1002 – W-Shape Tension Member Design According to AISC

Description

An ASTM A992 W-shape member—with parameters defined in the table below—is selected to carry a dead load of 30.000 kips and a live load of 90.000 kips in tension. Verify the member strength by both LRFD and ASD, see [1], with the bolted end connection, as shown in Figure 1. Verify that the member satisfies the recommended slenderness limit. Assume that connection limit states do not govern.

Material		Modulus of Elasticity	E	29000.000	ksi
		Yield Strength	F_y	50.000	ksi
		Ultimate Strength	F_u	65.000	ksi
Geometry	Structure	Length	L	25.000	ft
	Cross-section W 8×21	Gross Area	A_g	6.160	in^2
		Flange Width	b_f	5.270	in
		Depth	d	8.280	in
		Flange Thickness	t_f	0.400	in
		Radius of Gyration	r_y	1.260	in
Load		Dead	P_D	30.000	kips
		Live	P_L	90.000	kips

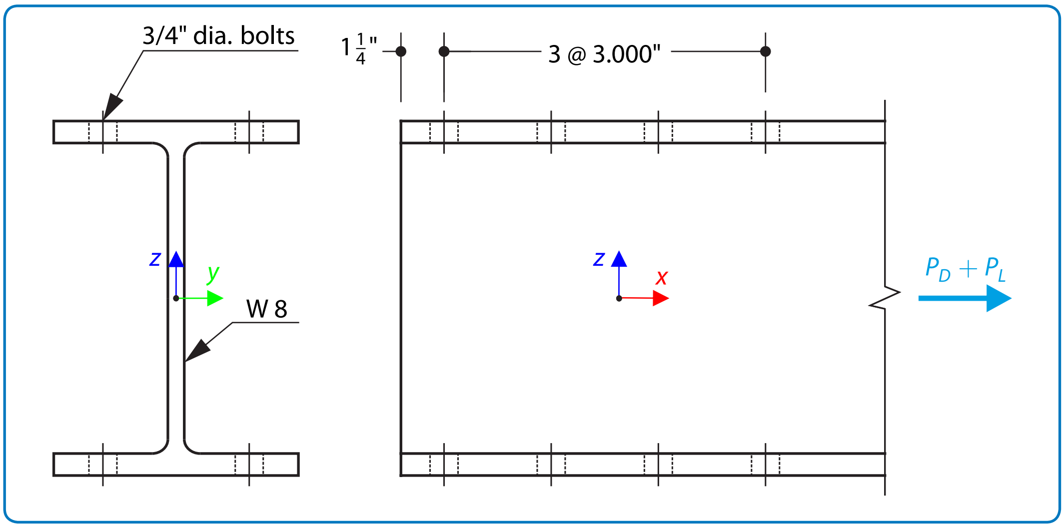

Figure 1: Connection geometry for Example D.1

AISC Solution

The WT-shape corresponding to a W 8×21 is a WT 4×10.5. From AISC Manual Table 1-8, the geometric properties are WT 4×10.5 and $\bar{y} = 0.831$ in.

1 Tensile Yielding

From AISC Manual Table 5-1, the available tensile yielding strength of a W 8×21 is

LRFD	ASD
$\phi_t P_n = 277.000 > 180.000$ kips	$P_n/\Omega_t = 184.000 > 120.000$ kips

2 Tensile Rupture

Verify the table assumption that $A_e/A_g \geq 0.750$ for this connection, where

$$A_e = A_n U \qquad (1002-1)$$

is the effective net area—the product of the net area A_n and the shear lag factor U as per AISC Specification Section D3.

First, calculate U as the larger of the values from AISC Specification Section D3, Table D3.1 Case 2 and Case 7.

Section D3: for open cross-sections, U need not be less than the ratio of the gross area of the connected element(s) to the member gross area,

$$U = \frac{2 b_f t_f}{A_g} \approx 0.684 \qquad (1002-2)$$

Table D3.1 Case 2: Determine U based on two WT-shapes per AISC Specification Commentary Figure C-D3.1, with $\bar{x} = \bar{y} = 0.831$ in and connection length $l = 9.000$ in,

$$U = 1 - \frac{\bar{x}}{l} \approx 0.908 \qquad (1002-3)$$

Table D3.1 Case 7: Because the flange is connected with three or more fasteners per line in the direction of loading and $b_f = 5.270 < \frac{2}{3}d = 5.520$, the shear lag factor is taken

$$U = 0.850 \qquad (1002-4)$$

Therefore, comparing **(1002 – 2)**, **(1002 – 3)**, and **(1002 – 4)**, use the larger $U = 0.908$.

