8.2*	8.1 *7.1*		3.2
170	16.7 *2.9*	15.6 *6.0*	14.8 *8.0*	14.0 *9.2*	13.3 *10.0*	12.7 *10.4*	11.7 *10.4*	10.8 *10.0*	10.1 *9.6*	9.5 *9.0*	8.9 *8.5*			2.9
180	16.3 *4.1*	15.4 *7.0*	14.5 *8.8*	13.8 *10.0*	13.1 *10.5*	12.5 *10.9*	11.5 *10.8*	10.6 *10.4*	9.9 *9.8*	9.2 *9.3*	8.7 *8.7*			2.6
200	15.8 *5.9*	14.9 *8.4*	14.1 *9.9*	13.4 *10.8*	12.8 *11.4*	12.2 *11.6*	11.1 *11.4*	10.3 *10.9*	9.5 *10.3*					2.1
220	15.4 *7.3*	14.6 *9.5*	13.8 *10.8*	13.2 *11.6*	12.5 *12.0*	11.9 *12.1*	10.9 *11.8*	10.0 *11.3*						1.7
240	15.1 *8.3*	14.3 *10.3*	13.6 *11.5*	13.0 *12.1*	12.3 *12.4*	11.7 *12.5*								1.4

Girders so proportioned that the computed shear is less than that given in right-hand column do not require intermediate stiffeners.

* For single angle stiffeners, multiply by 1.8; For single plate stiffeners, multiply by 2.4.

—This and preceding tables for the ASTM specification steels presented here by courtesy of American Institute of Steel Construction.

Bethlehem TABLE 3-45—Steel of 45 ksi Yield Point

See Notes Below

Slenderness ratios h/t: web depth to web thickness; Aspect ratios a/h: stiffener spacing to web depth

h/t	0.5	0.6	0.7	0.8	0.9	1.0	1.2	1.3	1.4	1.5	1.6	1.8	2.0	2.5	3.0	over 3.0
60	18.00	18.00	18.00	18.00	18.00	18.00	18.00	18.00	18.00	18.00	18.00	18.00	18.00	17.95	17.65	16.96
70	18.00	18.00	18.00	18.00	18.00	18.00	17.93	17.47	17.09	16.79	16.53	16.13	15.84	15.45 0.1 0.1	15.25 0.2 0.3	14.54
80	18.00	18.00	18.00	18.00	17.92	16.82	15.69	15.44 0.2 0.3	15.27 0.5 0.6	15.11 0.7 0.9	14.97 0.9 1.1	14.73 1.1 1.3	14.53 1.2 1.4	14.14 1.2 1.5	13.88 1.2 1.4	12.72
90	18.00	18.00	18.00	17.19	15.93	15.33 0.6 0.7	14.85 1.5 1.8	14.64 1.7 2.1	14.45 1.9 2.4	14.27 2.0 2.5	14.11 2.1 2.6	13.82 2.2 2.7	13.50 2.3 2.9	12.81 2.3 2.9	12.34 2.2 2.8	10.27
100	18.00	18.00	17.00	15.54 0.1 0.1	15.13 1.2 1.5	14.76 2.0 2.5	14.23 2.7 3.4	13.89 3.1 3.9	13.55 3.4 4.3	13.25 3.6 4.5	12.97 3.8 4.7	12.49 3.9 4.8	12.09 3.9 4.8	11.33 3.6 4.5	10.81 3.2 4.1	8.31
110	18.00	17.38	15.54 0.1 0.1	15.09 1.5 1.8	14.67 2.4 3.0	14.20 3.3 4.2	13.30 4.6 5.7	12.91 4.9 6.1	12.57 5.1 6.4	12.25 5.2 6.5	11.96 5.2 6.5	11.46 5.2 6.5	11.04 5.0 6.3	10.24 4.5 5.7	9.68 4.0 5.0	6.87
120	18.00	15.93	15.17 1.3 1.7	14.71 2.6 3.2	14.11 3.9 4.9	13.46 5.1 6.4	12.56 6.1 7.6	12.17 6.3 7.8	11.82 6.4 7.9	11.49 6.4 8.0	11.20 6.3 7.9	10.68 6.2 7.7	10.24 5.9 7.4	9.41 5.2 6.5	8.82 4.6 5.8	5.77
130	17.06	15.35 0.8 1.0	14.85 2.4 3.0	14.22 4.0 5.1	13.52 5.6 6.9	12.89 6.6 8.2	11.98 7.2 9.0	11.59 7.3 9.2	11.23 7.3 9.2	10.91 7.3 9.1	10.60 7.2 9.0	10.08 6.9 8.6	9.63 6.6 8.2	8.76 5.8 7.2	8.15 5.1 6.3	4.92
140	15.84	15.08 1.8 2.2	14.50 3.6 4.5	13.72 5.6 6.9	13.04 6.8 8.6	12.43 7.7 9.6	11.53 8.1 10.2	11.13 8.2 10.2	10.77 8.1 10.2	10.44 8.0 10.0	10.13 7.9 9.8	9.59 7.5 9.4	9.13 7.1 8.9	8.25 6.2 7.8	7.61 5.4 6.8	4.24
150	15.40 0.7 0.9	14.85 2.6 3.3	14.06 5.0 6.3	13.31 6.8 8.5	12.66 7.9 9.9	12.06 8.6 10.7	11.16 8.9 11.1	10.76 8.9 11.1	10.40 8.8 11.0	10.06 8.6 10.8	9.75 8.4 10.5	9.20 8.0 10.0	8.74 7.6 9.5	7.83 6.6 8.2	7.18 5.7 7.1	3.70
160	15.19 1.5 1.9	14.52 3.8 4.8	13.70 6.2 7.8	12.98 7.8 9.7	12.14 8.7 10.9	11.76 9.3 11.6	10.86 9.5 11.9	10.46 9.4 11.8	10.09 9.3 11.6	9.75 9.1 11.4	9.44 8.9 11.1	8.89 8.4 10.5	8.41 7.9 9.9	7.49 6.9 8.6		3.25
170	15.01 2.2 2.8	14.19 5.1 6.3	13.40 7.2 9.0	12.71 8.6 10.8	12.08 9.4 11.8	11.51 9.9 12.4	10.61 10.0 12.5	10.21 9.9 12.4	9.84 9.7 12.1	9.50 9.5 11.9	9.18 9.3 11.6	8.62 8.7 10.9	8.14 8.2 10.3			2.88
180	14.82 3.0 3.8	13.91 6.1 7.6	13.15 8.1 10.1	12.48 9.3 11.6	11.86 10.0 12.5	11.30 10.4 13.0	10.40 10.4 13.0	10.00 10.3 12.8	9.62 10.1 12.6	9.28 9.8 12.3	8.96 9.6 12.0	8.40 9.0 11.3	7.92 8.5 10.6			2.57
200	14.10 5.1 6.3	13.48 7.7 9.6	12.76 9.4 11.7	12.12 10.4 13.0	11.52 11.0 13.7	10.97 11.2 14.0	10.07 11.1 13.9	9.66 10.9 13.6	9.29 10.6 13.3	8.94 10.4 12.9	8.62 10.1 12.6					2.08
220	13.92 6.6 8.2	13.15 8.9 11.1	12.47 10.3 12.9	11.85 11.2 14.0	11.27 11.6 14.5	10.73 11.8 14.8	9.83 11.6 14.5	9.42 11.3 14.2	9.04 11.1 13.8	8.70 10.7 13.4						1.72
240	13.63 7.7 9.7	12.91 9.8 12.3	12.25 11.1 13.8	11.64 11.8 14.7	11.06 12.2 15.2	10.55 12.3 15.3										1.44
260	13.41 8.6 10.8	12.71 10.5 13.1	12.08 11.6 14.5	11.49 12.3 15.3	10.93 12.6 15.7	10.40 12.6 15.8										1.23

