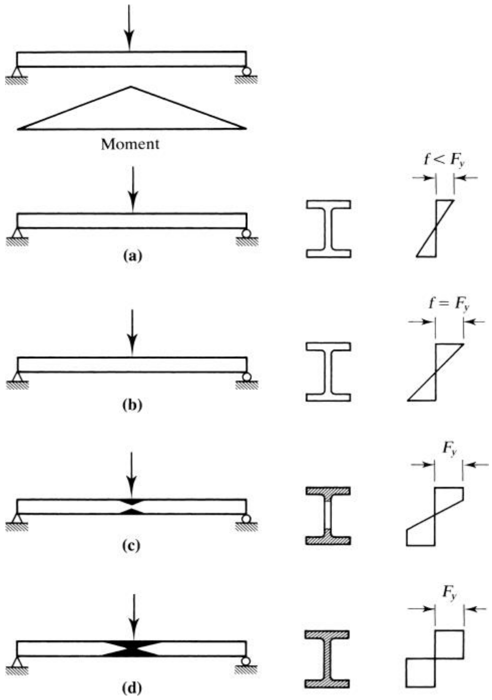
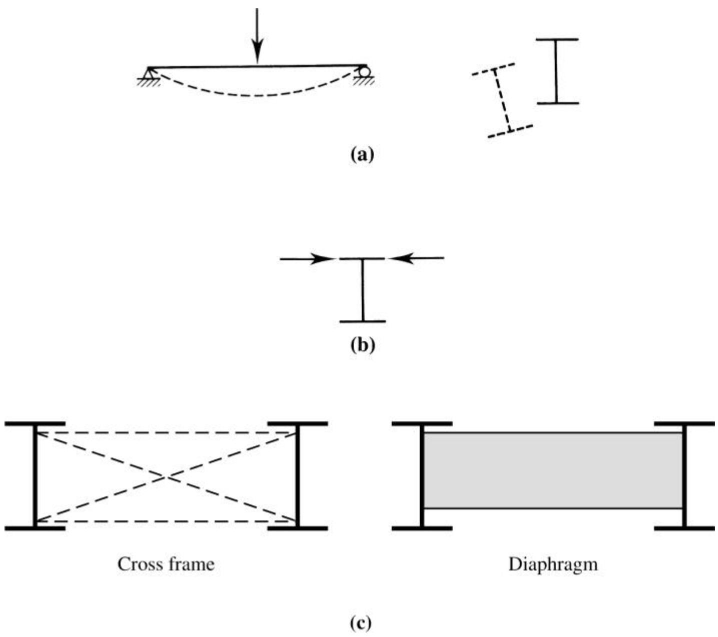
Composite Section

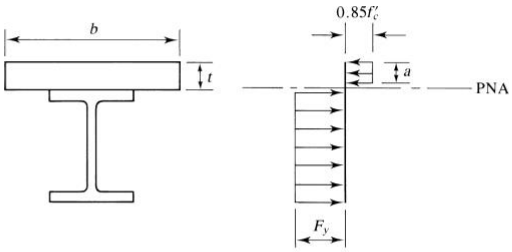
1. My and Mp



2. Lateral-torsional buckling and lateral bracing



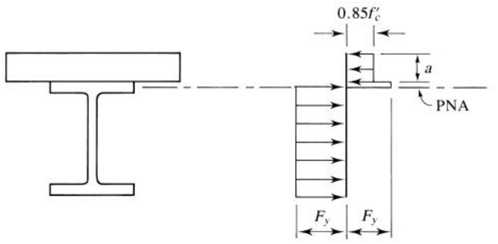
★ stress distribution when a composite beam has reached the plastic limit state



(a)

-full tensile yielding of steel and partial compression of the concrete

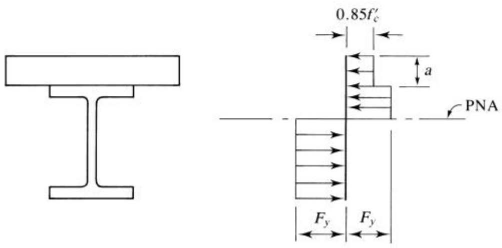
-PNA is in the slab.



(b)

-the concrete stress block extends the full depth of the slab

-PNA is in the flange of the steel shape.



(c)

-PNA is in the web.

