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B4. MEMBER PROPERTIES

1. Classification of Sections for Local Buckling

For compression, sections are classified as nonslender element or *slender-element sections*. For a nonslender element section, the width-to-thickness ratios of its compression elements shall not exceed λ_r from Table B4.1a. If the width-to-thickness ratio of any compression element exceeds λ_r, the section is a slender-element section.

For flexure, sections are classified as *compact*, *noncompact* or slender-element sections. For a section to qualify as compact, its flanges must be continuously connected to the web or webs and the width-to-thickness ratios of its compression elements shall not exceed the limiting width-to-thickness ratios, λ_p, from Table B4.1b. If the width-to-thickness ratio of one or more compression elements exceeds λ_p, but does not exceed λ_r from Table B4.1b, the section is noncompact. If the width-to-thickness ratio of any compression element exceeds λ_r, the section is a slender-element section.

Compressive Element (Column, Brace)
- Non-slender Section
- Slender Section

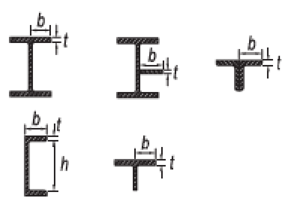
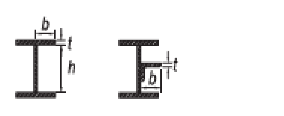
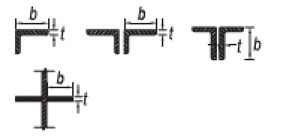
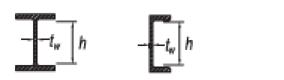
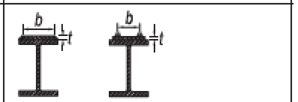
Flexural Element (Beam, Girder)
- Compact Section
- Non-Compact Section
- Slender Section

For H Shape Compressive Element,

$$\frac{b}{t} > 0.56 \cdot \sqrt{\frac{E}{F_y}} \Rightarrow \text{- Non-Slender Flange} \Rightarrow \text{Chapter E.3}$$

$$\frac{d}{t} > 1.49 \cdot \sqrt{\frac{E}{F_y}} \Rightarrow \text{- Non-Slender Web} \Rightarrow \text{Chapter E.3}$$

TABLE B4.1a
Width-to-Thickness Ratios: Compression Elements
Members Subject to Axial Compression

	Case	Description of Element	Width-to-Thickness Ratio	Limiting Width-to-Thickness Ratio λ_r (nonslender/slender)	Examples
Unstiffened Elements	1	Flanges of rolled I-shaped sections, plates projecting from rolled I-shaped sections; outstanding legs of pairs of angles connected with continuous contact, flanges of channels, and flanges of tees	b/t	$0.56\sqrt{\frac{E}{F_y}}$	
	2	Flanges of built-up I-shaped sections and plates or angle legs projecting from built-up I-shaped sections	b/t	$0.64\sqrt{\frac{k_cE}{F_y}}$ [a]	
	3	Legs of single angles, legs of double angles with separators, and all other unstiffened elements	b/t	$0.45\sqrt{\frac{E}{F_y}}$	
	4	Stems of tees	d/t	$0.75\sqrt{\frac{E}{F_y}}$	
Stiffened Elements	5	Webs of doubly-symmetric I-shaped sections and channels	h/t_w	$1.49\sqrt{\frac{E}{F_y}}$	
	6	Walls of rectangular HSS and boxes of uniform thickness	b/t	$1.40\sqrt{\frac{E}{F_y}}$	
	7	Flange cover plates and diaphragm plates between lines of fasteners or welds	b/t	$1.40\sqrt{\frac{E}{F_y}}$	
	8	All other stiffened elements	b/t	$1.49\sqrt{\frac{E}{F_y}}$	
	9	Round HSS	D/t	$0.11\frac{E}{F_y}$	

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E1. GENERAL PROVISIONS

The *design compressive strength*, $\phi_c P_n$, and the *allowable compressive strength*, P_n/Ω_c, are determined as follows.

The *nominal compressive strength*, P_n, shall be the lowest value obtained based on the applicable *limit states* of *flexural buckling*, *torsional buckling*, and *flexural-torsional buckling*.