Figures given in top horizontal line opposite each h/t value indicate allowable shear stresses F_v.

Figures given in second horizontal line indicate required gross area of pairs of intermediate stiffeners, as per cent of web area A_w, using 45 ksi yield-point steel for the stiffeners ($Y = 1.00$; $D = 1.00$).

Figures given in third horizontal line indicate required gross area of pairs of intermediate stiffeners, as per cent of web area A_w, using 36 ksi yield-point steel for the stiffeners ($Y = 1.25$; $D = 1.00$).

Girders so proportioned that the computed shear is less than that given in the extreme right-hand column do not require intermediate stiffeners.

For single angle stiffeners, multiply values in second and third horizontal lines by 1.8.

For single plate stiffeners, multiply values in second and third horizontal lines by 2.4.

—This and following tables for some of the proprietary steels presented here by courtesy of Bethlehem Steel Corp. (Similar tables have been developed by United States Steel Corp.)

ALLOWABLE SHEAR STRESSES IN PLATE GIRDERS, KSI
And Required Gross Area of Pairs of Intermediate Stiffeners

Bethlehem TABLE 3-50—Steel of 50 ksi Yield Point
See Notes Below

Slenderness ratios h/t: web depth to web thickness; Aspect ratios a/h: stiffener spacing to web depth

