DESIGN OF STEEL STRUCTURES

BASIC LECTURES

ON

STRUCTURAL STEEL

FOR

FOURTH STAGE

IN CIVIL ENGINEERING COLLEGE

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COMPRESSION MEMBERS

LECTURE # 04

COMPRESSION MEMBERS

AISC manual / Chapter B & E

Compression members are structural elements that subjected to compression forces. These forces applied along longitudinal axis through centered of the cross section.

The axial stress should be calculated using the following expression:

$$f = P / A$$

Type of columns according to failure shape:

1- Long column which failed by buckling.

3- Intermediate columns which failed by inelastic buckling.

2- Short column failed by yielding.

Buckling is a slight bent around one or more side of column. In our covered subject the buckling is around x or y axes of cross section.

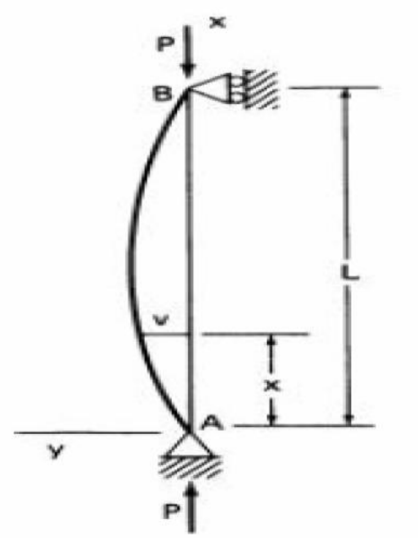
So the column will be effected by normal axial load & side deflection, therefore a moment should be calculated due to load x deflection distance.

$$M = P\Delta,$$

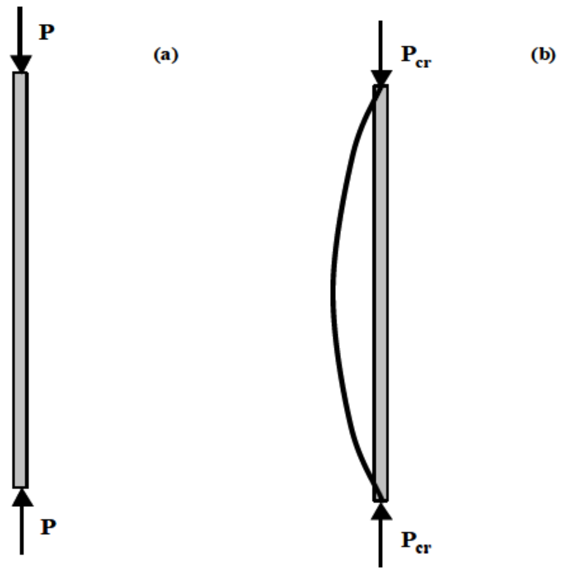
Euler Buckling of Columns:

Global buckling of a member happens when the member in compression becomes unstable due to its slenderness and load. Buckling can be elastic (longer thin members) or inelastic (shorter members). Here we shall derive the Euler buckling (critical) load for an elastic column.

Leonhard Euler (1707–1783).



Buckling or slight bent



Buckling occurs when a straight column subjected to axial compression suddenly undergoes bending as shown in the Figure (b). Buckling is identified as a failure limit-

Buckling of axially loaded compression members

The following differential equation giving the deflected shape of elastic member subjected to bending:

$$(d^2\Delta / dz^2) + P\Delta / EI = 0$$

-The expression given by Euler to calculate the buckling load or critical load for buckling is:

$$Pcr = \pi^2 E I / (L)^2$$

Where, I = moment of inertia about axis of buckling.

-Then the developed expression to calculate the critical buckling stress is:

$$Fcr = (Pcr / Ag) + (\pi^2 E / (L/r)^2).$$

Where: (L / r) is the slenderness ratio.

According to AISC manual chapter E, the basic expression for compression load is:

$Pu \leq Øc\ Pn = Øc\ (Ag\ Fcr)$ ……………………………….. (AISC / E3-1).

LRFD STRENGTH REDUCTION FACTOR
Øc = 0.9

Pn is the nominal compressive strength.

ØcPn is the design compressive strength.

Where Øc is the resistance factor for compression members & Fcr is the critical buckling stress (elastic or inelastic).

But the elastic buckling stress Fe is:

$Fe = Fcr = \frac{\pi^2 E}{(\frac{kL}{r})^2}$……………………………………. (AISC / E3-4).

Where, k = effective length factor based on end boundary conditions.

So the critical buckling load Pcr developed for columns is theoretically computed by:

$$Pcr = \frac{\pi^2 E I}{(Kl)^2}$$; Which represent critical buckling load.

$$Fcr = \frac{\pi^2 E}{(\frac{kL}{r})^2}$$; Which represent critical buckling stress.

Equation for Pcr is valid only when the material everywhere in the cross-section is in the elastic region. If the material goes inelastic then this Equation becomes useless and cannot be used.