To determine which of the three cases governs, compute the compressive resultants as the smallest of

① $A_s F_y$

② $0.85 f_c' A_c$

③ $\sum Q_n$

A_s cross-sectional area of steel shape

A_c area of concrete= tb (see Fig.(a))

$\sum Q_n$ total shear strength of the shear connectors

When ① controls, the steel is being fully utilized. □stress distribution (a) normal case for full composite behavior exists.

When ② controls, the concrete is being fully utilized. stress distribution (b) or (c)

When ③ controls, partial composite behavior

Composite Section Strength for LRFD Method

Classification of Composite Sections

1. Composite Sections in Positive Flexure

- Composite sections in kinked continuous or horizontally curved steel girder bridges shall be considered as noncompact sections
- Composite sections in straight bridges that satisfy the following requirements shall qualify as compact sections
  - The specified minimum yield strengths of the flanges do not exceed 70.0ksi,
  - the web satisfies the requirement: $\frac{D}{t_w} \le 150$
  - the section satisfies the web slenderness limit: $\frac{2D_{cp}}{t_w} \le 3.76\sqrt{\frac{E}{F_{yc}}}$

D depth of the web

D_{cp} depth of the web in compression at the plastic moment assuming steel beam is fully yielded

F_{yc} compression flange steel yield stress (ksi)

2. Composite Sections in Negative Flexure and Noncomposite Sections

For composite sections in negative flexure and noncomposite sections, the provisions of Article 6.10.8 limit the nominal flexural resistance to be less than or equal to the moment at first yield. As a result, the nominal flexural resistance for these sections is conveniently expressed in terms of the elastically computed flange stress.

Flexural Resistance of Composite Sections in Positive Flexure

1. For compact sections

- **Strength limit state**

$$M_u + \frac{1}{3} f_l S_{xt} \le \phi_f M_n$$

ϕ_f the resistance factor for flexural (=1.0)

f_l flange lateral bending stress

M_n nominal flexure resistance of the composite section

M_u factored bending moment about the major axis of the composite section

S_{xt} elastic section modulus about the major axis of the section to the tension flange taken as M_{yt}/F_{yt}

M_{yt} yield moment with respect to the tension flange

- **Nominal flexural resistance**
  - If $D_p \le 0.1 D_t$, then $M_n = M_p$
  - Otherwise, $M_n = M_p \left(1.07 - 0.7 \frac{D_p}{D_t} \right)$

    D_p distance from the top of the concrete deck to the neutral axis of the composite section at plastic moment (in)

    D_t total depth of the composite section (in)

  - In continuous span, $M_n \le 1.3 R_h M_y$

    R_h hybrid factor

=1.0 if a higher-strength steel in the web than in both flanges

$= \frac{12+\beta\left(3\rho-\rho^{3}\right)}{12+2\beta}$, otherwise

$\beta = \frac{2D_n t_w}{A_{fn}}$

ρ = the smaller of F_{yw}/f_n and 1.0

f_{yw} specified minimum yield strength of a web

A_{fn} sum of the flange area and the area of any cover plates on the side of

the neutral axis corresponding to D_n. For composite sections in

negative flexure, the area of the longitudinal reinforcement may be

included in calculating A_{fn} for the top flange

D_n larger of the distances from the elastic neutral axis of the cross-

section

to the inside face of either flange. For sections where the neutral axis

is

at the mid-depth of the web, the distance from the neutral axis to the inside face of the flange on the side of the neutral axis where yielding occurs first.

f_n for sections where yielding occurs first in the flange, a cover plate or

the

longitudinal reinforcement on the side of the neutral axis

corresponding

to D_n the largest of the specified minimum yield strengths of each

component included in the calculation of A_{fn}. Otherwise, the largest

of

the elastic stresses in the flange, cover plate or longitudinal reinforcement on the side of the neutral axis corresponding to D_n at first yield on the opposite side of the neutral axis.

2. For Noncompact sections

- **Strength limit state for compression flange**

$$f_{bu} \le \phi_f F_{nc}$$

ϕ_f resistance factor for flexure (=1.0)

f_{bu} flange stress calculated without consideration of flange lateral bending

F_{nc} nominal flexural resistance of the compression flange

- **Strength limit state for tension flange**

$$f_{bu} + \frac{1}{3} f_l \le \phi_f F_{nt}$$

f_l flange lateral bending stress

F_{nt} nominal flexural resistance of the tension flange

- **Nominal flexural resistance of the compression flange**

$$F_{nc} = R_b R_h F_{yc}$$

- **Nominal flexural resistance of the tension flange**

$$F_{nc} = R_h F_{yt}$$

3. Ductility Requirement

Compact and noncompact sections shall satisfy $D_p = 0.42 D_t$ to ensure significant yielding of the bottom flange when the crushing strain is reached at the top of concrete deck.