h/t	0.5	0.6	0.7	0.8	0.9	1.0	1.2	1.3	1.4	1.5	1.6	1.8	2.0	2.5	3.0	over 3.0
60	20.00	20.00	20.00	20.00	20.00	20.00	20.00	20.00	20.00	20.00	20.00	19.84	19.48	18.92	18.61	17.88
70	20.00	20.00	20.00	20.00	20.00	20.00	18.90	18.41	18.02	17.69	17.42	17.13 0.2 0.3	16.93 0.4 0.5	16.57 0.6 0.8	16.32 0.6 0.8	15.33
80	20.00	20.00	20.00	20.00	18.89	17.73	16.96 0.6 0.9	16.74 0.9 1.3	16.54 1.2 1.6	16.36 1.3 1.8	16.19 1.4 2.0	15.90 1.6 2.2	15.66 1.6 2.3	15.20 1.6 2.2	14.88 1.5 2.1	12.99
90	20.00	20.00	19.91	18.12	17.12 0.4 0.6	16.71 1.3 1.8	16.15 2.1 2.9	15.90 2.3 3.2	15.68 2.5 3.4	15.43 2.6 3.7	15.13 2.8 3.9	14.61 3.1 4.2	14.18 3.1 4.3	13.37 3.0 4.2	12.82 2.8 3.8	10.27
100	20.00	20.00	17.92	16.98 0.9 1.2	16.53 1.9 2.6	16.10 2.6 3.7	15.23 3.7 5.2	14.81 4.1 5.7	14.43 4.4 6.1	14.08 4.5 6.3	13.77 4.6 6.4	13.22 4.6 6.4	12.76 4.5 6.3	11.89 4.1 5.7	11.29 3.7 5.1	8.31
110	20.00	18.32	17.01 0.9 1.2	16.51 2.1 3.0	16.00 3.2 4.4	15.27 4.5 6.2	14.26 5.5 7.7	13.83 5.8 8.0	13.44 5.9 8.2	13.09 5.9 8.2	12.76 5.9 8.2	12.19 5.8 8.0	11.71 5.6 7.7	10.80 5.0 6.9	10.15 4.4 6.1	6.87
120	19.48	17.17 0.4 0.6	16.62 2.0 2.8	16.03 3.4 4.8	15.23 5.0 7.0	14.53 6.1 8.5	13.57 6.9 9.5	13.09 7.0 9.7	12.69 7.0 9.8	12.33 7.0 9.7	12.00 6.9 9.6	11.41 6.7 9.3	10.92 6.4 8.9	9.97 5.6 7.8	9.29 4.9 6.8	5.77
130	17.98	16.84 1.5 2.1	16.27 3.1 4.3	15.40 5.1 7.1	14.63 6.5 9.0	13.95 7.4 10.2	12.95 7.9 11.0	12.51 7.9 11.0	12.11 7.9 11.0	11.74 7.8 10.9	11.40 7.7 10.7	10.81 7.4 10.2	10.30 7.0 9.7	9.32 6.1 8.5	8.02 5.3 7.4	4.92
140	17.17 0.5 0.7	16.56 2.5 3.4	15.73 4.7 6.6	14.90 6.5 9.0	14.16 7.7 10.6	13.50 8.4 11.6	12.49 8.7 12.1	12.05 8.7 12.1	11.64 8.6 12.0	11.27 8.5 11.8	10.93 8.3 11.5	10.32 7.9 11.0	9.81 7.5 10.4	8.80 6.5 9.0	8.09 5.7 7.9	4.24
150	16.92 1.4 1.9	16.22 3.6 5.0	15.29 6.0 8.4	14.49 7.6 10.6	13.77 8.6 11.9	13.13 9.2 12.7	12.12 9.4 13.0	11.68 9.3 13.0	11.27 9.2 12.8	10.89 9.0 12.5	10.55 8.8 12.2	9.93 8.3 11.6	9.41 7.9 10.9	8.39 6.8 9.4	7.66 5.9 8.2	3.70
160	16.70 2.1 3.0	15.81 4.9 6.8	14.93 7.1 9.9	14.16 8.5 11.8	13.46 9.3 13.0	12.83 9.8 13.7	11.82 9.9 13.8	11.38 9.8 13.6	10.96 9.7 13.4	10.59 9.4 13.1	10.24 9.2 12.8	9.62 8.7 12.1	9.08 8.2 11.4	8.05 7.1 9.8		3.25
170	16.49 2.9 4.0	15.48 6.0 8.4	14.63 8.0 11.1	13.88 9.2 12.8	13.20 10.0 13.9	12.58 10.4 14.4	11.57 10.4 14.4	11.12 10.2 14.2	10.71 10.0 13.9	10.33 9.8 13.6	9.98 9.5 13.3	9.35 9.0 12.5	8.82 8.5 11.8			2.80
180	16.16 4.1 5.7	15.20 6.9 9.6	14.38 8.8 12.2	13.65 9.9 13.7	12.98 10.5 14.6	12.37 10.8 15.1	11.36 10.8 15.0	10.91 10.6 14.7	10.50 10.4 14.4	10.12 10.1 14.0	9.76 9.8 13.7	9.13 9.3 12.9	8.59 8.7 12.1			2.57
200	15.65 5.9 8.2	14.77 8.4 11.7	13.99 9.9 13.8	13.29 10.8 15.1	12.64 11.3 15.8	12.04 11.6 16.1	11.03 11.4 15.8	10.58 11.1 15.5	10.17 10.9 15.1	9.78 10.6 14.7	9.42 10.3 14.3					2.08
220	15.27 7.3 10.1	14.44 9.5 13.1	13.70 10.8 15.0	13.02 11.6 16.1	12.39 12.0 16.6	11.80 12.1 16.8	10.79 11.8 16.4	10.34 11.5 16.0	9.92 11.2 15.6	9.53 10.9 15.2						1.72
240	14.98 8.3 11.6	14.20 10.3 14.3	13.48 11.5 15.9	12.82 12.1 16.8	12.20 12.4 17.3	11.61 12.5 17.4										1.44

Figures given in top horizontal line opposite each h/t value indicate allowable shear stresses F_v.
Figures given in second horizontal line indicate required gross area of pairs of intermediate stiffeners, as per cent of web area A_w, using 50 ksi yield-point steel for the stiffeners ($Y = 1.00$; $D = 1.00$).
Figures given in third horizontal line indicate required gross area of pairs of intermediate stiffeners, as per cent of web area A_w, using 36 ksi yield-point steel for the stiffeners ($Y = 1.39$; $D = 1.00$).
Girders so proportioned that the computed shear is less than that given in the extreme right-hand column do not require intermediate stiffeners.
For single angle stiffeners, multiply values in second and third horizontal lines by 1.8.
For single plate stiffeners, multiply values in second and third horizontal lines by 2.4.