Effective length:

The effective length factor, K, for calculation of member slenderness, KL/r, shall be determined in accordance with Chapter C or Appendix 7, where:

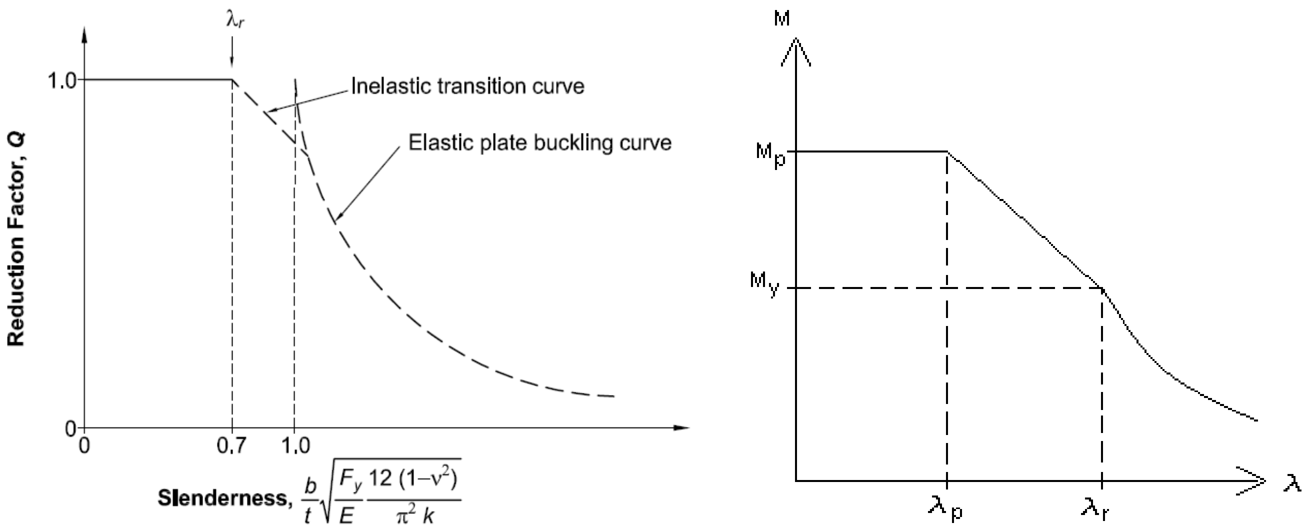
L = laterally unbraced length of the member, in.

r = radius of gyration, in.

Maximum effective slenderness ratio:

$$KL/r \le 200.$$

Effective length factors are given on page 16.1-189 of the AISC manual.



Relation of slenderness & reduction factor

General prevented & unprevented side sway:

- Braced frame (Side sway prevented) →1.0 ≥ K > 0.5
- Un-braced frame (Side sway Unperfected) unstable→ K > 1.0

Pinned connection: If Ic/Lc is large and Ig/Lg is small; the connection is more pinned.

Fixed connection: If (Ic/Lc) is small and (Ig/Lg) is large the connection is more fixed.

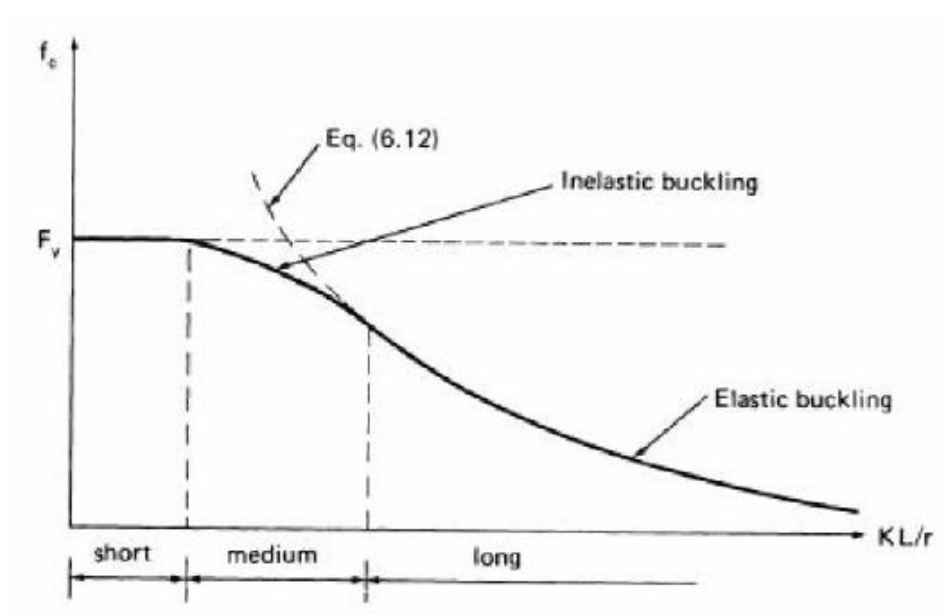
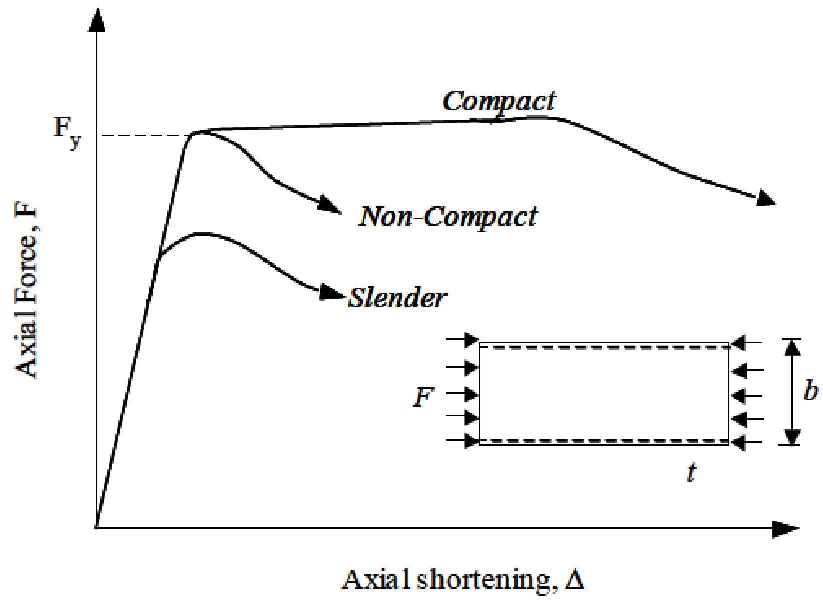