Flexural Resistance of Composite Sections in Negative Flexure and Noncomposite Sections

- **Strength Limit State**

$$F_{bu} + \frac{1}{3} f_l = \phi_f F_{nc}$$

ϕ_f the resistance factor for flexure (=1.0)

f_{bu} flange stress calculated without consideration of flange lateral bending

f_l flange lateral bending stress

F_{nc} nominal flexural resistance of the flange

- **Application of Design Vehicular Live Loads**

AASHTO LRFD 3.6.1.3.1

Unless otherwise specified, the extreme force effect shall be taken as the larger of the following:

- The effect of the design tandem combined with the effect of the design lane load(=0.64klf), or
- The effect of one design truck with the variable axle spacing (14~30ft) combined with the effect of the design lane load, and
- For negative moment between points of contraflexure under a uniform load on all spans, and reaction at interior piers only, 90 percent of the effect of two design trucks spaced a minimum of 50.0ft between the lead axle of one truck and the rear axle of the other truck, combined with 90 percent of the effect of the design lane load. The distance between the 32.0-kip axles of each truck shall be taken as 14.0ft. The two design trucks shall be placed in adjacent spans to produce maximum force effects.

- **Local Buckling Resistance**

- If $\lambda_f \le \lambda_{pf}$, then $F_{nc} = R_b R_h F_{yc}$
- Otherwise $F_{nc} = \left[1 - \left(1 - \frac{F_{yc}}{R_h F_{yc}}\right)\left(\frac{\lambda_f - \lambda_{pf}}{\lambda_{rf} - \lambda_{pf}}\right)\right] R_b R_h F_{yc}$

F_{yc} yield stress of the compression flange steel

$\lambda_{f=}\frac{b_{fc}}{2t_{fc}}$ slenderness ratio for the compression flange

$\lambda_{pf}=0.38\sqrt{\frac{E}{F_{yc}}}$

$\lambda_{rf}=0.56\sqrt{\frac{E}{F_{yr}}}$ limiting slenderness ratio for a noncompact flange

R_b web load-shedding factor *AASHTO LRFD 6.10.1.10.2*

- $R_b=1.0$

  if the section is composite and is in positive flexure

  or longitudinal stiffeners are provided, and $\frac{D}{t_w}\le 0.95\sqrt{\frac{Ek}{F_{yc}}}$

  or web satisfy $\frac{2D_c}{t_w}\le\lambda_{rw}=5.7\sqrt{\frac{E}{F_y}}$

- Otherwise, $R_b=1-\left(\frac{a_{wc}}{1200+300a_{wc}}\right)\left(\frac{2D_c}{t_w}-\lambda_{rw}\right)\le 1.0$

  $\lambda_{rw}=5.7\sqrt{\frac{E}{F_y}}$: limiting slenderness ratio for a noncompact web

  $a_{wc}=\frac{2D_c t_w}{b_{fc}t_{fc}}$: ratio of two times the web area in compression to the

  area of the compression flange

  D web depth

  D_c depth of the web in compression in the elastic range

  k bend-buckling coefficient for webs

F_{yc} specified minimum yield strength of a compression flange

R_h hybrid factor <u>*AASHTO LRFD 6.10.1.10.1*</u>

 Lateral-Torsional Buckling Resistance of Compression Flange

 if $L_b \le L_p = 1.0 r_t \sqrt{\frac{E}{F_y}}$ then $F_{nc} = R_b R_h F_y$

 if $L_p \le L_b \le L_r$ then $F_{nc} = C_b \left[1 - \left(1 - \frac{F_{yr}}{R_h F_{yc}} \right) \left(\frac{L_b - L_p}{L_r - L_p} \right) \right] R_b F_y \le R_b F_y$

 if $L_b \ge L_r = \pi r_t \sqrt{\frac{E}{F_y}}$ then $F_{nc} = F_{cr} \le R_b F_y$

$$r_t = \frac{b_{fc}}{\sqrt{12\left(1 + \frac{1}{3}\frac{D_c t_w}{b_{fc} t_{fc}}\right)}}$$