Secondly, for **(1002 – 1)**, calculate A_n using AISC Specification Section B4.3b

$$A_n = A_g - 4 \cdot d_h + \frac{1}{16} \cdot t_f \approx 4.760 \text{ in}^2 \qquad (1002-5)$$

Substituting **(1002 – 5)** and **(1002 – 3)** into **(1002 – 1)**, there is

$$A_e = A_n U \approx 4.320\ \text{in}^2 \qquad (1002-6)$$

Hence,

$$\frac{A_e}{A_g} \approx 0.701 < 0.750 \qquad (1002-7)$$

Because $A_e/A_g < 0.750$, the tensile rupture strength from AISC Manual Table 5-1 is not valid. The available tensile rupture strength is determined using AISC Specification Section D2 as follows

$$P_n = F_u A_e = 281.000\ \text{kips} \qquad (1002-8)$$

From AISC Specification Section D.2, the available tensile rupture strength is

LRFD	ASD
$\phi_t = 0.750$ $\phi_t P_n = 211.000 > 180.000$ kips	$\Omega_t = 2.000$ $P_n/\Omega_t = 141.000 > 120.000$ kips

Note that the W 8×21 available tensile strength is governed by the tensile rupture limit state at the end connection versus the tensile yielding limit state.[1]

The Recommended Slenderness Limit per AISC Specification Section D.1 is met

$$\frac{L}{r} = \frac{25 \cdot 12}{1.26} = 238.000\ \text{in} < 300.000\ \text{in} \qquad (1002-9)$$

RFEM 6 Settings

- Modeled in RFEM 6.01.0007
- Isotropic linear elastic model is used
- Shear stiffness of members is activated

Results

Design	Tensile Failure	AISC Solution [kips]	RFEM Solution [kips]	Ratio [-]
LRFD	Yielding	277.200	277.000	1.001
	Rupture	210.701	211.000	0.999
ASD	Yielding	184.431	184.000	1.002
	Rupture	140.468	141.000	0.996

[1] See Chapter J for illustrations of connection limit state checks.

Available Yielding/Tensile Strength

1 LRFD

Example (Shape)	Tensile Failure	RFEM Solution [kips]	AISC Solution [kips]	Ratio [-]
D.2 (L 4×4×1/2)	Yielding	121.500	122.000	0.996
	Rupture	125.124	125.000	1.001
D.3 (WT 6×20)	Yielding	262.800	263.000	0.999
	Rupture	247.689	245.000	0.999
D.4 (HSS 6×4×0.375)	Yielding	278.100	278.000	1.000
	Rupture	242.312	242.000	1.001
D.5 (HSS 6×0.500)	Yielding	334.926	335.000	0.999
	Rupture	352.005	352.000	1.000
D.6 (2LA4×4×1/2 (3/8" Gap))	Yielding	243.000	243.000	1.000
	Rupture	272.255	272.000	1.001

2 ASD

Example (Shape)	Tensile Failure	RFEM Solution [kips]	AISC Solution [kips]	Ratio [-]
D.2 (L 4×4×1/2)	Yielding	80.838	80.800	1.001
	Rupture	83.416	83.500	0.999
D.3 (WT 6×20)	Yielding	174.851	175.000	0.999
	Rupture	165.126	163.000	1.013
D.4 (HSS 6×4×0.375)	Yielding	185.030	185.000	1.000
	Rupture	161.541	162.000	0.997
D.5 (HSS 6×0.500)	Yielding	222.838	223.000	0.999
	Rupture	234.670	235.000	0.999
D.6 (2LA4×4×1/2 (3/8" Gap))	Yielding	161.677	162.000	0.998
	Rupture	181.503	182.000	0.997

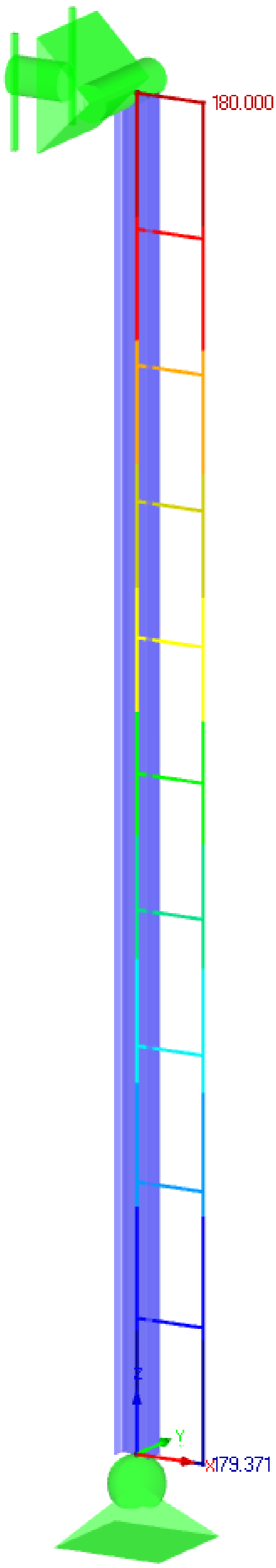

Figure 2: RFEM 6 Results - Axial Forces (LRFD)

References

[1] AMERICAN INSTITUTE OF STEEL CONSTRUCTION, *Specification for Structural Steel Buildings*. 2016.