CE 415
DESIGN OF STEEL STRUCTURES

LECTURE 20
REVIEW (CONT.)

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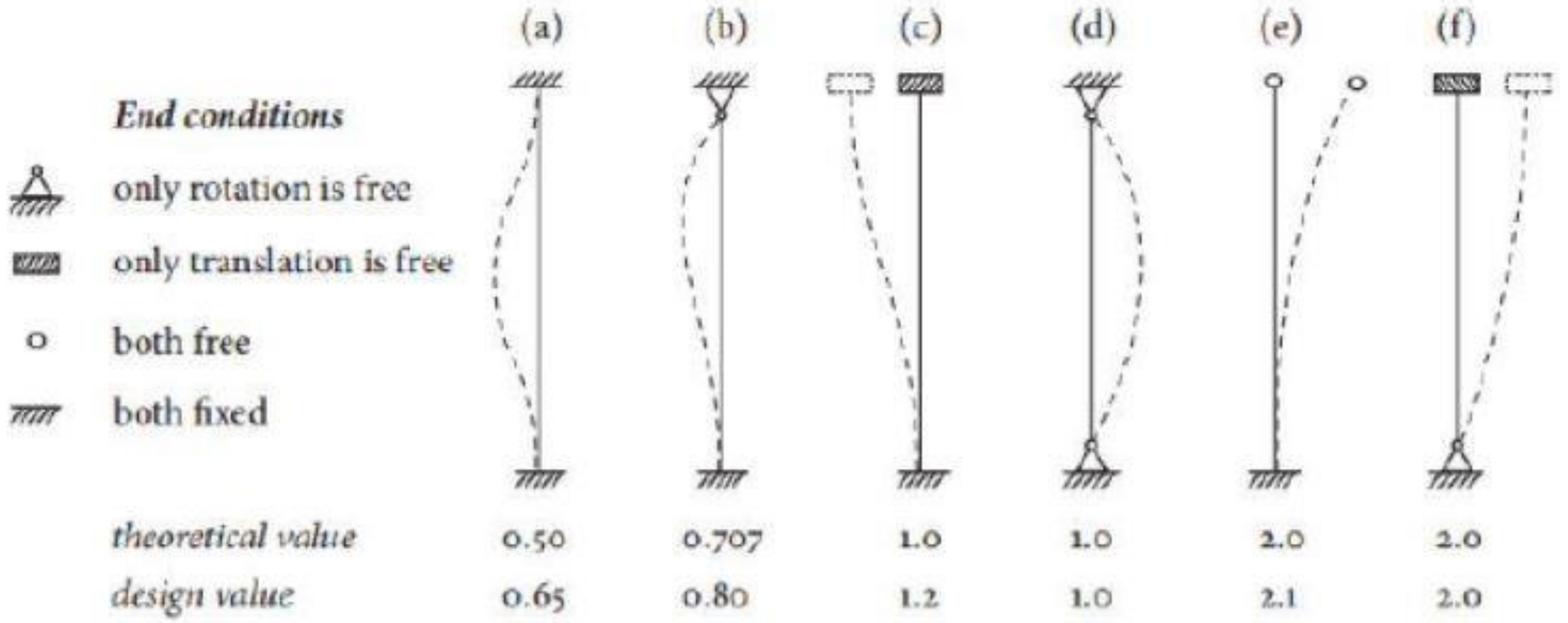
OUTLINE

