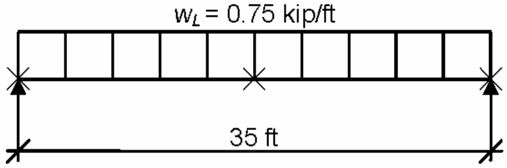
Example F.1-3a. W-Shape Flexural Member design in Strong-Axis Bending, Braced at Midspan

Given:

Verify the strength of the W18×50 beam selected in Example F.1-1a if the beam is braced at the ends and center point rather than continuously braced.



Beam Loading & Bracing Diagram
(bracing at ends & midpoint)

Solution:

Required flexural strength at midspan from Example F.1-1a

LRFD	ASD
M_u = 266 kip-ft	M_a = 184 kip-ft

$$L_b = \frac{35.0\,\text{ft}}{2} = 17.5\,\text{ft}$$

For a uniformly loaded beam braced at the ends and at the center point, C_b = 1.30. There are several ways to make adjustments to Table 3-10 to account for C_b greater than 1.0. Manual Table 3-1

Procedure A.

Available moments from the sloped and curved portions of the plots in from Manual Table 3-10 may be multiplied by C_b, but may not exceed the value of the horizontal portion (ϕM_n for LRFD, M_n/Ω for ASD).

Obtain the available strength of a W18×50 with an unbraced length of 17.5 ft from Manual Table 3-10

Enter Table 3-10 and find the intersection of the curve for the W18×50 with an unbraced length of 11.7 ft. Obtain the available strength from the appropriate vertical scale to the left.

LRFD	ASD
$\phi_b M_n \approx$ 222 kip-ft $\phi_b M_p \approx$ 379 kip-ft (upper limit on $C_b M_n$) *Adjust for C_b* (1.30)(222 kip-ft) = 288 kip-ft	$M_n / \Omega_b \approx$ 147 kip-ft $M_p / \Omega_b \approx$ 252 kip-ft (upper limit on $C_b M_n$) *Adjust for C_b* (1.30)(147 kip-ft) = 191 kip-ft

Manual Table 3-10

Check Limit 288 kip-ft ≤ $\phi_b M_p$ = 379 kip-ft **o.k.** *Check available versus required strength* 288 kip-ft > 266 kip-ft **o.k.**	*Check Limit* 191 kip-ft ≤ M_p / Ω_b = 252 kip-ft **o.k.** *Check available versus required strength* 191 kip-ft > 184 kip-ft **o.k.**

Procedure B.

For preliminary selection, the required strength can be divided by C_b and directly compared to the strengths in Table 3-10. Members selected in this way must be checked to ensure that the required strength does not exceed the available plastic moment strength of the section.

Calculate the adjusted required strength

LRFD	ASD
M_u' = 266 kip-ft / 1.3 = 205 kip-ft	M_a' = 184 kip-ft / 1.3 = 142 kip-ft

Obtain the available strength for a W18×50 with an unbraced length of 17.5 ft from Manual Table 3-10

Manual Table 3-10

LRFD	ASD
$\phi_b M_n$ ≈ 222 kip-ft > 205 kip-ft **o.k.** $\phi_b M_p$ ≈ 379 kip-ft > 266 kips **o.k.**	M_n / Ω_b ≈ 147 kip-ft > 142 kip-ft **o.k.** M_p / Ω_b ≈ 252 kip-ft > 184 kips **o.k.**

Example F.1-3b. W-Shape Flexural Member Design in Strong-Axis Bending, Braced at Midspan

Given:

Example F.1-3a was solved by utilizing the tables of the AISC *Steel Construction Manual*. Alternatively, this problem can be solved by applying the requirements of the AISC Specification directly.

Solution:

Geometric Properties:

W18×50 r_{ts} = 1.98 in. S_x = 88.9 in.^{3} J = 1.24 in.^{4} h_o = 17.4 in.

Required strength from Example F.1-3a

LRFD	ASD
M_u = 266 kip-ft	M_a = 184 kip-ft

Calculate the nominal flexural strength, M_n

Calculate C_b

$$C_b = \frac{12.5M_{max}}{2.5M_{max}+3M_A+4M_B+3M_C}R_m \le 3.0$$ Eqn. F1-1

The required moments for Equation F1-1 can be calculated as a percentage of the maximum midspan moment as: M_{max}= 1.00, M_A = 0.438, M_B = 0.750, and M_C = 0.938.

