1002 – AISC D.1 - W-Shape Tension Member

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
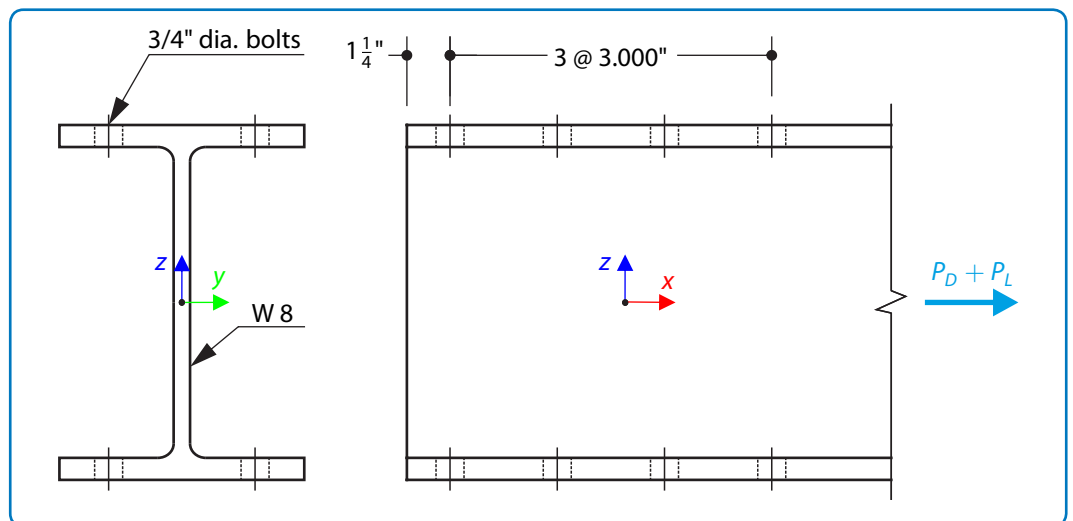
Figure 1: Connection geometry for Example D.1

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

$$A_e = A_n U \tag{1002 – 1}$$

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,

$$U = \frac{2b_f t_f}{A_g} \approx 0.684 \tag{1002 – 2}$$

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,

$$U = 1 - \frac{\bar{x}}{l} \approx 0.908 \tag{1002 – 3}$$

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

$$U = 0.850 \tag{1002 – 4}$$

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

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

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

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

Hence,

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

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

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

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

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

RFEM 5 Settings

- Modeled in RFEM 5.14.03
- 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	277.202	277.000	1.001
	Rupture	210.703	211.000	0.999
ASD	Yielding	184.432	184.000	1.002
	Rupture	140.469	141.000	0.996