$c_b = 1.0$, if $f_{mid} \ge f_2$, or $f_2 = 0$

Otherwise $c_b = 1.75 - 1.05\left(\frac{f_1}{f_2}\right) + 0.3\left(\frac{f_1}{f_2}\right)^2 \le 2.3$

$$F_{cr} = \frac{C_b R_b \pi^2 E}{\left(\frac{L_b}{r_t}\right)^2}$$

f_{mid} factored maximum stress at the middle of the unbraced compression flange (compression as positive)

f_2 factored maximum stress at either end of the unbraced compression flange (compression as positive) if it is in tension, $f_2 = 0$.

f_0 factored maximum stress at the braced point opposite to the one corresponding to f_2 (compression as positive)

$$f_1 = 2f_{mid} - f_2 \geq f_0$$

 Flexural Resistance of Tension Flange

$$F_{nt} = R_h F_{yt}$$

Shear Resistance of Composite Sections

- **Strength Limit State of straight and curved web panels**

$V_u \le \phi_v V_n$

V_n nominal shear resistance for unstiffened and stiffened webs

V_u factored shear in the web

ϕ_v the resistance factor for shear (=1.0)

- **Nominal resistance of unstiffened webs**

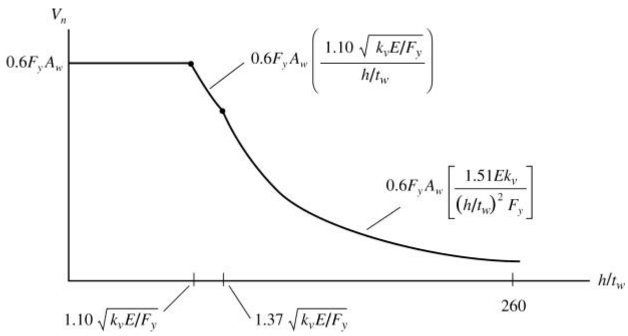
$V_n = CV_p$

$V_p = 0.58 F_y D t_w$

the constant C

- for $\frac{D}{t_w} \langle 1.12\sqrt{\frac{Ek}{F_y}}$,

$C = 1.0$



FIY, Steel Construction Design Manual (AISC, 2005)

- for $1.12\sqrt{\frac{Ek}{F_y}} \le \frac{D}{t_w} \langle 1.40\sqrt{\frac{Ek}{F_y}}$, $\quad C = \frac{1.12}{\left(\frac{D}{t_w}\right)}\sqrt{\frac{Ek}{F_y}}$

- for $\frac{D}{t_w} \ge 1.40\sqrt{\frac{Ek}{F_y}}$, $\quad C = \frac{1.57}{\left(\frac{D}{t_w}\right)^2}\left(\frac{Ek}{F_y}\right)$

$k = 5 + \left\lfloor 5/(d_0/D)^2 \right\rfloor$ buckling coefficient (k=5 for unstiffened beams)