Figure show type of column related to kL/r



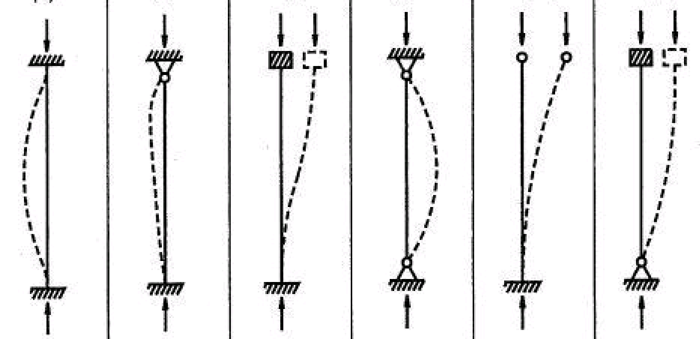
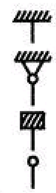
Local buckling behavior and classification of plate elements

Evaluation of K factor:

1- By table C.C2.1 AISC manual page
2- By Stiffness of joint.
3- By Monograph.

Table C.C2.1

Effective length factor K for axially loaded columns with various end conditions

Buckled shape of column is shown by dashed line	(a)	(b)	(c)	(d)	(e)	(f)
						
Theoretical K Value	0.5	0.7	1.0	1.0	2.0	2.0
Recommended design value when ideal condition are approximated	0.65	0.80	1.2	1.0	2.10	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					

In examples, home works, and exams, use the recommended design values of K.

For unbraced frames, the structure depends on its own bending stiffness for lateral stability. If a portal frame is not externally braced in its own plane to prevent side sway, the effective length KL is larger than the actual unbraced length, that is, K > 1. This will result in a reduction of the load-carrying capacity of the columns when sideways is not prevented.

For unbraced portal frames, the effective column length can be determined for the specific ratio of $(I / L)_{beam} / (I / L)_{col}$ and the condition of the foundation. If the actual footing provides a rotational restraint between hinged and fixed bases, the K value can be obtained by interpolation.

The K values to be used for the design of unbraced multistory or multibays frames can be obtained from the alignment chart.

G is defined as:

$$G = \frac{\Sigma Ic/Lc}{\Sigma Ib/Lb}$$

In which *Ic* is the moment of inertia and Lc is the unbraced length of the column, and *Ib* is the moment of inertia and Lb is the unbraced length of the beam.

In practical design:

- For hinged column end with ground surface, use G = 10 (because G in this case is infinity).
- For rigid & fixed end column, use G = 1.0 (because the real value is zero).

In the use of the chart, the beam stiffness Ib / Lb should be multiplied by a factor as follows when the conditions at the far end of the beam are known:

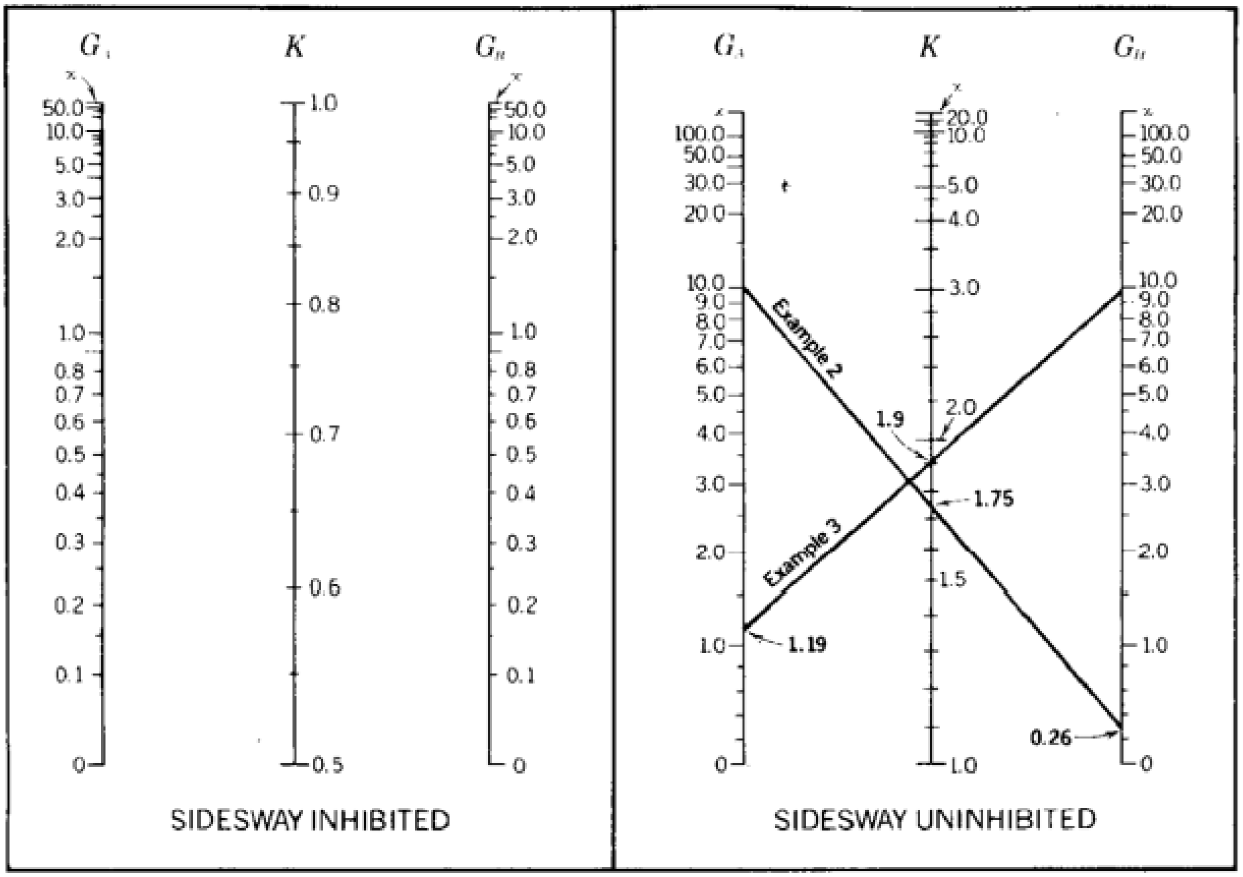
Factors consider for far end condition of beam (α)

Case	Hinged far end of beam	Fixed far end of beam
Side sway is prevented	1.5	2
Side sway is not prevented	0.5	0.67

So the stiffness ratio should be as follow:

$$G = \frac{Ic\ Lc}{\alpha\ Ib\ Lb}\ where\ \alpha\ is\ the\ considered\ far\ end\ condition\ of\ beam$$

Alignment charts (monograph)

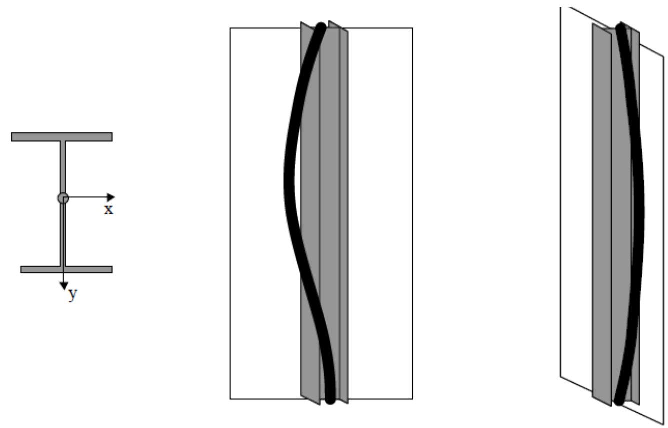


For pinned column base, use Gbottom =10

For fixed column base, use Gbottom=1.0

After determining G_A and G_B for joints A and B at two ends of the column section, the K value obtained from the alignment chart by constructing a straight line between the appropriate points on the scales for G_A and G_B.

EX: Determine the buckling strength of a W 12 x 50 column. Its length is 20 ft. For major axis buckling, it is pinned at both ends. For minor buckling, it is pinned at one end and fixed at the other end.



(a) Cross-section; (b) major-axis buckling; (c) minor-axis buckling

For the W12 x 50 (or any wide flange section), x is the major axis and y is the minor axis. Major axis means axis about which it has greater moment of inertia (Ix > Iy).

1- Sp, dim, properties:

Sec.	Ag	Iy	ry	Ix	rx	
W12x50		56.3		391		

According to Table C-C2.1 of the AISC Manual:

ky	Ly	kLy	kx	Lx	kLx
0.8	20	16	1.0	20	20

Critical load for buckling about x – axis = $Pcr - x = \frac{\pi^2 E\ Ix}{(kx\ Lx)^2} = 1940 kips$

Critical load for buckling about y – axis = $Pcr - y = \frac{\pi^2 E\ Iy}{(ky\ Ly)^2} = 682\ kips$

Buckling strength of the column = Smaller of (Pcr-x, Pcr-y) = Pcr-y = 682 kips

So Minor (y) axis buckling governs.