$\phi_c = 0.90$ (LRFD) $\quad \Omega_c = 1.67$ (ASD)

$L_c := k \cdot L \qquad r := \sqrt{\dfrac{I}{A_g}} \qquad E := 200000\ MPa$

$$F_e := \frac{\pi^2 \cdot E}{\left(\dfrac{L_c}{r_x}\right)^2}$$

$$F_{cr} := \text{if}\left(\frac{L_c}{r_x} > 4.71 \cdot \sqrt{\frac{E}{F_y}}, 0.877 \cdot F_e, 0.658^{\frac{F_y}{F_e}} \cdot F_y\right)$$

$\phi := 0.9$

$\phi P_n := \phi \cdot F_{cr} \cdot A_g$

E3. FLEXURAL BUCKLING OF MEMBERS WITHOUT SLENDER ELEMENTS

This section applies to nonslender-element compression members, as defined in Section B4.1, for elements in axial compression.

User Note: When the torsional effective length is larger than the lateral effective length, Section E4 may control the design of wide-flange and similarly shaped columns.

The nominal compressive strength, P_n, shall be determined based on the limit state of flexural buckling:

$$P_n = F_{cr} A_g \qquad \text{(E3-1)}$$

The critical stress, F_{cr}, is determined as follows:

(a) When $\dfrac{L_c}{r} \leq 4.71\sqrt{\dfrac{E}{F_y}}$ (or $\dfrac{F_y}{F_e} \leq 2.25$)

$$F_{cr} = \left(0.658^{\frac{F_y}{F_e}}\right) F_y \qquad \text{(E3-2)}$$

(b) When $\dfrac{L_c}{r} > 4.71\sqrt{\dfrac{E}{F_y}}$ (or $\dfrac{F_y}{F_e} > 2.25$)

$$F_{cr} = 0.877 F_e \qquad \text{(E3-3)}$$

where

A_g = gross cross-sectional area of member, in.^{2} (mm^2)

E = modulus of elasticity of steel = 29,000 ksi (200 000 MPa)

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E3. FLEXURAL BUCKLING OF MEMBERS WITHOUT SLENDER ELEMENTS

This section applies to nonslender element compression members as defined in Section B4.1 for elements in uniform compression.

> **User Note:** When the torsional *unbraced length* is larger than the lateral unbraced length, Section E4 may control the design of wide flange and similarly shaped *columns*.

The *nominal compressive strength*, P_n, shall be determined based on the *limit state* of *flexural buckling*.

$$P_n = F_{cr} A_g \tag{E3-1}$$

The *critical stress*, F_{cr}, is determined as follows:

(a) When $\dfrac{KL}{r} \le 4.71\sqrt{\dfrac{E}{F_y}}$ (or $\dfrac{F_y}{F_e} \le 2.25$)

$$F_{cr} = \left[0.658^{\frac{F_y}{F_e}}\right] F_y \tag{E3-2}$$

(b) When $\dfrac{KL}{r} > 4.71\sqrt{\dfrac{E}{F_y}}$ (or $\dfrac{F_y}{F_e} > 2.25$)

$$F_{cr} = 0.877 F_e \tag{E3-3}$$

where

F_e = elastic *buckling* stress determined according to Equation E3-4, as specified in Appendix 7, Section 7.2.3(b), or through an elastic buckling analysis, as applicable, ksi (MPa)

$$F_e = \frac{\pi^2 E}{\left(\dfrac{KL}{r}\right)^2} \tag{E3-4}$$

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E2. EFFECTIVE LENGTH

The effective length, L_c, for calculation of member slenderness, L_c/r, shall be determined in accordance with Chapter C or Appendix 7,

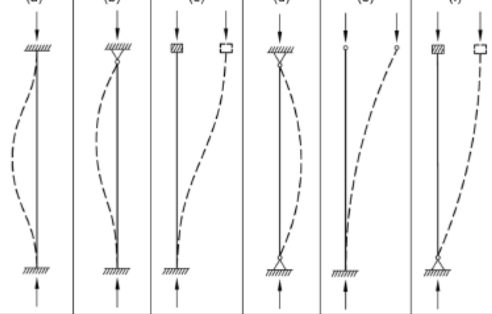
where

K = effective length factor
$L_c = KL$ = effective length of member, in. (mm)
L = laterally unbraced length of the member, in. (mm)
r = radius of gyration, in. (mm)

> **User Note:** For members designed on the basis of compression, the effective slenderness ratio, L_c/r, preferably should not exceed 200.