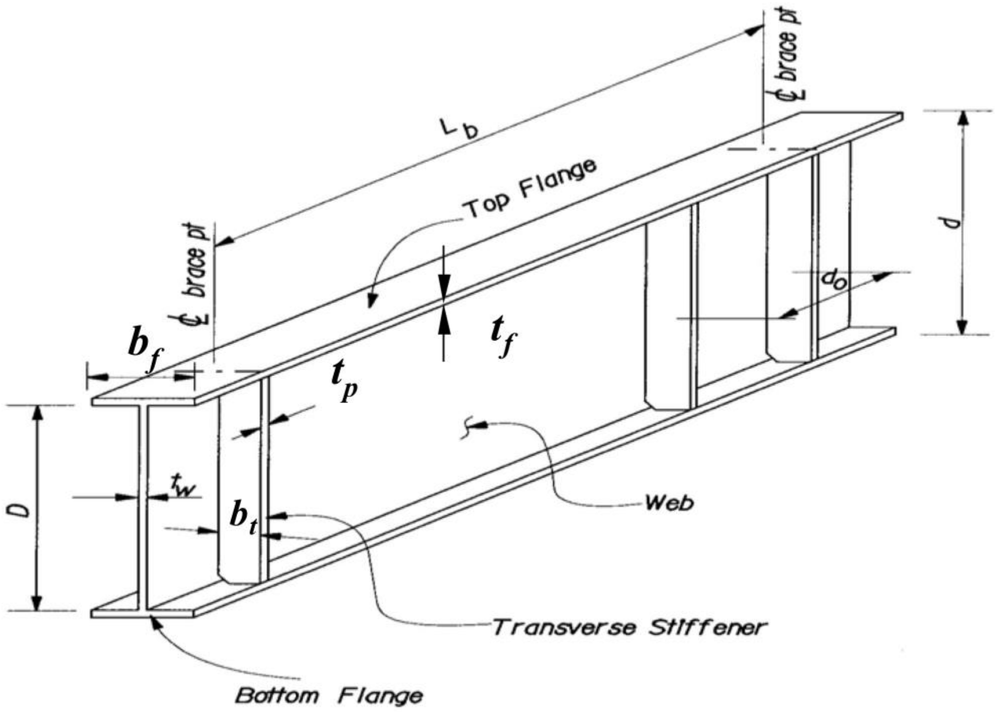
D clear distance between flanges

d_0 distance between transverse stiffeners

□ **Nominal resistance of stiffened webs**

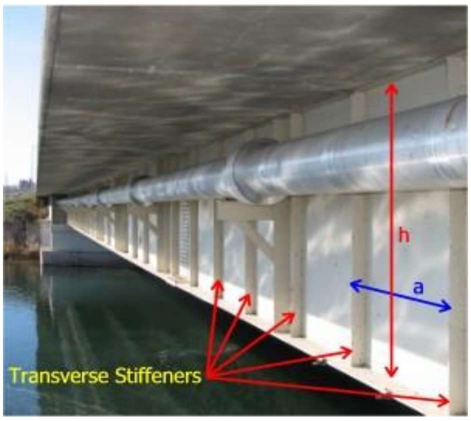
$$V_n = V_p\left(C + \frac{0.87(1-C)}{\sqrt{1+\left(d_0/D\right)^2} + d_0/D}\right)$$

□ **Transverse stiffener**



□ **Typical shear-dominant failure mode of stiffened web**

 Transverse stiffeners, longitudinal stiffeners, and bearing stiffeners




Transverse Web Stiffener
Bearing Stiffener
Jacking Stiffeners
29 1 2002

Design of Shear Connectors

How to design?

AASHTO requires that shear connectors be designed to account for fatigue and checked for ultimate strength. *Note that the ultimate strength method is not an alternative approach, but a required check*.

Fatigue Check

Fatigue is caused by the repetitive loading and unloading of a structural member.

- For fatigue load, use only one truck with a load factor of 0.75.
- Range of horizontal shear at slab-beam interface(k/in)

$$S_r = \frac{V_r Q}{I}$$

V_r range of shear due to live load plus impact, k

Q statical moment about neutral axis, in^3

I moment of inertia of composite beam, in^4

- Allowable range of horizontal shear for on one stud(k)

$$Z_r = \alpha \cdot d^2$$

$\alpha = 34.5 - 4.28 \log N$ constant based on number of stress cycles

d diameter of stud, in (5/8, 6/8, 7/8in, typically)

- Fatigue check

$$S_r \le Z_r$$

- Pitch of shear connectors

$$p = \frac{n \cdot Z_r}{S_r}$$

n number of shear connectors, in

Geometric Constraints

- Maximum pitch of shear connectors: 24 in
- Minimum spacing between shear connectors in a row: $s \ge 4 \cdot d$

- Minimum spacing between the edge of the stringer flange to the edge of the shear connector: 1 in
- Minimum concrete cover over the top of the shear connector: 2 in.
- The shear stud should extend at least 2 in above the bottom of the concrete deck slab.
- The ratio of the length of the connector to its diameter should not be less than 4.

Geometric Constraints

- Ultimate tensile strength of the steel stringer

$$P_1 = A_s F_y$$

A_s total area of steel stringer

F_y minimum yield point of steel used

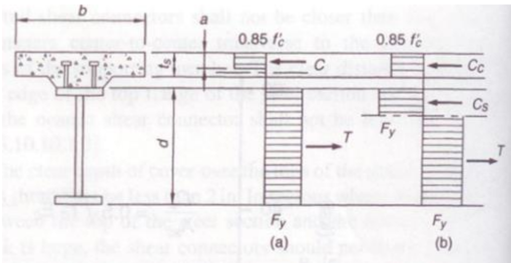


Fig. 19.58 Nominal horizontal shear force: (a) PNA in slab and (b) PNA in steel.