Let us find the critical stress Fcr = $\frac{Pcr}{A} = \frac{682}{14.6} = 47\ ksi$ which it is < Fy = 50 ksi, this mean that the critical stress within the elastic range for a 50 ksi yield stress material.

EX: A W10x30 is used as a 16ft long pin-connected column. Using the Euler expression. Use A36 steel material.

a) Determine the column's critical or buckling load.

b) Resolve the example using L= 8ft.

Solution:

1- Spec, dimensions & properties:

steel	Fy	Fu	sec	A	rx	ry	k	
A36	35	58	W10x	8.84	4.38	1.37	1.0	

2- Use smaller ry, to have larger value of $(\frac{kL}{r})$.

Slenderness $\frac{kL}{r} = \frac{16x12}{1.37} = 140.15 < 200$ ………………………….O.K

3- Buckling stress $Fe = \frac{\pi^2 E}{\left(\frac{kL}{ry}\right)^2} = 14.6\ ksi < 36\ ksi$ ……O.K

The limit between elastic and inelastic buckling is defined to be:

$$\frac{kL}{r} = 4.71\sqrt{\frac{E}{Fy}} \quad or \quad \frac{Fy}{Fe} = 2.25$$

These are the same as Fe = 0.44Fy that was used in the 2005 Specification.

For convenience, these limits are defined in Table C-E3.1 at 16.1 / page 260, for the common values of Fy.

TABLE C-E3.1
Limiting values of *KL/r* and F_e

F_y ksi (MPa)	Limiting $\frac{KL}{r}$	F_e ksi (MPa)
36 (250)	134	16.0 (111)
50 (345)	113	22.2 (153)
60 (415)	104	26.7 (184)
70 (485)	96	31.1 (215)

Figure show columns, beams & bracing

Local Buckling:

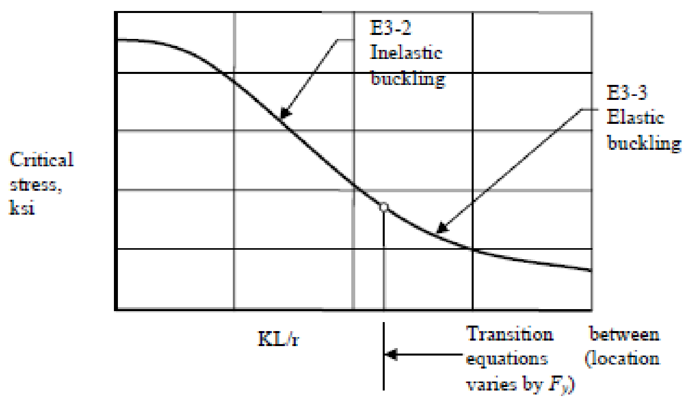
Local buckling limit state:

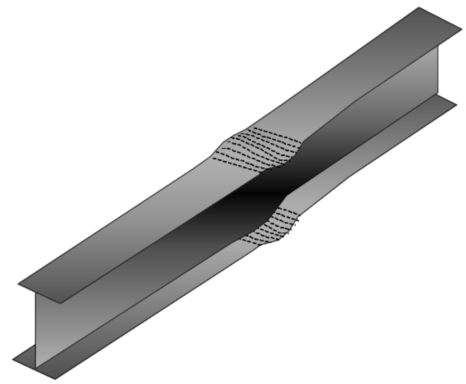
- The AISC specifications for column strength assume that column buckling is the governing limit state. However, if the column section is made of thin (slender) plate elements, then failure can occur due to local buckling of the flanges or the webs.

- If local buckling of the individual plate elements occurs, then the column may not be able to develop its buckling strength.
- Therefore, the local buckling limit state must be prevented from controlling the column strength.
- Local buckling depends on the slenderness (width-to-thickness b/t ratio) of the plate element and the yield stress (Fy) of the material.
- Each plate element must be stocky enough, i.e., have a b/t ratio that prevents local buckling from governing the column strength.

- The AISC specification B10 provides the slenderness (b/t) limits that the individual plate elements must satisfy so that local buckling does not control.

- The AISC specification provides two slenderness limits (λp and λr) for the local buckling of plate elements.





Relation of critical stress with effective slenderness ratio   local buckling of flange

Local buckling is instability due to the plates of the member becoming unstable. The local buckling of a member depends on its slenderness which is defined as the width-thickness ratio (b/t ratio), b is the width of the section and t is its thickness. Steel sections are classified as compact, noncom pact or slender depending on the width-thickness ratio of their elements.

Compact section: is capable of developing a fully plastic stress distribution and possess rotation capacity of approximately three before the onset of local buckling; i.e., local buckling is not an issue.

Noncom pact section: can develop the yield stress in compression elements before local buckling occurs, but will not resist inelastic local buckling at strain levels required for a fully plastic stress distribution. Local buckling can occur in the inelastic zone.