Bethlehem TABLE 3-55—Steel of 55 ksi Yield Point
See Notes Below

Slenderness ratios h/t: web depth to web thickness; Aspect ratios a/h: stiffener spacing to web depth

h/t	0.5	0.6	0.7	0.8	0.9	1.0	1.2	1.3	1.4	1.5	1.6	1.8	2.0	2.5	3.0	over 3.0
60	22.00	22.00	22.00	22.00	22.00	22.00	22.00	22.00	22.00	21.65	21.32	20.81	20.43	19.84	19.52	18.75
70	22.00	22.00	22.00	22.00	22.00	21.26	19.82	19.31	18.96 0.1 0.1	18.79 0.3 0.5	18.62 0.5 0.7	18.34 0.7 1.1	18.10 0.8 1.3	17.66 0.9 1.5	17.36 0.9 1.4	16.07
80	22.00	22.00	22.00	21.38	19.81	18.87 0.3 0.5	18.28 1.2 1.9	18.03 1.5 2.3	17.80 1.7 2.6	17.58 1.8 2.8	17.39 1.9 3.0	17.05 2.0 3.1	16.77 2.1 3.1	16.00 2.1 3.2	15.44 2.0 3.1	12.99
90	22.00	22.00	20.88	19.02	18.53 1.1 1.7	18.07 1.9 2.9	17.43 2.6 4.0	17.05 3.0 4.6	16.64 3.3 5.1	16.26 3.5 5.4	15.92 3.7 5.6	15.34 3.8 5.8	14.85 3.8 5.8	13.93 3.5 5.4	13.30 3.2 4.9	10.27
100	22.00	21.13	18.96 0.2 0.3	18.41 1.5 2.3	17.90 2.5 3.8	17.30 3.5 5.3	16.20 4.7 7.1	15.73 5.0 7.6	15.30 5.2 7.9	14.92 5.3 8.0	14.57 5.3 8.1	13.95 5.2 8.0	13.43 5.1 7.8	12.45 4.6 7.0	11.76 4.1 6.2	8.31
110	22.00	19.21	18.47 1.5 2.3	17.91 2.7 4.2	17.12 4.2 6.5	16.33 5.4 8.2	15.23 6.3 9.6	14.75 6.5 9.9	14.32 6.5 10.0	13.92 6.5 10.0	13.56 6.5 9.9	12.92 6.3 9.6	12.39 6.0 9.2	11.36 5.3 8.1	10.63 4.7 7.2	6.87
120	20.43	18.67 1.1 1.7	18.06 2.6 4.0	17.21 4.5 6.8	16.35 5.9 9.1	15.59 6.9 10.5	14.49 7.5 11.4	14.01 7.6 11.6	13.57 7.6 11.6	13.16 7.5 11.5	12.79 7.4 11.3	12.14 7.1 10.8	11.59 6.8 10.3	10.52 5.9 9.0	9.77 5.2 7.9	5.77
130	18.99 0.1 0.2	18.32 2.1 3.2	17.51 4.2 6.4	16.57 6.0 9.2	15.75 7.3 11.1	15.02 8.0 12.3	13.91 8.4 12.9	13.43 8.5 12.9	12.98 8.4 12.8	12.57 8.3 12.6	12.20 8.1 12.4	11.54 7.7 11.8	10.97 7.3 11.2	9.88 6.4 9.7	9.10 5.5 8.5	4.92
140	18.69 1.1 1.7	18.00 3.1 4.8	16.96 5.6 8.6	16.07 7.3 11.1	15.28 8.3 12.7	14.56 8.9 13.7	13.45 9.2 14.0	12.97 9.1 14.0	12.52 9.0 13.8	12.11 8.9 13.5	11.73 8.7 13.2	11.05 8.2 12.6	10.48 7.8 11.9	9.36 6.7 10.2	8.56 5.8 8.9	4.24
150	18.43 1.9 3.0	17.51 4.6 7.0	16.52 6.8 10.5	15.66 8.3 12.6	14.89 9.2 14.0	14.19 9.7 14.8	13.08 9.8 15.0	12.60 9.7 14.8	12.15 9.5 14.6	11.73 9.3 14.3	11.35 9.1 13.9	10.67 8.6 13.2	10.08 8.1 12.4	8.95 7.0 10.7	8.14 6.1 9.3	3.70
160	18.21 2.7 4.1	17.10 5.8 8.9	16.16 7.8 11.9	15.33 9.1 13.9	14.58 9.9 15.1	13.89 10.3 15.7	12.78 10.3 15.7	12.29 10.2 15.5	11.84 10.0 15.2	11.42 9.7 14.9	11.03 9.5 14.5	10.35 8.9 13.7	9.76 8.4 12.9	8.61 7.2 11.0		3.25
170	17.84 3.9 6.0	16.77 6.8 10.4	15.87 8.6 13.2	15.06 9.8 14.9	14.32 10.4 15.9	13.64 10.8 16.5	12.53 10.7 16.4	12.04 10.5 16.1	11.59 10.3 15.8	11.16 10.1 15.4	10.78 9.8 14.9	10.08 9.2 14.1	9.49 8.7 13.2			2.68
180	17.51 5.0 7.6	16.49 7.6 11.7	15.62 9.3 14.2	14.83 10.3 15.8	14.10 10.9 16.7	13.43 11.2 17.1	12.32 11.1 16.9	11.83 10.9 16.6	11.37 10.6 16.2	10.95 10.3 15.8	10.56 10.0 15.3	9.86 9.4 14.4	9.26 8.9 13.5			2.57
190	17.23 5.9 9.0	16.26 8.3 12.8	15.41 9.9 15.1	14.63 10.8 16.5	13.92 11.3 17.3	13.25 11.5 17.6	12.15 11.3 17.3	11.65 11.1 17.0	11.19 10.9 16.6	10.77 10.6 16.1	10.38 10.3 15.7	9.68 9.6 14.7				2.30
200	17.00 6.7 10.2	16.06 9.0 13.7	15.23 10.4 15.9	14.46 11.2 17.1	13.76 11.7 17.8	13.10 11.8 18.1	12.00 11.6 17.7	11.50 11.4 17.3	11.04 11.1 16.9	10.61 10.8 16.4	10.22 10.4 16.0					2.08
210	16.79 7.3 11.2	15.88 9.5 14.5	15.07 10.8 16.5	14.32 11.6 17.7	13.62 12.0 18.3	12.97 12.1 18.5	11.86 11.8 18.0	11.37 11.6 17.6	10.91 11.3 17.2	10.48 10.9 16.7						1.85
220	16.61 7.9 12.1	15.73 9.9 15.2	14.94 11.2 17.1	14.20 11.9 18.1	13.51 12.2 18.7	12.80 12.3 18.8	11.75 12.0 18.3	11.25 11.7 17.9	10.79 11.4 17.4	10.36 11.1 16.9						1.72