R_m = 1.0 for doubly-symmetric members

$$C_b = \frac{12.5(1.00)}{2.5(1.00)+3(0.438)+4(0.750)+3(0.938)}(1.0) = 1.30$$

L_p = 5.83 ft Manual
L_r = 17.0 ft Table 3-6

For a compact beam with an unbraced length $L_b > L_r$, the limit state of elastic lateral-torsional buckling applies.

Calculate F_{cr} with L_b = 17.5 ft

$$F_{cr} = \frac{C_b\pi^2 E}{\left(\frac{L_b}{r_{ts}}\right)^2}\sqrt{1+0.078\frac{Jc}{S_x h_o}\left(\frac{L_b}{r_{ts}}\right)^2}$$ where c = 1.0 for doubly symmetric I-shapes Eqn. F2-4

$$F_{cr} = \frac{1.30\pi^2(29{,}000\text{ ksi})}{\left(\frac{17.5\text{ ft}(12\text{ in./ft})}{1.98\text{ in.}}\right)^2}\sqrt{1+0.078\frac{(1.24\text{ in.}^4)1.0}{(88.9\text{ in.}^3)(17.4\text{ in.})}\left(\frac{17.5\text{ ft}(12\text{ in./ft})}{1.98\text{ in.}}\right)^2} = 43.2\text{ ksi}$$

$M_n = F_{cr} S_x \le M_p$ Eqn. F2-3

$M_n = 43.2 \text{ ksi}(88.9 \text{ in.}^3) = 3840 \text{ kip-in.} < 5050 \text{ kip-in.}$
$M_n = 3840 \text{ kip-in or } 320 \text{ kip-ft}$

Calculate the available flexural strength

Section F1

LRFD	ASD
$\phi_b = 0.90$	$\Omega_b = 1.67$
$\phi_b M_n = 0.90(320 \text{ kip-ft}) = 288 \text{ kip-ft}$	$\dfrac{M_n}{\Omega_b} = \dfrac{320 \text{ kip-ft}}{1.67} = 192 \text{ kip-ft}$
288 kip-ft > 266 kip-ft **o.k.**	192 kip-ft > 184 kip-ft **o.k.**

STEEL BEAM DESIGN

GEOMETRY

Column Designation	W18X50		
Steel Yield Strength Fy	50.0	ksi	OK
Modulus of Elasticity Es ..	29000	ksi	
Member Length L	35.00	ft	
Left Cantilever	0.00	ft	
Right Cantilever	0.00	ft	
Unbraced Length Lb	17.50	ft	OK

PROPERTIES

Area ..	14.7	in2	Sx ...	88.9	in3
Depth	18.0	in	Zx ...	101.0	in3
bf	7.5	in	rx	7.38	in
tw	0.36	in	Iy	40.1	in4
tf	0.57	in	Sy ...	10.7	in3
k des .	0.97	in	Zy ...	16.6	in3
Ix	800.0	in4	ry	1.65	in

SERVICE LOADS (ASD)

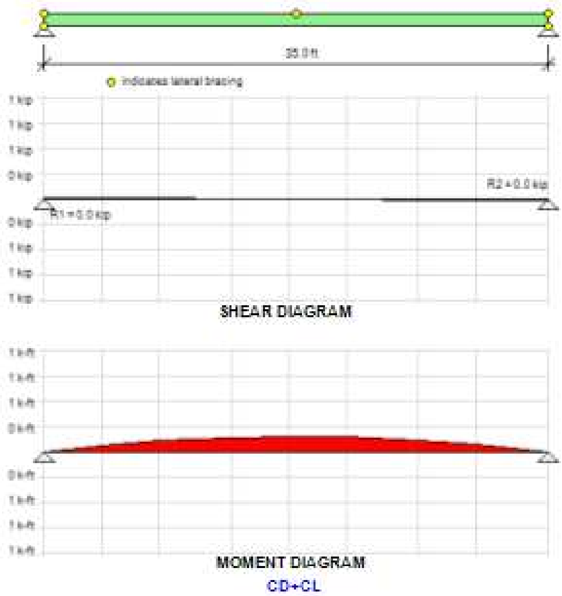
	Uniform (k/ft)		Concentrated (kip)					
	w1	w2	P1	P2	P3	P4	P5	P6
Dead Load	0.45	0.00	0.0	0.0	0.0	0.0	0.0	0.0
Live Load	0.75	0.00	0.0	0.0	0.0	0.0	0.0	0.0
Const. Dead Load ..	0.00	0.00	0.0	0.0	0.0	0.0	0.0	0.0
Const. Live Load	0.00	0.00	0.0	0.0	0.0	0.0	0.0	0.0
Start Distance (ft) ...	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
End Distance (ft)	35.00	0.00						