Compact sections have small b/t ratio and do not buckle locally; non compact section can buckle locally; slender sections have a large b/t ratio. Let us define the width-thickness ration of an element of the cross-section (flange or web of WF shapes) as

$$\lambda = b/t$$

Then the members are classified as follows:

Compact section: $\lambda \leq \lambda p$ for all elements.
Non compact sections: $\lambda p < \lambda \leq \lambda r$.
Slender: $\lambda > \lambda r$

The limiting values λp and λr for λ are given in Table B5.1 of the LRFD Specifications.

The strength corresponding to any buckling mode cannot be developed if the elements of the cross-section fail in local buckling. When b/t exceeds a limit λr (Table B5.1 of the LRFD Specifications), the member is classified as slender. Slender members can fail in local buckling resulting in reduced design strength. For slender members, Appendix B of the LRFD Specifications describes the reduction factors Q to be used for calculation of the critical stress Fcr.
Basically, the design strength needs to be reduced if the member is slender. Table B5.1 of the LRFD Specifications defines the following limits for sections that are not slender:
Unstiffened elements (flange): $\frac{bf}{2tf} \leq \lambda r$ where $\lambda r = 0.56\sqrt{E/Fy}$

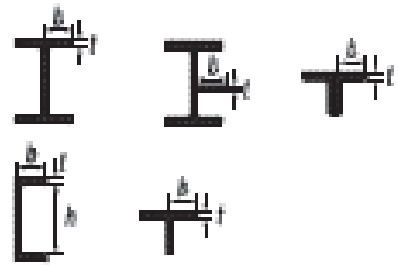
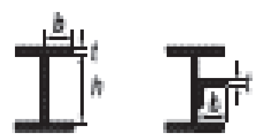
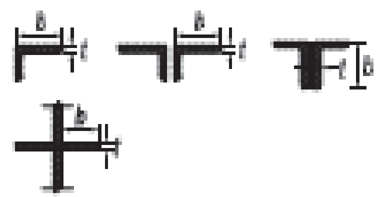
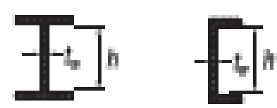
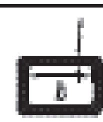
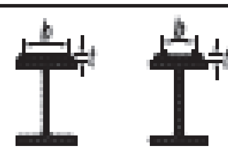
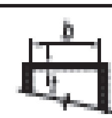
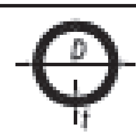
Stiffened element (web): $\frac{h}{tw} \leq \lambda r$ where $\lambda r = 1.49\sqrt{E/Fy}$

Note:

If the above values met the requirements so the member is not slender.

If the above values doesn't met the requirements so the member is slender.

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	$0.64\sqrt{\frac{k_c E}{F_y}}$ [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	$0.11\frac{E}{F_y}$	

Long, Short, and Intermediate Columns:

Columns are sometimes classed as being long, short, or intermediate. A brief discussion of each of these classifications is presented below.

1) **Long Columns:** the Euler formula predicts very well the strength of long columns where the axial buckling stress remains below the proportional limit. Such columns will buckle elastically. The slenderness ratio of long column is greater than 150, ($kL/r > 150$)

2) **Short Columns:** For very short columns, the failure stress will equal the yield stress and no buckling will occur. For a column to fall into this class, it would have to be so short as to have no practical application. Thus, no further reference is made to them here. The slenderness ratio of short column is smaller than 40, ($kL/r < 40$).

3) **Intermediate Columns:** Some of the fibers will reach the yield stress and some will not. The members will fail by both yielding and buckling, and their behavior is said to be inelastic. Most columns fall into this range. The slenderness ratio of intermediate column is between 40 and 150 , ($40 < kL/r < 150$).

Relation of moment to w/t ratio:

According to AISC-E3, The nominal strength (Pn) of Rolled compression members with slender sections is calculated as follow:

Pn = Ag Fcr ………………………………………………….E7-1

Øc Pn = Øc Fcr Ag … where Øc = 0.9 in LRFD

For inelastic columns :

- If (KL/r) ≤ 4.71 (√(E/Qfy)) 0r Fe ≥ 0.44QFy

So Fcr = Q (0.658*) Fy ………………………………………….E7-2

Where * = Q Fy/Fe

- If KL/r > 4.71 (√ (E/Q fy)) or Fe < 0.44 Q fy

So Fcr = 0.877Fe ……………………………………………E7-3

Where Fe is elastic critical buckling stress.

Q = 1.0 For members with compact or noncompact sections.

= Qs Qa for members with slender-element sections.

Two types of columns should be considered:

1- Unstiffened elements: unsupported along one edge parallel to the load direction, (AISC table B4.1, p 16.1-16).

2- Stiffened elements: supported along both edges parallel to load direction, (AISC table B4.1. p 16.1-17).

Steel shapes are classified as compact, non compact & slender. So section B4 of AISC manual provides limiting values of width / thickness (denoted by λ_r).