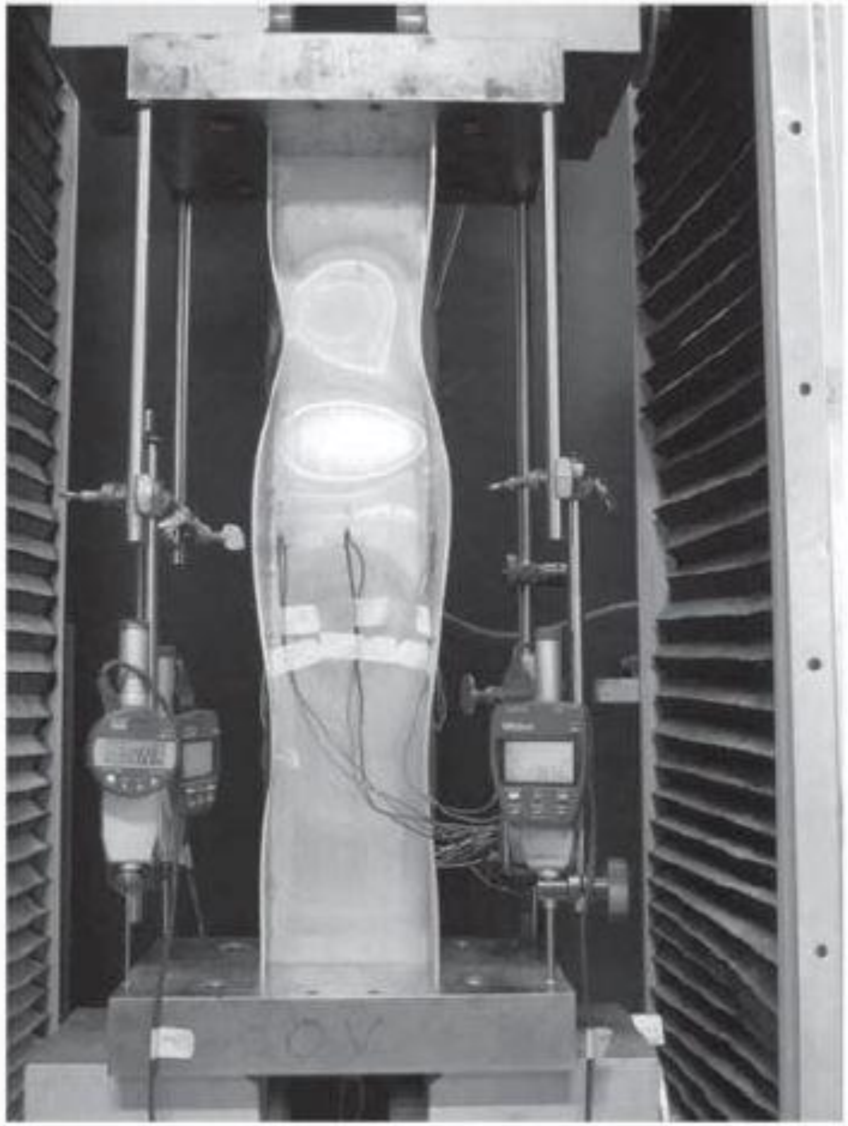
- ➢ Value of k
- ➢ Local buckling
- ➢ Beam moment strength

Values of K



Summary:
Fixed-Free=2.0
Pinned-Pinned=1
Pinned-Fixed= 0.8
Fixed-Fixed=0.65

LOCAL BUCKLING



Local buckling primarily depends on the ratio, *b/t*, of the width (*b*) and thickness (*t*) of the plate elements that builds up a section.

LOCAL BUCKLING

Based on the width/thickness ratio steel sections are defined as

Compact: A compact section reaches its cross-sectional material strength, or capacity, before local buckling occurs.

Non-Compact: In a non-compact section, only a portion of the cross-section reaches its yield strength before local buckling occurs.

Slender: In a slender section, the cross-section does not yield and the strength of the member is governed by local buckling.

The use of slender sections as compression members is not efficient or economical; therefore, the use of slender section in design practice is not recommended.