- Ultimate compressive strength of the concrete slab

$$P_2 = 0.85 f_c' b_{eff} t$$

- Ultimate strength of a single shear connector, kips

$$Q_r = \phi_{sc} Q_n$$
$$Q_n = 0.5A\sqrt{f_c' E_c} \leq AF_u$$

ϕ_{sc} resistance factor for shear conncetors= 0.85

A cross-sectional area of stud, in^2

E_c modulus of elasticity of concrete, ksi

F_u specified tensile strength of shear stud, ksi

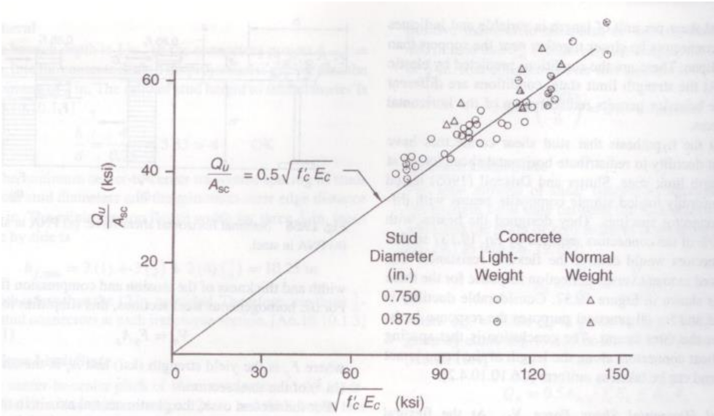


Fig. 19.55 Comparison of connector strength with concrete strength and modulus of elasticity (Ollgaard et al., 1971).

- Minimum number of shear connectors required between points of maximum positive moment and the adjacent end supports

$$n = \frac{P}{Q_r}$$

P smaller of P_1 and P_2

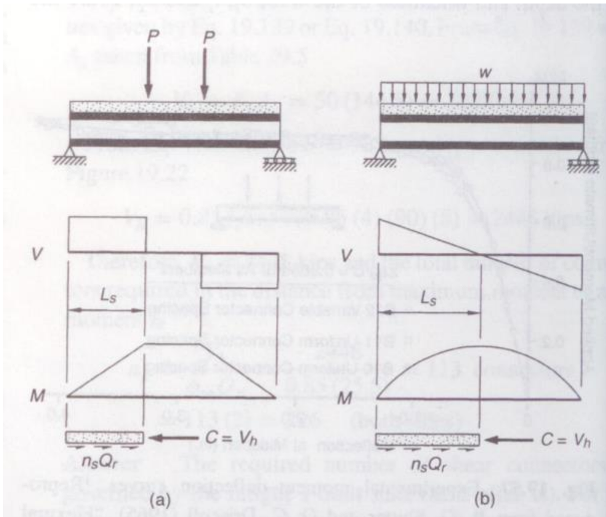


Fig. 19.56 Total number of shear connectors required: (a) concentrated loading and (b) uniformly distributed loading.

- Minimum number of shear connectors in areas between maximum positive moment and adjacent maximum negative moment in continuous spans

$$n = \frac{P + P_3}{Q_r}$$

P_3 force in slab at points of maximum negative moment

P_3 =0 reinforcement of the concrete slab is ignored for negative bending

P_3 lesser of $A_s F_y$ and $0.45 f_c' b_{eff} t$

-> represents the combined contribution of deck reinforcement and the tensile strength of the concrete-> more conservative in AASHTO LRFD!