For slender unstiffened elements, Qs:

The reduction factor Qs is defined as follow:

A- For flanges, angles, & plates projecting from rolled columns or other compression members:

(i) When $b / t \leq 0.56 (\sqrt{E/F_y})$

So $Q_s = 1.0$ ……………………………………………………………E7-4

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

So $Q_s = 1.415 - 0.74 (b/t) (\sqrt{F_y/E})$ ……………………………….E7-5

(iii) When $b/t \geq 1.03 (\sqrt{/F_y})$

So $Q_s = 0.69 E / F_y (b/t)^2$ ……………………………………………E7-6

B- For flanges, angles, & plates projecting from built up columns or other compression members:

(i) When $b / t \leq 0.64 (\sqrt{EK_c/F_y})$

So: $Q_s = 1.0$ ………………………………………………………...E7-7

(ii) When $0.64 (\sqrt{(EK_c/F_y)}) < b / t < 1.17 (\sqrt{EK_c/F_y})$

So: $Q_s = 1.415 - 0.65 (b/t) (\sqrt{(F_y/EK_c)})$ ………………..............E7-8

(iii) When $b/t \geq 1.17 (\sqrt{(EK_c/F_y)})$

So: $Q_s = 0.9 E K_c / F_y (b/t)^2$ ……………………………........E7-9

Where $K_c = 4 / (\sqrt{h / t_w})$

C- for single angles:

(i) When $b / t \le 0.45 (\sqrt{E/F_y})$

So $Q_s = 1.0$ ……………………………………………………E7-10

- Slender stiffened elements, Q_a:
- Reduction factor, Q_a , for slender stiffened element is defined as follow:

$Q_a = A_e / A_g$

Where: A_g = gross area of section.

A_e = summation of the effective area of the cross section based on effective width, b_e.

- The reduced effective width b_e is determined as follow:

A - For uniformly compressed slender element, with

$b / t \ge 1.49 \sqrt{E/f}$ except flange of square & rectangular sections of uniform thickness

$b_e = 1.92 t \sqrt{E/f} [1- \{(0.34/(b/t)\} \sqrt{E/f}] \le b$ ………………..E7-17

Where f is taken as F_{cr} with F_{cr} calculated based ion Q = 1.0

B- For flanges of square & rectangular slender-element sections of uniform thickness with $b / t \ge 1.4 [\sqrt{E/f}]$:

$b_e = 1.92 t \sqrt{E/f} [1 – \{0.38 / (b/t)\} \sqrt{E/f}] \le b$ ……………E7-18

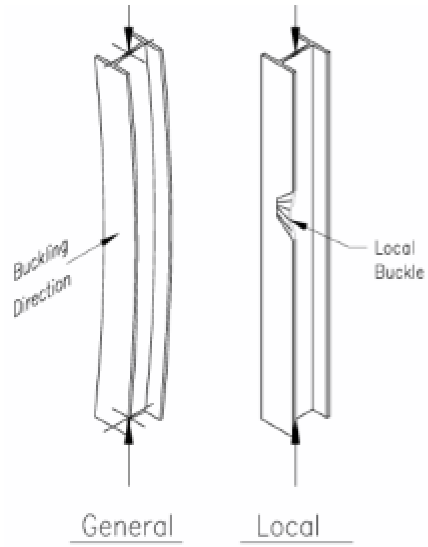
Where $f = P_n / A_e$ (note you may use $f = F_y$ the result will be slightly conservative

estimate of column strength.

- For circular axially loaded section:
- When $0.11 (E / F_y) < (D / t) < 0.45 (E / F_y)$:
- $Q = Q_a = 0.038 [E / F_y (D / t)] + 2/3$ ………………….E7-19

Where

D = outside diameter of round HSS & t = thickness of wall



Failure of column

How to Use Manual Table 4-2:

Design strength in axial compression is calculated as

- Table contains ØcPnc for various values of $K_y L_y$, assuming buckling about y-axis.
- How to check buckling about x-axis:

If buckling is about x-axis.

$ky\ Ly < \frac{kx\ Lx}{rx/ry}$ ….. So buckling is about x axis.

- How to read ØPnc if buckling is about x-axis:

Use the length as ($\frac{kx\ Lx}{rx/ry}$) in Table 4-2.

TABLE B4.1
Limiting Width-Thickness Ratios for Compression Elements

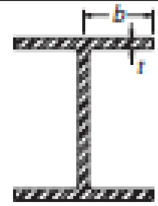
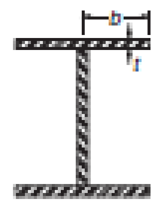
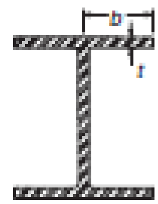
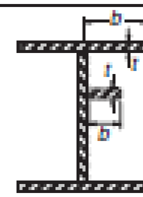
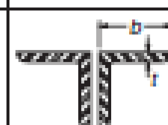
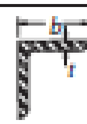
	Case	Description of Element	Width Thick-ness Ratio	Limiting Width-Thickness Ratios: λ_p (compact)	Limiting Width-Thickness Ratios: λ_r (noncompact)	Example
Unstiffened Elements	1	Flexure in flanges of rolled I-shaped sections and channels	b/t	$0.38\sqrt{E/F_y}$	$1.0\sqrt{E/F_y}$	
	2	Flexure in flanges of doubly and singly symmetric I-shaped built-up sections	b/t	$0.38\sqrt{E/F_y}$	$0.95\sqrt{k_c E/F_L}$ [a],[b]	
	3	Uniform compression in flanges of rolled I-shaped sections, plates projecting from rolled I-shaped sections; outstanding legs of pairs of angles in continuous contact and flanges of channels	b/t	NA	$0.56\sqrt{E/F_y}$	
	4	Uniform compression in flanges of built-up I-shaped sections and plates or angle legs projecting from built-up I-shaped sections	b/t	NA	$0.64\sqrt{k_c E/F_y}$ [a]	
	5	Uniform compression in legs of single angles, legs of double angles with separators, and all other unstiffened elements	b/t	NA	$0.45\sqrt{E/F_y}$	
	6	Flexure in legs of single angles	b/t	$0.54\sqrt{E/F_y}$	$0.91\sqrt{E/F_y}$	