Desig-nation	Nom. Weight, lb/ft	Area, A, in^2	Depth, d, in	Flange		Web Thickness t_w, in	Axis X-X			Axis Y-Y		
				Width, b_f, in	Thick-ness, tt_f, in		I, in^4	S, in^3	r, in	I, in^4	S, in^3	r, in
W10 x 68	68	20	10.4	10.13	0.77	0.47	394	75.7	4.44	134	26.4	2.59
W10 x 60	60	17.6	10.22	10.08	0.68	0.42	341	66.7	4.39	116	23	2.57
W10 x 54	54	15.8	10.09	10.03	0.615	0.37	303	60	4.37	103	20.6	2.56
W10 x 49	49	14.4	9.98	10	0.56	0.34	272	54.6	4.35	93.4	18.7	2.54
W10 x 45	45	13.3	10.1	8.02	0.62	0.35	248	49.1	4.32	53.4	13.3	2.01
W10 x 39	39	11.5	9.92	7.985	0.53	0.315	209	42.1	4.27	45	11.3	1.98
W10 x 33	33	9.71	9.73	7.96	0.435	0.29	170	35	4.19	36.6	9.2	1.94
W10 x 30	30	8.84	10.47	5.81	0.51	0.3	170	32.4	4.38	16.7	5.75	1.37
W10 x 26	26	7.61	10.33	5.77	0.44	0.26	144	27.9	4.35	14.1	4.89	1.36
W10 x 22	22	6.49	10.17	5.75	0.36	0.24	118	23.2	4.27	11.4	3.97	1.33

EXAMPLE:

Determine the allowable compressive load carrying capacity of the column shown in Fig. It consists of W10×45 section having A992 (F_y = 50 ksi) steel. There are hinge support at top and bottom that allows rotation in any direction. Also the column has weak direction support (braced) at mid-height so that lateral deflection is prevented in x direction. Use ASD approach.

SOLUTION:

For W10×45 section, from AISC Manual Chart we have

A = 13.3 in^2, r_x=4.32 in., r_y=2.01 in.

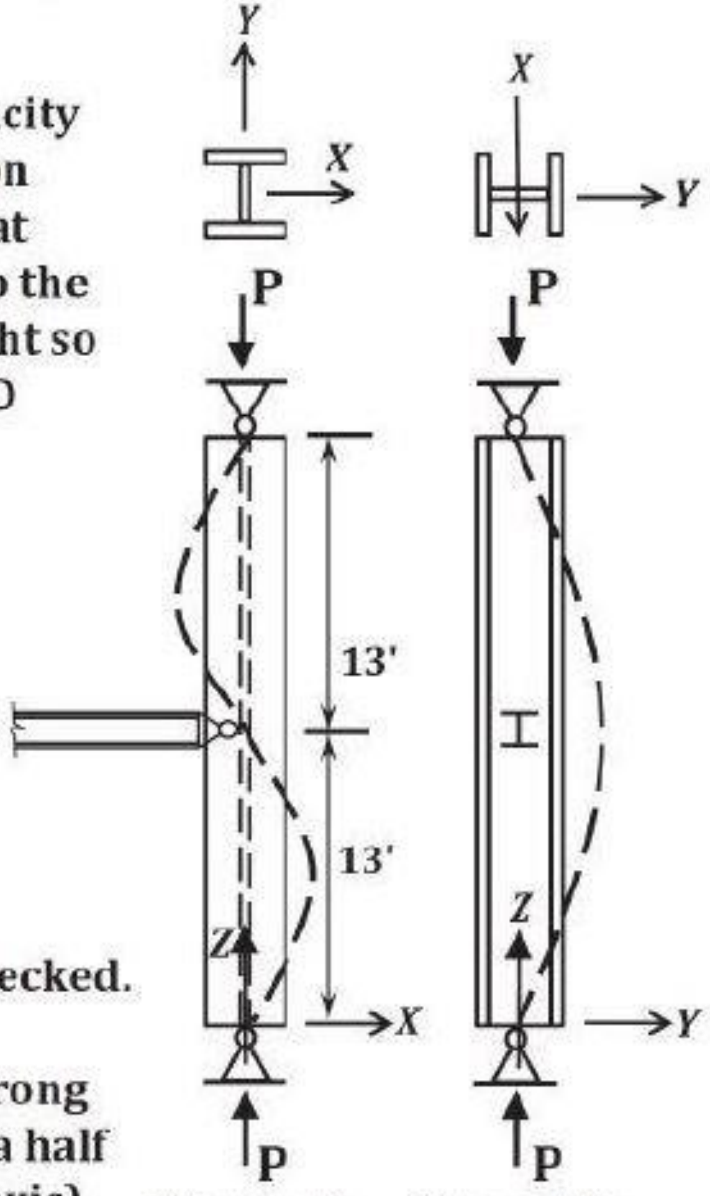
$x \rightarrow$ strong axis

$y \rightarrow$ weak axis

Column length, L = (13×2) ×12 = 312 in.