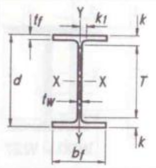


Table 1–1 (continued)
W Shapes
Dimensions

Shape	Area, A	Depth, d		Web Thickness, t_w		$t_w/2$	Flange Width, b_f		Flange Thickness, t_f		k_{des}	k_{det}	k_1	T	Workable Gage
	in.^{2}	in.		in.		in.	in.		in.		in.	in.	in.	in.	in.
W24×370[h]	109	28.0	28	1.52	1 1/2	3/4	13.7	13 5/8	2.72	2 3/4	3.22	3 5/8	1 9/16	20 3/4	5 1/2
×335[h]	98.4	27.5	27 1/2	1.38	1 3/8	11/16	13.5	13 1/2	2.48	2 1/2	2.98	3 3/8	1 1/2		
×306[h]	89.8	27.1	27 1/8	1.26	1 1/4	5/8	13.4	13 3/8	2.28	2 1/4	2.78	3 3/16	1 7/16		
x279[h]	82.0	26.7	26 3/4	1.16	1 3/16	5/8	13.3	13 1/4	2.09	2 1/16	2.59	3	1 7/16		
×250	73.5	26.3	26 3/8	1.04	1 1/16	9/16	13.2	13 1/8	1.89	1 7/8	2.39	2 13/16	1 3/8		
×229	67.2	26.0	26	0.960	15/16	1/2	13.1	13 1/8	1.73	1 3/4	2.23	2 5/8	1 5/16		
×207	60.7	25.7	25 3/4	0.870	7/8	7/16	13.0	13	1.57	1 9/16	2.07	2 1/2	1 1/4		
×192	56.3	25.5	25 1/2	0.810	13/16	7/16	13.0	13	1.46	1 7/16	1.96	2 3/8	1 1/4		
×176	51.7	25.2	25 1/4	0.750	3/4	3/8	12.9	12 7/8	1.34	1 5/16	1.84	2 1/4	1 3/16		
×162	47.7	25.0	25	0.705	11/16	3/8	13.0	13	1.22	1 1/4	1.72	2 1/8	1 3/16		
×146	43.0	24.7	24 3/4	0.650	5/8	5/16	12.9	12 7/8	1.09	1 1/16	1.59	2	1 1/8		
×131	38.5	24.5	24 1/2	0.605	5/8	5/16	12.9	12 7/8	0.960	15/16	1.46	1 7/8	1 1/8		
×117[c]	34.4	24.3	24 1/4	0.550	9/16	5/16	12.8	12 3/4	0.850	7/8	1.35	1 3/4	1 1/8		
×104[c]	30.6	24.1	24	0.500	1/2	1/4	12.8	12 3/4	0.750	3/4	1.25	1 5/8	1 1/16		
W24×103[c]	30.3	24.5	24 1/2	0.550	9/16	5/16	9.00	9	0.980	1	1.48	1 7/8	1 1/8	20 3/4	5 1/2
×94[c]	27.7	24.3	24 1/4	0.515	1/2	1/4	9.07	9 1/8	0.875	7/8	1.38	1 3/4	1 1/16		
×84[c]	24.7	24.1	24 1/8	0.470	1/2	1/4	9.02	9	0.770	3/4	1.27	1 11/16	1 1/16		
×76[c]	22.4	23.9	23 7/8	0.440	7/16	1/4	8.99	9	0.680	11/16	1.18	1 9/16	1 1/16		
×68[c]	20.1	23.7	23 3/4	0.415	7/16	1/4	8.97	9	0.585	9/16	1.09	1 1/2	1 1/16		
W24×62[c]	18.2	23.7	23 3/4	0.430	7/16	1/4	7.04	7	0.590	9/16	1.09	1 1/2	1 1/16	20 3/4	3 1/2[g]
×55[c,v]	16.2	23.6	23 5/8	0.395	3/8	3/16	7.01	7	0.505	1/2	1.01	1 7/16	1	20 3/4	3 1/2[g]
W21×201	59.2	23.0	23	0.910	15/16	1/2	12.6	12 5/8	1.63	1 5/8	2.13	2 1/2	1 5/16	18	5 1/2
×182	53.6	22.7	22 3/4	0.830	13/16	7/16	12.5	12 1/2	1.48	1 1/2	1.98	2 3/8	1 1/4		
×166	48.8	22.5	22 1/2	0.750	3/4	3/8	12.4	12 3/8	1.36	1 3/8	1.86	2 1/4	1 3/16		
×147	43.2	22.1	22	0.720	3/4	3/8	12.5	12 1/2	1.15	1 1/8	1.65	2	1 3/16		
×132	38.8	21.8	21 7/8	0.650	5/8	5/16	12.4	12 1/2	1.04	1 1/16	1.54	1 15/16	1 1/8		
×122	35.9	21.7	21 5/8	0.600	5/8	5/16	12.4	12 3/8	0.960	15/16	1.46	1 13/16	1 1/8		
×111	32.7	21.5	21 1/2	0.550	9/16	5/16	12.3	12 3/8	0.875	7/8	1.38	1 3/4	1 1/8		
×101[c]	29.8	21.4	21 3/8	0.500	1/2	1/4	12.3	12 1/4	0.800	13/16	1.30	1 11/16	1 1/16		

[c] Shape is slender for compression with F_y = 50 ksi.
[g] The actual size, combination, and orientation of fastener components should be compared with the geometry of the cross-section to ensure compatibility.
[h] Flange thickness greater than 2 in. Special requirements may apply per AISC Specification Section A3.1c.
[v] Shape does not meet the h/t_w limit for shear in Specification Section G2.1a with F_y = 50 ksi.