> **User Note:** The effective length, L_c, can be determined through methods other than those using the effective length factor, K.

TABLE C-A-7.1
Approximate Values of Effective Length Factor, *K*

	(a)	(b)	(c)	(d)	(e)	(f)
Buckled shape of column is shown by dashed line						
Theoretical *K* value	0.5	0.7	1.0	1.0	2.0	2.0
Recommended design value when ideal conditions are approximated	0.65	0.80	1.2	1.0	2.1	2.0
End condition code	Rotation fixed and translation fixed Rotation free and translation fixed Rotation fixed and translation free Rotation free and translation free					

E7. MEMBERS WITH SLENDER ELEMENTS

This section applies to slender-element compression members, as defined in Section B4.1 for elements in uniform compression.

The *nominal compressive strength*, P_n, shall be the lowest value based on the applicable *limit states* of *flexural buckling*, *torsional buckling*, and *flexural-torsional buckling*.

$$P_n = F_{cr} A_g \tag{E7-1}$$

The critical *stress*, F_{cr}, shall be determined as follows:

(a) When $\dfrac{KL}{r} \le 4.71\sqrt{\dfrac{E}{QF_y}}$ $\left(\text{or } \dfrac{QF_y}{F_e} \le 2.25\right)$

$$F_{cr} = Q\left[0.658^{\frac{QF_y}{F_e}}\right]F_y \tag{E7-2}$$

(b) When $\dfrac{KL}{r} > 4.71\sqrt{\dfrac{E}{QF_y}}$ $\left(\text{or } \dfrac{QF_y}{F_e} > 2.25\right)$

$$F_{cr} = 0.877F_e \tag{E7-3}$$

where

F_e = elastic *buckling stress*, calculated using Equations E3-4 and E4-4 for doubly symmetric members, Equations E3-4 and E4-5 for singly symmetric members, and Equation E4-6 for unsymmetric members, except for single angles with $b/t \le 20$, where F_e is calculated using Equation E3-4, ksi (MPa)

Q = net reduction factor accounting for all slender compression elements;
= 1.0 for members without slender elements, as defined in Section B4.1, for elements in uniform compression
= Q_sQ_a for members with *slender-element sections*, as defined in Section B4.1, for elements in uniform compression.

$$Q := Q_s \cdot Q_a$$

Q = 1.0 Non-Slender Section
(Section E.3 Applicable)

(a) When $\dfrac{KL}{r} \le 4.71\sqrt{\dfrac{E}{F_y}}$ $\left(\text{or } \dfrac{F_y}{F_e} \le 2.25\right)$

$$F_{cr} = \left[0.658^{\frac{F_y}{F_e}}\right]F_y$$

(b) When $\dfrac{KL}{r} > 4.71\sqrt{\dfrac{E}{F_y}}$ $\left(\text{or } \dfrac{F_y}{F_e} > 2.25\right)$

$$F_{cr} = 0.877F_e$$

Members with
Slender Flange Section

1. **Slender Unstiffened Elements, Q_s**

The reduction factor, Q_s, for slender *unstiffened elements* is defined as follows:

(a) For flanges, angles and plates projecting from rolled *columns* or other compression members:

(i) When $\frac{b}{t} \le 0.56\sqrt{\frac{E}{F_y}}$

$$Q_s = 1.0 \tag{E7-4}$$

(ii) When $0.56\sqrt{\frac{E}{F_y}} < \frac{b}{t} < 1.03\sqrt{\frac{E}{F_y}}$

$$Q_s = 1.415 - 0.74\left(\frac{b}{t}\right)\sqrt{\frac{F_y}{E}} \tag{E7-5}$$

(iii) When $\frac{b}{t} \ge 1.03\sqrt{\frac{E}{F_y}}$

$$Q_s = \frac{0.69E}{F_y\left(\frac{b}{t}\right)^2} \tag{E7-6}$$

Members with
Slender Web Section

2. **Slender Stiffened Elements, Q_a**

The reduction factor, Q_a , for slender *stiffened elements* is defined as follows:

$$Q_a = \frac{A_e}{A_g} \tag{E7-16}$$

where

A_g = gross cross-sectional area of member, in.^{2} (mm^2)