Possibility of buckling in both x and y directions to be checked.

Buckling in y direction causes bending about x axis or strong axis. For strong axis buckling, the buckling shape is like a half sine wave over full column length. Thus for strong (or x axis) axis buckling, K_x = 1.0



Weak-axis buckling

Strong-axis buckling

$\therefore K_xL/r_x = 1.0\times312/4.32 = 72.22$
$\therefore F_{ex} = \pi^2E/(K_xL/r_x)^2 = 3.14^2\times29000/(72.2)^2 = 54.82$ ksi. ($> F_y$, note)
And $4.71\sqrt{(E/F_y)} = 4.71\sqrt{(29000/50)} = 113.43 \therefore K_xL/r_x < 4.71\sqrt{(E/F_y)}$

$$F_{cr} = \left[0.658^{\frac{F_y}{F_e}}\right]F_y = [0.658^{(50/54.82)}]50 = 34.13 \text{ ksi}$$

Nominal strength for x-axis buckling $P_{nx} = F_{cr}A_g = 34.13\times13.3 = 454$ kip

Buckling in x direction causes bending about y axis or weak axis. For weak axis buckling, the buckling shape is like a full sine wave over full column length. Thus for weak (or y axis) axis buckling, $K_y = 0.5$

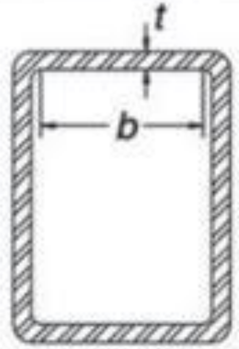
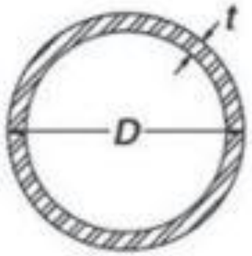
$\therefore K_yL/r_y = 0.5\times312/2.01 = 77.61$
$\therefore F_{ey} = \pi^2E/(K_yL/r_y)^2 = 3.14^2\times29000/(77.61)^2 = 47.47$ ksi.

$$F_{cr} = \left[0.658^{\frac{F_y}{F_e}}\right]F_y = [0.658^{(50/47.47)}]50 = 32.17 \text{ ksi}$$

Nominal strength for y axis buckling $P_{ny} = F_{cr}A_g = 32.17\times13.3 = 427.9$ kip

$\therefore P_n =$ smaller of P_{nx} and $P_{ny} = 427.9$ kip

$\therefore$ Allowable strength $P = P_n/\Omega = 427.9/1.67 = 256.2$ kip

Case	Description of Element	Width Thick-ness Ratio	Limiting Width-Thickness Ratios		Example
			λ_p (compact)	λ_r (noncompact)	
12	Uniform compression in flanges of rectangular box and hollow structural sections of uniform thickness subject to bending or compression; flange cover plates and diaphragm plates between lines of fasteners or welds	b/t	$1.12\sqrt{E/F_y}$	$1.40\sqrt{E/F_y}$	
15	Circular hollow sections				
	In uniform compression	D/t	NA	$0.11\,E/F_y$	
	In flexure	D/t	$0.07\,E/F_y$	$0.31\,E/F_y$	