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[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	121.501	122.000	0.996
	Rupture	125.124	125.000	1.001
D.3 (WT 6×20)	Yielding	262.802	263.000	0.999
	Rupture	244.844	245.000	0.999
D.4 (HSS 6×4×0.375)	Yielding	278.102	278.000	1.000
	Rupture	242.313	242.000	1.001
D.5 (HSS 6×0.500)	Yielding	334.928	335.000	0.999
	Rupture	352.007	352.000	1.000
D.6 (2LA4×4×1/2 (3/8" Gap))	Yielding	243.002	243.000	1.000
	Rupture	272.256	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	80.839	80.800	1.001
	Rupture	83.416	83.500	0.999
D.3 (WT 6×20)	Yielding	174.851	175.000	0.999
	Rupture	163.229	163.000	1.001
D.4 (HSS 6×4×0.375)	Yielding	185.0311	185.000	1.000
	Rupture	161.542	162.000	0.997
D.5 (HSS 6×0.500)	Yielding	222.840	223.000	0.999
	Rupture	234.672	235.000	0.999
D.6 (2LA4×4×1/2 (3/8" Gap))	Yielding	161.678	162.000	0.998
	Rupture	180.735	182.000	0.993
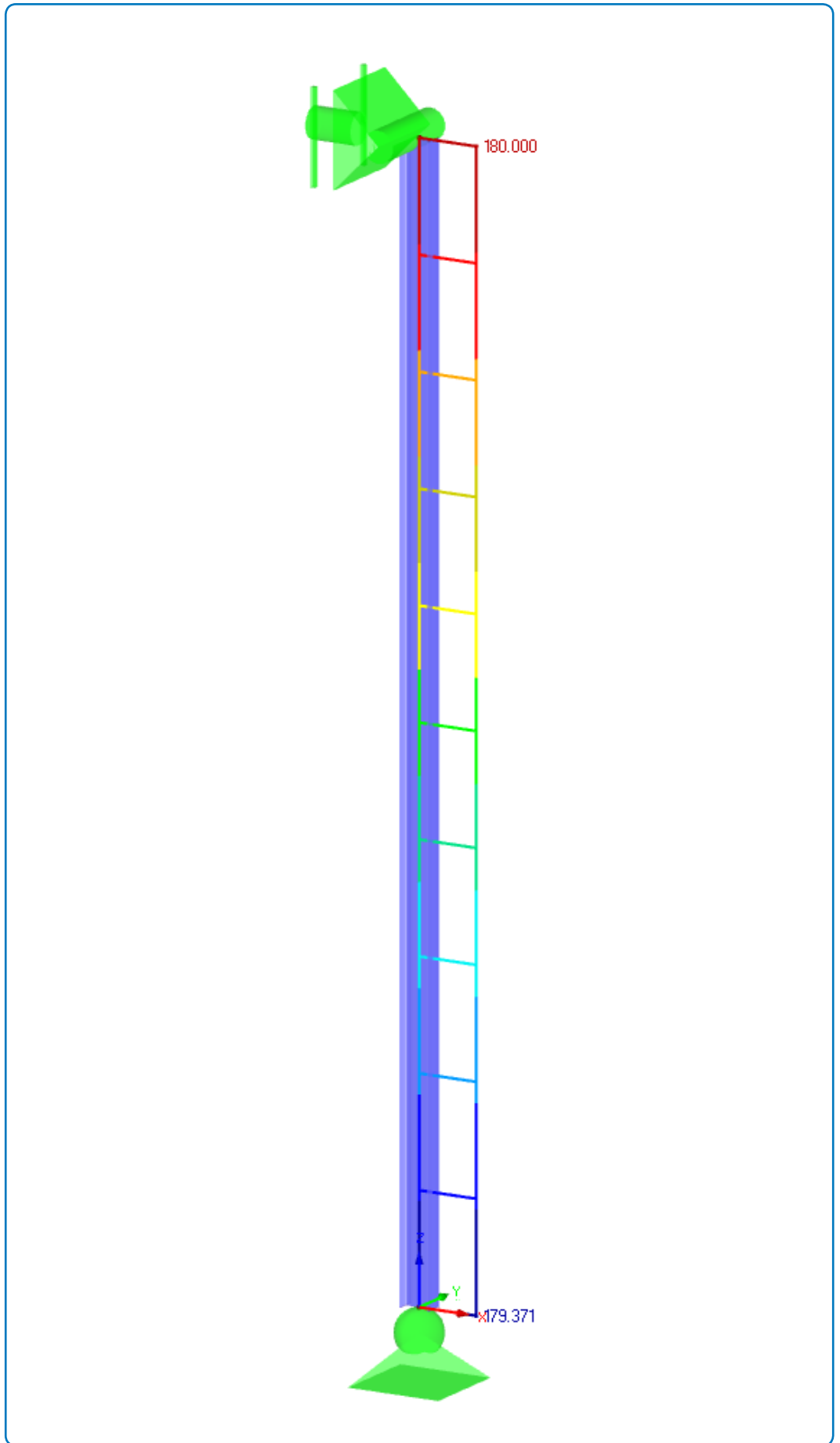
Figure 2: RFEM 5 Results - Axial Forces (LRFD)

References

[1] AMERICAN INSTITUTE OF STEEL CONSTRUCTION, *Specification for Structural Steel Buildings*. 2015.