A_e = summation of the effective areas of the cross section based on the reduced *effective width*, b_e, in.^{2} (mm^2)

The reduced effective width, b_e, is determined as follows:

(a) For uniformly compressed slender elements, with $\frac{b}{t} \ge 1.49\sqrt{\frac{E}{f}}$, except flanges of square and rectangular sections of uniform thickness:

$$b_e = 1.92t\sqrt{\frac{E}{f}}\left[1 - \frac{0.34}{(b/t)}\sqrt{\frac{E}{f}}\right] \le b \tag{E7-17}$$

where

f is taken as F_{cr} with F_{cr} calculated based on $Q = 1.0$

(b) For flanges of square and rectangular *slender-element sections* of uniform thickness with $\frac{b}{t} \ge 1.40\sqrt{\frac{E}{f}}$:

$$b_e = 1.92t\sqrt{\frac{E}{f}}\left[1 - \frac{0.38}{(b/t)}\sqrt{\frac{E}{f}}\right] \le b \tag{E7-18}$$

where

$f = P_n/A_e$

User Note: In lieu of calculating $f = P_n/A_e$, which requires iteration, f may be taken equal to F_y. This will result in a slightly conservative estimate of *column available strength*.

EXAMPLE E.1A W-SHAPE COLUMN DESIGN WITH PINNED ENDS

Given:

Select a W-shape column to carry the loading as shown in Figure E.1A. The column is pinned top and bottom in both axes. Limit the column size to a nominal 14-in. shape. A column is selected for both ASTM A992 and ASTM A913 Grade 65 material.

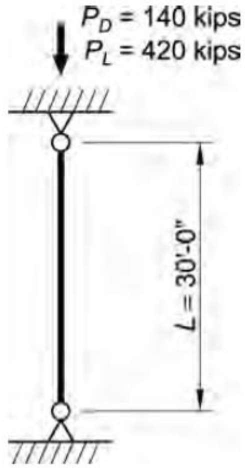


Fig. E.1A. Column loading and bracing

Given:

Verify a W14×90 is adequate to carry the loading as shown in Figure E.1B. The column is pinned top and bottom in both axes and braced at the midpoint about the *y-y* axis and torsionally. The column is verified for both ASTM A992 and ASTM A913 Grade 65 material.

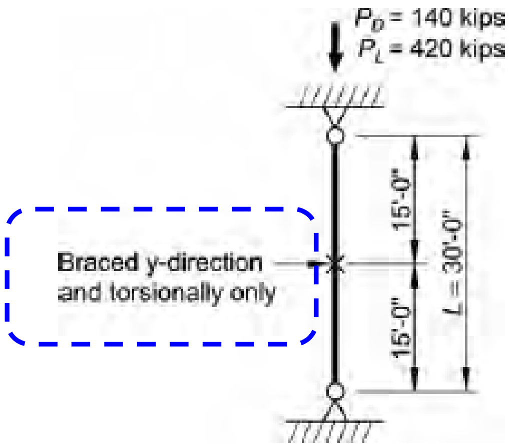


Fig. E.1B. Column loading and bracing.

$$1.2 \cdot 140\ kip + 1.6 \cdot 420\ kip = 840\ kip$$

$$1.2 \cdot 140\ kip + 1.6 \cdot 420\ kip = \left(3.737 \cdot 10^3\right)\ kN$$

To Do List

1. 압축에 대한 판 폭 두께비

   전체 부재에 대해 산정 (AISC KS BS EN)

1. 단일 부재에 대한 계산 과정 작성
2. 전체 부재에 대해 산정
3. Table (Python Pandas) OK/NG 작성
4. OK Table 작성
5. 중량 기준 오름차순 정렬
6. www.googChapter H. Design of Members

   for combined Forces and Torsion (Later)

(a) When $\frac{P_r}{P_c} \geq 0.2$

$$\frac{P_r}{P_c} + \frac{8}{9}\left(\frac{M_{rx}}{M_{cx}} + \frac{M_{ry}}{M_{cy}}\right) \leq 1.0 \qquad \text{(H1-1a)}$$

(b) When $\frac{P_r}{P_c} < 0.2$

$$\frac{P_r}{2P_c} + \left(\frac{M_{rx}}{M_{cx}} + \frac{M_{ry}}{M_{cy}}\right) \leq 1.0 \qquad \text{(H1-1b)}$$