LOCAL BUCKLING

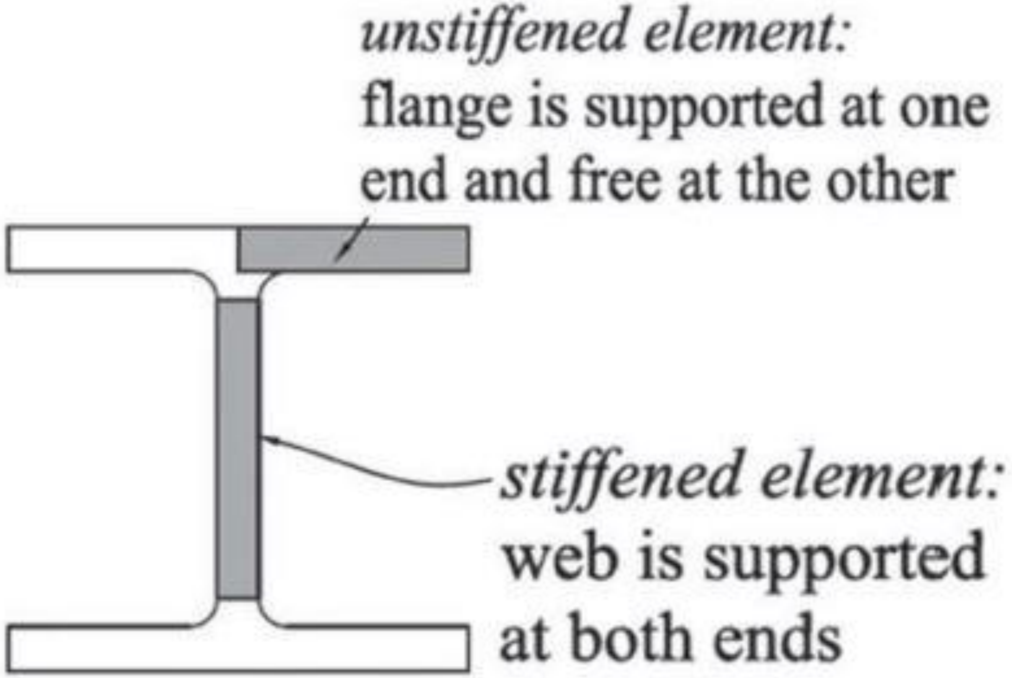
There are also two type of elements of a column section :