Figures given in top horizontal line opposite each h/t value indicate allowable shear stresses F_v.
Figures given in second horizontal line indicate required gross area of pairs of intermediate stiffeners, as per cent of web area A_w, using 55 ksi yield-point steel for the stiffeners ($Y = 1.00$; $D = 1.00$).
Figures given in third horizontal line indicate required gross area of pairs of intermediate stiffeners, as per cent of web area A_w, using 36 ksi yield-point steel for the stiffeners ($Y = 1.53$; $D = 1.00$).
Girders so proportioned that the computed shear is less than that given in the extreme right-hand column do not require intermediate stiffeners.
For single angle stiffeners, multiply values in second and third horizontal lines by 1.8.
For single plate stiffeners, multiply values in second and third horizontal lines by 2.4.

—Note that AISC and Bethlehem values for steel of 50 ksi yield vary only slightly. The Bethlehem table is included here for the additional values for area of stiffeners fabricated from A36 steel.

ALLOWABLE SHEAR STRESSES IN PLATE GIRDERS, KSI
And Required Gross Area of Pairs of Intermediate Stiffeners

Bethlehem TABLE 3-60—Steel of 60 ksi Yield Point
See Notes Below

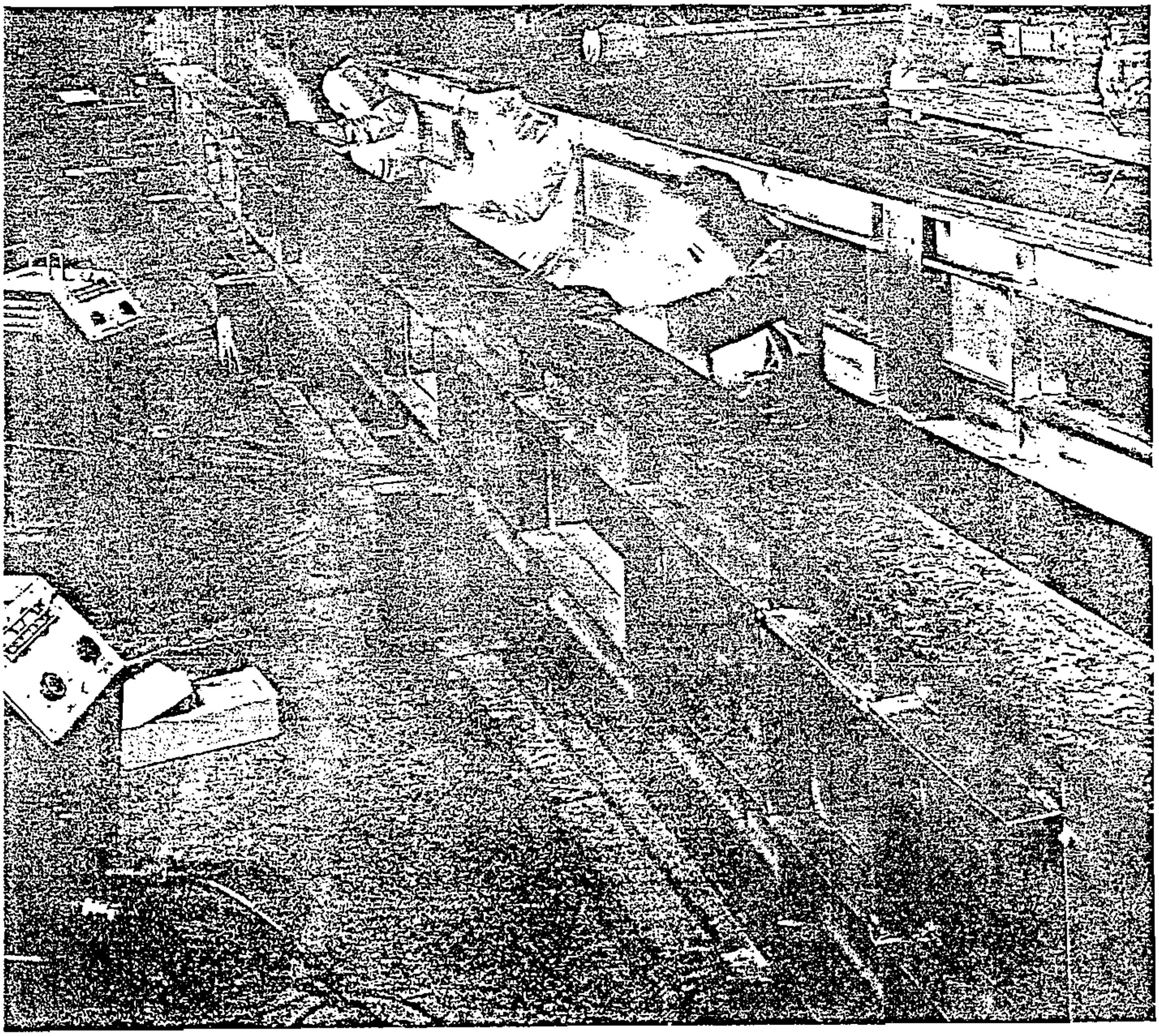
Slenderness ratios h/t: web depth to web thickness	Aspect ratios a/h: stiffener spacing to web depth																
h/t	0.4	0.5	0.6	0.7	0.8	0.9	1.0	1.2	1.3	1.4	1.5	1.6	1.8	2.0	2.5	3.0	over 3.0
60	24.00	24.00	24.00	24.00	24.00	24.00	24.00	24.00	23.53	23.03	22.61	22.27	21.73	21.34	20.74	20.49 0.1 0.2	19.59
70	24.00	24.00	24.00	24.00	24.00	23.64	22.20	20.73	20.48 0.4 0.6	20.25 0.6 1.1	20.05 0.8 1.4	19.86 1.0 1.6	19.53 1.2 1.9	19.25 1.3 2.1	18.73 1.3 2.1	18.38 1.2 2.0	16.79
80	24.00	24.00	24.00	24.00	22.33	20.74 0.1 0.1	20.25 0.9 1.6	19.59 1.8 3.0	19.30 2.0 3.4	19.03 2.2 3.7	18.79 2.3 3.8	18.57 2.4 4.0	18.01 2.6 4.3	17.50 2.7 4.5	16.56 2.7 4.5	15.91 2.5 4.1	12.99
90	24.00	24.00	24.00	21.81	20.47 0.7 1.1	19.92 1.7 2.8	19.42 2.5 4.1	18.48 3.5 5.8	17.97 3.9 6.4	17.51 4.1 6.9	17.10 4.3 7.2	16.72 4.4 7.3	16.07 4.4 7.4	15.52 4.4 7.3	14.49 4.0 6.7	13.77 3.6 6.0	10.27
100	24.00	24.00	22.07	20.44 0.8 1.4	19.83 2.1 3.5	19.25 3.1 5.1	18.37 4.4 7.3	17.16 5.4 9.1	16.65 5.7 9.5	16.18 5.8 9.7	15.75 5.9 9.8	15.36 5.9 9.8	14.68 5.7 9.6	14.11 5.5 9.2	13.01 4.9 8.2	12.24 4.4 7.3	8.31