Table 1–1 (continued)
W Shapes
Properties

W24 – W21

Nominal Wt.	Compact Section Criteria $b_f/2t_f$	h/t_w	Axis X-X I	S	r	Z	Axis Y-Y I	S	r	Z	r_{ts}	h_o	Torsional Properties J	C_w
lb/ft			in.^{4}	in.^{3}	in.	in.^{3}	in.^{4}	in.^{3}	in.	in.^{3}	in.	in.	in.^{4}	in.^{6}
370	2.51	14.2	13400	957	11.1	1130	1160	170	3.27	267	3.92	25.3	201	186000
335	2.73	15.6	11900	864	11.0	1020	1030	152	3.23	238	3.86	25.0	152	161000
306	2.94	17.1	10700	789	10.9	922	919	137	3.20	214	3.81	24.9	117	142000
279	3.18	18.6	9600	718	10.8	835	823	124	3.17	193	3.76	24.6	90.5	125000
250	3.49	20.7	8490	644	10.7	744	724	110	3.14	171	3.71	24.5	66.6	108000
229	3.79	22.5	7650	588	10.7	675	651	99.4	3.11	154	3.67	24.3	51.3	96100
207	4.14	24.8	6820	531	10.6	606	578	88.8	3.08	137	3.62	24.1	38.3	84100
192	4.43	26.6	6260	491	10.5	559	530	81.8	3.07	126	3.60	24.0	30.8	76300
176	4.81	28.7	5680	450	10.5	511	479	74.3	3.04	115	3.57	23.9	23.9	68400
162	5.31	30.6	5170	414	10.4	468	443	68.4	3.05	105	3.57	23.8	18.5	62600
146	5.92	33.2	4580	371	10.3	418	391	60.5	3.01	93.2	3.53	23.7	13.4	54600
131	6.70	35.6	4020	329	10.2	370	340	53.0	2.97	81.5	3.49	23.5	9.50	47100
117	7.53	39.2	3540	291	10.1	327	297	46.5	2.94	71.4	3.46	23.4	6.72	40800
104	8.50	43.1	3100	258	10.1	289	259	40.7	2.91	62.4	3.42	23.3	4.72	35200
103	4.59	39.2	3000	245	10.0	280	119	26.5	1.99	41.5	2.40	23.6	7.07	16600
94	5.18	41.9	2700	222	9.87	254	109	24.0	1.98	37.5	2.40	23.4	5.26	15000
84	5.86	45.9	2370	196	9.79	224	94.4	20.9	1.95	32.6	2.37	23.3	3.70	12800
76	6.61	49.0	2100	176	9.69	200	82.5	18.4	1.92	28.6	2.34	23.2	2.68	11100
68	7.66	52.0	1830	154	9.55	177	70.4	15.7	1.87	24.5	2.30	23.1	1.87	9430
62	5.97	50.1	1550	131	9.23	153	34.5	9.80	1.38	15.7	1.75	23.2	1.71	4620
55	6.94	54.6	1350	114	9.11	134	29.1	8.30	1.34	13.3	1.71	23.1	1.18	3870
201	3.86	20.6	5310	461	9.47	530	542	86.1	3.02	133	3.55	21.4	40.9	62000
182	4.22	22.6	4730	417	9.40	476	483	77.2	3.00	119	3.51	21.2	30.7	54400
166	4.57	25.0	4280	380	9.36	432	435	70.0	2.99	108	3.48	21.1	23.6	48500
147	5.44	26.1	3630	329	9.17	373	376	60.1	2.95	92.6	3.45	20.9	15.4	41100
132	6.01	28.9	3220	295	9.12	333	333	53.5	2.93	82.3	3.42	20.8	11.3	36000
122	6.45	31.3	2960	273	9.09	307	305	49.2	2.92	75.6	3.40	20.7	8.98	32700
111	7.05	34.1	2670	249	9.05	279	274	44.5	2.90	68.2	3.37	20.6	6.83	29200
101	7.68	37.5	2420	227	9.02	253	248	40.3	2.89	61.7	3.35	20.6	5.21	26200