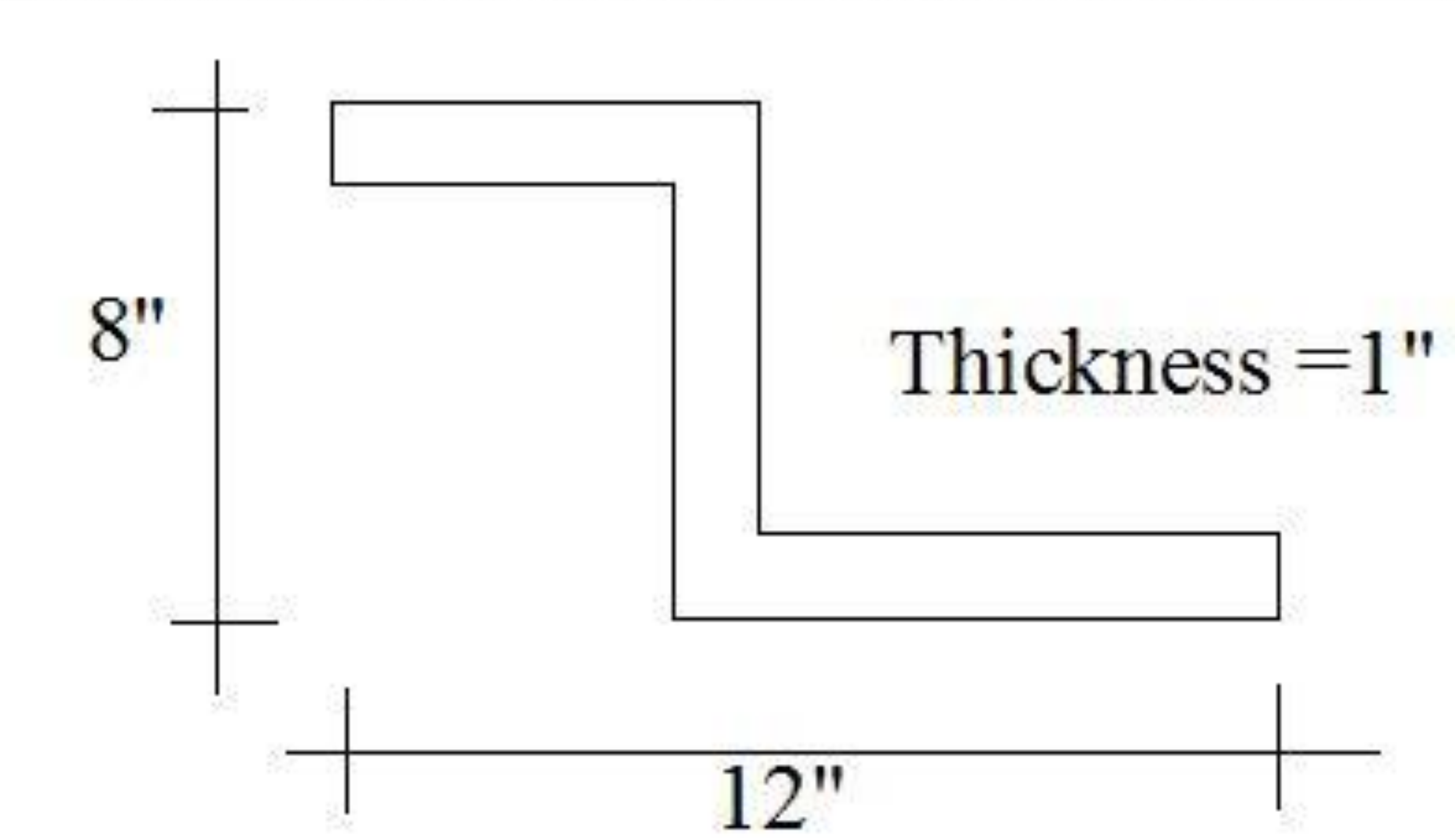
Stiffened:

Stiffened elements are supported along both edges parallel to the applied axial load. An example of this is the web of an I-shaped column where the flanges are connected on either end of the web.

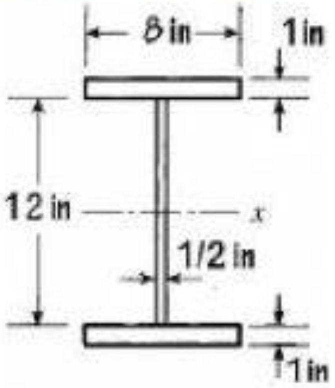
Unstiffened:

An unstiffened element has only one unsupported edge parallel to the axial load—for example, the outstanding flange of an I-shaped column that is connected to the web on one edge and free along the other edge.




8"
Thickness =1"
12"

Ques. Investigate the local stability of the following section.



Flange Buckling Check

$$\lambda = \frac{b_f}{2t_f} = \frac{8}{2 \times 1} = 4$$

$$\lambda_p = 0.38\sqrt{E/F_y} = 0.38\sqrt{29000/50} = 9.15$$

Since $\lambda(4) < \lambda_p(9.15)$, flange is compact.

Web Buckling Check

$$\lambda = \frac{h}{t_w} = \frac{12}{0.5} = 24$$

$$\lambda_p = 3.76\sqrt{E/F_y} = 3.76\sqrt{29000/50} = 90.6$$

Since $\lambda(24) < \lambda_p(90.6)$, web is also compact.

Ans. Section is compact.