110	24.00	23.28	20.58 0.5 0.8	19.92 2.1 3.5	19.20 3.5 5.9	18.24 5.1 8.5	17.39 6.2 10.3	16.19 6.9 11.5	15.67 7.0 11.7	15.19 7.1 11.8	14.76 7.0 11.7	14.36 7.0 11.6	13.66 6.7 11.2	13.06 6.4 10.7	11.91 5.6 9.4	11.11 4.9 8.2	6.87
120	24.00	21.34	20.16 1.7 2.8	19.42 3.3 5.6	18.38 5.4 8.9	17.47 6.7 11.1	16.66 7.5 12.5	15.45 8.0 13.4	14.92 8.1 13.4	14.44 8.0 13.4	14.00 7.9 13.2	13.59 7.8 13.0	12.87 7.4 12.4	12.26 7.1 11.8	11.08 6.2 10.3	10.24 5.4 9.0	5.77
130	23.92	20.53 0.7 1.2	19.80 2.7 4.4	18.74 5.1 8.4	17.75 6.8 11.3	16.87 7.9 13.2	16.08 8.6 14.3	14.87 8.9 14.8	14.34 8.9 14.8	13.86 8.8 14.6	13.41 8.6 14.4	13.00 8.4 14.1	12.27 8.0 13.4	11.64 7.6 12.6	10.44 6.6 10.9	9.57 5.7 9.5	4.92
140	22.21	20.21 1.6 2.7	19.29 4.1 6.8	18.19 6.4 10.7	17.24 7.9 13.2	16.39 8.9 14.8	15.62 9.4 15.7	14.42 9.6 16.0	13.88 9.5 15.8	13.39 9.4 15.6	12.94 9.2 15.3	12.52 8.9 14.9	11.78 8.5 14.1	11.15 8.0 13.3	9.92 6.9 11.5	9.04 6.0 10.0	4.24
150	20.75	19.94 2.5 4.1	18.60 5.4 9.0	17.76 7.5 12.5	16.84 8.6 14.7	16.01 9.6 16.1	15.25 10.1 16.8	14.05 10.1 16.9	13.51 10.0 16.7	13.02 9.8 16.4	12.56 9.6 16.0	12.14 9.4 15.6	11.40 8.8 14.7	10.75 8.3 13.9	9.51 7.2 11.9	8.61 6.2 10.4	3.70
160	20.51 0.8 1.3	19.58 3.6 5.9	18.39 6.5 10.9	17.40 8.4 14.0	16.51 9.6 16.0	15.70 10.3 17.1	14.95 10.6 17.7	13.75 10.6 17.7	13.21 10.4 17.4	12.71 10.2 17.0	12.26 10.0 16.6	11.83 9.7 16.2	11.08 9.1 15.2	10.43 8.6 14.3	9.17 7.4 12.3		3.25
170	20.28 1.5 2.5	19.18 4.7 7.9	18.06 7.4 12.4	17.10 9.2 15.3	16.23 10.2 17.0	15.44 10.8 18.0	14.70 11.1 18.5	13.50 11.0 18.3	12.96 10.8 18.0	12.46 10.5 17.6	12.00 10.3 17.1	11.57 10.0 16.6	10.81 9.4 15.7	10.16 8.8 14.7			2.88
180	20.09 2.1 3.5	18.86 5.7 9.5	17.78 8.2 13.7	16.85 9.8 16.3	16.00 10.7 17.9	15.22 11.2 18.7	14.49 11.5 19.1	13.29 11.3 18.8	12.75 11.1 18.5	12.25 10.8 18.0	11.78 10.5 17.5	11.36 10.2 17.0	10.59 9.6 16.0	9.94 9.0 15.0			2.57
190	19.87 2.8 4.7	18.59 6.5 10.9	17.55 8.9 14.8	16.64 10.3 17.2	15.81 11.2 18.6	15.03 11.6 19.4	14.32 11.8 19.7	13.11 11.6 19.3	12.57 11.3 18.9	12.07 11.0 18.4	11.60 10.7 17.9	11.17 10.4 17.4	10.41 9.8 16.3	9.74 9.2 15.3			2.30
200	19.56 3.8 6.3	18.34 7.2 12.1	17.35 9.4 15.7	16.46 10.8 17.9	15.64 11.5 19.2	14.88 11.9 19.9	14.17 12.1 20.1	12.96 11.8 19.7	12.42 11.5 19.2	11.91 11.2 18.7	11.45 10.9 18.2	11.02 10.6 17.6					2.08