DESIGN FOR SHEAR

Shear Coefficient Cv	1.00		
Maximum Shear Force V ..	21.0	kip	
Limit States	Nominal Vn		
Shear Yielding	191.7	kip	✔
Shear Buckling	191.7	kip	
Nominal Strength Vn	191.7	kip	
Safety Factor Ω	1.50		
Allowable Strength Vn/Ω .	127.8	kip	
V / Vn/Ω Design Ratio	0.16		OK

LOCAL BUCKLING

Flexure in Flanges	Compact
Compression in Flanges	Non-compact
Flexure in Web	Compact
Compression in Web	Slender



FLEXURE DESIGN (STEEL)

L. T. Buckling Cb-factor	1.30	
Max. Bending Moment M ..	183.8	k-ft

Limit States	Nominal Mn		
Yielding	420.8	k-ft	
Lateral Torsional Buckling	319.8	k-ft	✔
Flange Local Buckling	N.A.	k-ft	
Web Local Buckling	N.A.	k-ft	

Nominal Strength Mn	319.8	k-ft	
Safety Factor Ω	1.67		
Allowable Strength Mn/Ω .	191.5	k-ft	
M / Mn/Ω Design Ratio	0.96		OK

FLEXURE DESIGN (COMPOSITE)

Overall Slab Thickness	N.A
Interior Beam. Spacing = 5.0 ft	
Effective Slab Width	N.A
Concrete Strength f'c	N.A
Concrete Density	N.A
Metal Deck Type	VULCRAFT 2 VLI
Deck Ribs Height hr	N.A
Deck Ribs Avg. Width wr ...	N.A
Deck Ribs Run Perpendicular to the Beam	
Max. Bending Moment M ..	N.A

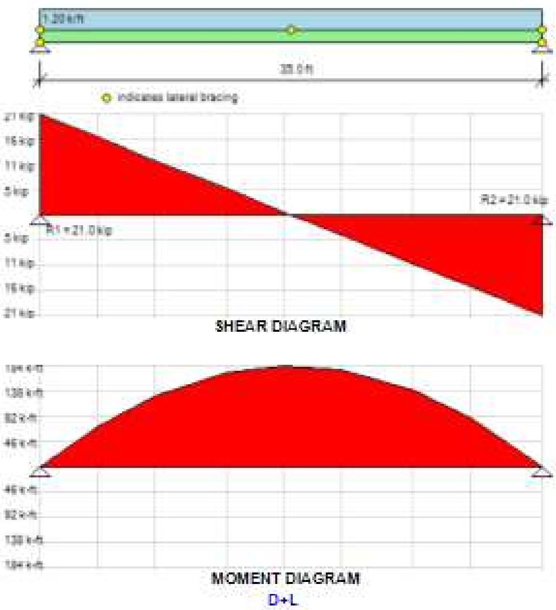
Limit States	Nominal Mn
Plastic Yielding	N.A
Elastic Yielding	N.A.

Nominal Strength Mn	N.A
Safety Factor Ω	1.67
Allowable Strength Mn/Ω .	N.A
M / Mn/Ω Design Ratio	N.A

DEFLECTIONS

Required Camber	0.00	in
Long-term Deflection	0.36	in

Loading	delta	L/delta	Ratio	
CL	0.00 in	9999	0.04	OK
CD+CL .	0.00 in	9999	0.02	OK
L	1.09 in	385	0.94	OK
D+L	1.75 in	240	1.00	OK



SHEAR CONNECTORS

Shear Stud Diameter	N.A
Shear Stud Length	N.A
Tensile Strength Fu	N.A
Nominal Strength Qn	N.A
Horizontal Shear Force	N.A
# of Studs for Full Composite	N.A
# of Studs for Partial Composite .	N.A
Partial Composite Action %	N.A
Minimum Spacing Allowed ..	N.A
# of Studs at Any Section	N.A
Max. Spacing Required	N.A