Slender Flange Sections

When the width/thickness ratio λ [$=b_f/(2t_f)$] exceeds the limit λ_r of AISC-B4, the section is referred to as "slender" and must be treated in accordance with AISC-F3.2(b). The nominal strength of such a section is

$$M_n = \frac{0.9Ek_cS_x}{\lambda^2} \qquad \text{[Eq. F3-2, page 49, AISC 360-05]}$$

$$k_c = \frac{4}{\sqrt{h/t_w}}, \text{ where } 0.35 \leq k_c \leq 0.763$$

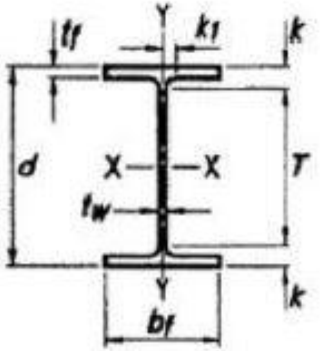


Table 1–1 (continued)
W Shapes
Dimensions

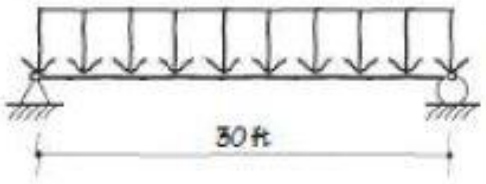
Shape	Area, A	Depth, d		Web Thickness, t_w		$\frac{t_w}{2}$	Flange Width, b_f		Flange Thickness, t_f		Distance k_{des}	Distance k_{det}	k_1	T	Workable Gage
	in.^{2}	in.		in.		in.	in.		in.		in.	in.	in.	in.	in.
W16×100	29.5	17.0	17	0.585	9/16	5/16	10.4	10 3/8	0.985	1	1.39	1 7/8	1 1/8	13 1/4	5 1/2
×89	26.2	16.8	16 3/4	0.525	1/2	1/4	10.4	10 3/8	0.875	7/8	1.28	1 3/4	1 1/16		
×77	22.6	16.5	16 1/2	0.455	7/16	1/4	10.3	10 1/4	0.760	3/4	1.16	1 5/8	1 1/16		
×67[c]	19.7	16.3	16 3/8	0.395	3/8	3/16	10.2	10 1/4	0.665	11/16	1.07	1 9/16	1		
W16×57	16.8	16.4	16 3/8	0.430	7/16	1/4	7.12	7 1/8	0.715	11/16	1.12	1 3/8	7/8	13 5/8	3 1/2[g]
×50[c]	14.7	16.3	16 1/4	0.380	3/8	3/16	7.07	7 1/8	0.630	5/8	1.03	1 5/16	13/16		
×45[c]	13.3	16.1	16 1/8	0.345	3/8	3/16	7.04	7	0.565	9/16	0.967	1 1/4	13/16		
×40[c]	11.8	16.0	16	0.305	5/16	3/16	7.00	7	0.505	1/2	0.907	1 3/16	13/16		
×36[c]	10.6	15.9	15 7/8	0.295	5/16	3/16	6.99	7	0.430	7/16	0.832	1 1/8	3/4		
W16×31[c]	9.13	15.9	15 7/8	0.275	1/4	1/8	5.53	5 1/2	0.440	7/16	0.842	1 1/8	3/4	13 5/8	3 1/2
×26[c,v]	7.68	15.7	15 3/4	0.250	1/4	1/8	5.50	5 1/2	0.345	3/8	0.747	1 1/16	3/4	13 5/8	3 1/2
W14×730[h]	215	22.4	22 3/8	3.07	3 1/16	1 9/16	17.9	17 7/8	4.91	4 15/16	5.51	6 3/16	2 3/4	10	3-7 1/2-3[g]
×665[h]	196	21.6	21 5/8	2.83	2 13/16	1 7/16	17.7	17 5/8	4.52	4 1/2	5.12	5 13/16	2 5/8		3-7 1/2-3[g]