Figures given in top horizontal line opposite each h/t value indicate allowable shear stresses F_v.
Figures given in second horizontal line indicate required gross area of pairs of intermediate stiffeners, as per cent of web area A_w, using 60 ksi yield-point steel for the stiffeners ($Y = 1.00$; $D = 1.00$).
Figures given in third horizontal line indicate required gross area of pairs of intermediate stiffeners, as per cent of web area A_w, using 36 ksi yield-point steel for the stiffeners ($Y = 1.67$; $D = 1.00$).
Girders so proportioned that the computed shear is less than that given in the extreme right-hand column do not require intermediate stiffeners.
For single-angle stiffeners, multiply values in second and third horizontal lines by 1.8.
For single plate stiffeners, multiply values in second and third horizontal lines by 2.4.

Bethlehem TABLE 3-65—Steel of 65 ksi Yield Point
See Notes Below

Slenderness ratios h/t: web depth to web thickness	Aspect ratios a/h: stiffener spacing to web depth																
h/t	0.4	0.5	0.6	0.7	0.8	0.9	1.0	1.2	1.3	1.4	1.5	1.6	1.8	2.0	2.5	3.0	over 3.0
50	26.00	26.00	26.00	26.00	26.00	26.00	26.00	26.00	26.00	26.00	26.00	26.00	26.00	26.00	25.89	25.46	24.46
60	26.00	26.00	26.00	26.00	26.00	26.00	26.00	25.13	24.49	23.97	23.54	23.18	22.62	22.32 0.1 0.2	21.87 0.4 0.7	21.57 0.4 0.8	20.39
70	26.00	26.00	26.00	26.00	26.00	24.61	23.11	22.07 0.6 1.1	21.79 0.9 1.6	21.53 1.1 2.0	21.29 1.3 2.3	21.08 1.4 2.6	20.70 1.6 2.8	20.38 1.6 2.9	19.78 1.6 2.9	19.37 1.5 2.7	16.97
80	26.00	26.00	26.00	25.54	23.25	22.15 0.6 1.1	21.62 1.5 2.7	20.88 2.3 4.1	20.55 2.5 4.5	20.25 2.6 4.7	19.81 2.9 5.2	19.42 3.1 5.6	18.74 3.3 5.9	18.17 3.3 6.0	17.11 3.2 5.7	16.39 2.9 5.2	12.99
90	26.00	26.00	25.52	22.70	21.90 1.2 2.2	21.31 2.2 4.0	20.74 3.0 5.3	19.44 4.3 7.7	18.88 4.6 8.3	18.38 4.8 8.7	17.93 4.9 8.9	17.52 5.0 9.0	16.80 5.0 9.0	16.19 4.8 8.7	15.05 4.4 7.9	14.25 3.9 7.0	10.27
100	26.00	26.00	22.97	21.90 1.4 2.5	21.24 2.6 4.7	20.37 4.0 7.2	19.43 5.2 9.3	18.12 6.1 11.0	17.56 6.3 11.4	17.05 6.4 11.5	16.59 6.4 11.5	16.16 6.3 11.5	15.41 6.2 11.1	14.78 5.9 10.7	13.57 5.2 9.5	12.71 4.6 8.3	8.31
110	26.00	24.23	22.08 1.0 1.9	21.36 2.6 4.7	20.37 4.4 8.0	19.36 5.9 10.6	18.46 6.8 12.3	17.15 7.4 13.4	16.58 7.5 13.6	16.07 7.5 13.6	15.59 7.5 13.5	15.15 7.4 13.3	14.39 7.1 12.8	13.73 6.7 12.1	12.47 5.9 10.6	11.58 5.1 9.3	6.87
120	26.00	22.43 0.2 0.3	21.64 2.2 3.9	20.66 4.2 7.6	19.56 6.1 11.0	18.59 7.3 13.2	17.72 8.1 14.6	16.41 8.5 15.3	15.84 8.5 15.3	15.31 8.4 15.2	14.83 8.3 15.0	14.39 8.1 14.7	13.61 7.7 14.0	12.94 7.3 13.3	11.64 6.4 11.5	10.72 5.6 10.0	5.77