Table 1–1 (continued)
W Shapes
Properties

W16 – W14

Nominal Wt.	Compact Section Criteria		Axis X-X				Axis Y-Y				r_{ts}	h_o	Torsional Properties	
	$\frac{b_f}{2t_f}$	$\frac{h}{t_w}$	I	S	r	Z	I	S	r	Z			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}
100	5.29	24.3	1490	175	7.10	198	186	35.7	2.51	54.9	2.92	16.0	7.73	11900
89	5.92	27.0	1300	155	7.05	175	163	31.4	2.49	48.1	2.88	15.9	5.45	10200
77	6.77	31.2	1110	134	7.00	150	138	26.9	2.47	41.1	2.85	15.8	3.57	8590
67	7.70	35.9	954	117	6.96	130	119	23.2	2.46	35.5	2.82	15.7	2.39	7300
57	4.98	33.0	758	92.2	6.72	105	43.1	12.1	1.60	18.9	1.92	15.7	2.22	2660
50	5.61	37.4	659	81.0	6.68	92.0	37.2	10.5	1.59	16.3	1.89	15.6	1.52	2270
45	6.23	41.1	586	72.7	6.65	82.3	32.8	9.34	1.57	14.5	1.88	15.6	1.11	1990
40	6.93	46.5	518	64.7	6.63	73.0	28.9	8.25	1.57	12.7	1.86	15.5	0.794	1730
36	8.12	48.1	448	56.5	6.51	64.0	24.5	7.00	1.52	10.8	1.83	15.4	0.545	1460
31	6.28	51.6	375	47.2	6.41	54.0	12.4	4.49	1.17	7.03	1.42	15.4	0.461	739
26	7.97	56.8	301	38.4	6.26	44.2	9.59	3.49	1.12	5.48	1.38	15.3	0.262	565
730	1.82	3.71	14300	1280	8.17	1660	4720	527	4.69	816	5.68	17.5	1450	362000
665	1.95	4.03	12400	1150	7.98	1480	4170	472	4.62	730	5.57	17.1	1120	305000

Problem. The following beam is W 16× 31 of A992 steel. It supports a reinforced concrete floor slab that provides continuous lateral support of compression flange. The service dead load is 450 lb/ft and service live load is 550 lb/ft. Does the beam has adequate moment strength?



Flange Check
From Table 1-1, $b_f = 5.53$ in, $t_f = 0.44$ in.

$$\lambda = \frac{b_f}{2t_f} = \frac{5.53}{2\times0.44} = 6.28$$

$$\lambda_p = 0.38\sqrt{E/F_y} = 0.38\sqrt{29000/50} = 9.15 > 6.28$$

Since, $\lambda < \lambda_p$ for flange, there is no local buckling in flange.

Web Check
From Table 1-1, $d = 15.9$ in, $k_{des} = 0.842$ in and $t_w = 0.275$ in

$$\lambda = \frac{h}{t_w} = \frac{d - 2k_{des}}{t_w} = \frac{15.9 - 2\times0.842}{0.275} = 51.7$$

$$\lambda_p = 3.76\sqrt{E/F_y} = 3.76\sqrt{29000/50} = 90.5 > 51.7$$

Since, $\lambda < \lambda_p$ for web, there is no local buckling in web either.

Determine Capacity

Since, both flange and web have no local buckling, the section can reach up to plastic moment before failure.
From Table 1-1, $Z_x = 54 \text{ in}^3$.

$$M_p = F_y Z_x = 50 \times 54 = 2700 \text{ k-in}$$
$$\phi_b M_n = \phi_b M_p = 0.9 \times 2700 \text{ k-in} = 2430 \text{ k-in} 202.5 \text{ k-ft}$$

Determine Demand

The dead load should be increased by self weight (31 lb/ft) of the beam since given dead load (450 lb/ft) is excluded of self weight.

$$w_D = 450 + 31 = 481 \text{ lb/ft}$$
$$w_L = 550 \text{ lb/ft}$$
$$w_u = 1.2 w_D + 1.6 w_L$$
$$= 1.2 \times 481 + 1.6 \times 550 = 1457 \text{ lb/ft} = 1.46 \text{ k/ft}$$
$$M_u = \frac{w_u L^2}{8} = \frac{1.46 \times 30^2}{8} = 164.3 \text{ k-ft} < \phi_b M_n$$

Since, $\phi_b M_n$(202.5 k-ft) > M_u(164.3 k-ft), the section W 16×31 has adequate moment capacity.

Ans. Yes. The beam has adequate moment strength.