130	24.09	22.05 1.2 2.2	21.18 3.3 6.0	19.97 5.8 10.5	18.92 7.4 13.4	17.99 8.4 15.2	17.14 9.0 16.3	15.84 9.3 16.8	15.26 9.2 16.7	14.73 9.1 16.4	14.24 8.9 16.1	13.79 8.7 15.7	13.00 8.3 14.9	12.32 7.8 14.1	10.99 6.8 12.2	10.05 5.9 10.6	4.92
140	23.11	21.72 2.1 3.8	20.58 4.9 8.8	19.43 7.1 12.8	18.42 8.5 15.3	17.51 9.3 16.8	16.69 9.8 17.7	15.38 9.9 17.9	14.80 9.8 17.7	14.27 9.6 17.4	13.78 9.4 17.0	13.32 9.2 16.6	12.52 8.7 15.7	11.82 8.2 14.8	10.48 7.1 12.7	9.52 6.1 11.1	4.24
150	22.31 0.5 0.9	21.39 3.0 5.5	20.09 6.1 11.1	18.99 8.1 14.6	18.01 9.3 16.8	17.13 10.0 18.1	16.32 10.4 18.8	15.01 10.4 18.8	14.43 10.3 18.6	13.89 10.1 18.2	13.40 9.8 17.8	12.94 9.6 17.3	12.13 9.0 16.3	11.43 8.5 15.3	10.07 7.3 13.2	9.09 6.3 11.4	3.70
160	22.05 1.3 2.3	20.92 4.3 7.8	19.68 7.1 12.9	18.63 8.9 16.1	17.68 10.0 18.1	16.82 10.6 19.2	16.02 10.9 19.8	14.71 10.9 19.6	14.13 10.7 19.3	13.59 10.4 18.8	13.09 10.2 18.4	12.63 9.9 17.9	11.81 9.3 16.8	11.10 8.7 15.8	9.73 7.5 13.5		3.25
170	21.83 1.9 3.5	20.53 5.4 9.8	19.35 8.0 14.4	18.33 9.6 17.3	17.41 10.6 19.1	16.56 11.1 20.1	15.77 11.4 20.5	14.46 11.2 20.2	13.88 11.0 19.8	13.33 10.7 19.4	12.83 10.4 18.9	12.37 10.1 18.3	11.54 9.5 17.2	10.83 9.0 16.2			2.88
180	21.62 2.5 4.6	20.20 6.3 11.4	19.07 8.7 15.7	18.08 10.2 18.4	17.18 11.1 20.0	16.34 11.5 20.8	15.56 11.7 21.2	14.25 11.5 20.8	13.66 11.3 20.3	13.12 11.0 19.8	12.62 10.7 19.3	12.15 10.4 18.7	11.32 9.7 17.6	10.61 9.1 16.5			2.57
190	21.26 3.6 6.4	19.92 7.1 12.8	18.84 9.3 16.8	17.87 10.7 19.3	16.98 11.5 20.7	16.15 11.9 21.4	15.38 12.0 21.7	14.07 11.7 21.2	13.49 11.5 20.7	12.94 11.2 20.2	12.44 10.9 19.6	11.97 10.6 19.0	11.14 9.9 17.9				2.30

Figures given in top horizontal line opposite each h/t value indicate allowable shear stresses F_v.
Figures given in second horizontal line indicate required gross area of pairs of intermediate stiffeners, as per cent of web area A_w, using 65 ksi yield-point steel for the stiffeners ($Y = 1.00$; $D = 1.00$).
Figures given in third horizontal line indicate required gross area of pairs of intermediate stiffeners, as per cent of web area A_w, using 36 ksi yield-point steel for the stiffeners ($Y = 1.81$; $D = 1.00$).
Girders so proportioned that the computed shear is less than that given in the extreme right-hand column do not require intermediate stiffeners.
For single angle stiffeners, multiply values in second and third horizontal lines by 1.8.
For single plate stiffeners, multiply values in second and third horizontal lines by 2.4.



Access holes cut in girder web must be reinforced. In regions of high bending moment, flanges must extend far enough beyond web opening to effectively transfer forces into main web of girder. Semi-automatic welding, with self-shielding cored electrode wire, is used here in attaching reinforcements at double the speed of manual welding.