TABLE B4.1 (cont.)
Limiting Width-Thickness Ratios for Compression Elements

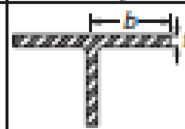
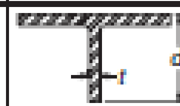
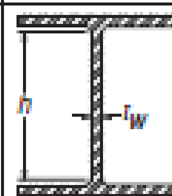
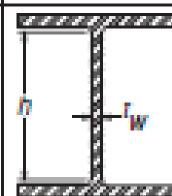
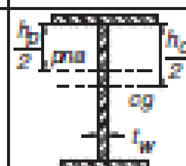
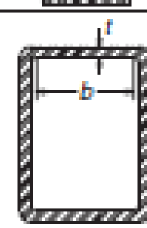
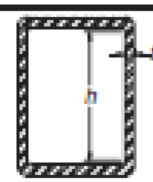
	Case	Description of Element	Width Thickness Ratio	Limiting Width-Thickness Ratios: λ_p (compact)	Limiting Width-Thickness Ratios: λ_r (noncompact)	Example
	7	Flexure in flanges of tees	b/t	$0.38\sqrt{E/F_y}$	$1.0\sqrt{E/F_y}$	
	8	Uniform compression in stems of tees	d/t	NA	$0.75\sqrt{E/F_y}$	
Stiffened Elements	9	Flexure in webs of doubly symmetric I-shaped sections and channels	h/t_w	$3.76\sqrt{E/F_y}$	$5.70\sqrt{E/F_y}$	
	10	Uniform compression in webs of doubly symmetric I-shaped sections	h/t_w	NA	$1.49\sqrt{E/F_y}$	
	11	Flexure in webs of singly-symmetric I-shaped sections	h_c/t_w	$\dfrac{\frac{h_c}{h_p}\sqrt{\frac{E}{F_y}}}{\left(0.54\frac{M_p}{M_y}-0.09\right)^2} \le \lambda_r$	$5.70\sqrt{E/F_y}$	
	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	$1.12\sqrt{E/F_y}$	$1.40\sqrt{E/F_y}$	
	13	Flexure in webs of rectangular HSS	h/t	$2.42\sqrt{E/F_y}$	$5.70\sqrt{E/F_y}$	

Table 4–22

Available Critical Stress for Compression Members

F_y = 35ksi			F_y = 36ksi			F_y = 42ksi			F_y = 46ksi			F_y = 50ksi		
$\frac{Kl}{r}$	F_{cr}/Ω_c ksi ASD	$\phi_c F_{cr}$ ksi LRFD	$\frac{Kl}{r}$	F_{cr}/Ω_c ksi ASD	$\phi_c F_{cr}$ ksi LRFD	$\frac{Kl}{r}$	F_{cr}/Ω_c ksi ASD	$\phi_c F_{cr}$ ksi LRFD	$\frac{Kl}{r}$	F_{cr}/Ω_c ksi ASD	$\phi_c F_{cr}$ ksi LRFD	$\frac{Kl}{r}$	F_{cr}/Ω_c ksi ASD	$\phi_c F_{cr}$ ksi LRFD
1	**21.0**	31.5	1	**21.6**	32.4	1	**25.1**	37.8	1	**27.5**	41.4	1	**29.9**	45.0
2	**21.0**	31.5	2	**21.6**	32.4	2	**25.1**	37.8	2	**27.5**	41.4	2	**29.9**	45.0
3	**20.9**	31.5	3	**21.5**	32.4	3	**25.1**	37.8	3	**27.5**	41.4	3	**29.9**	45.0
4	**20.9**	31.5	4	**21.5**	32.4	4	**25.1**	37.8	4	**27.5**	41.4	4	**29.9**	44.9
5	**20.9**	31.5	5	**21.5**	32.4	5	**25.1**	37.7	5	**27.5**	41.3	5	**29.9**	44.9
6	**20.9**	31.4	6	**21.5**	32.3	6	**25.1**	37.7	6	**27.5**	41.3	6	**29.9**	44.9
7	**20.9**	31.4	7	**21.5**	32.3	7	**25.1**	37.7	7	**27.5**	41.3	7	**29.8**	44.8
8	**20.9**	31.4	8	**21.5**	32.3	8	**25.1**	37.7	8	**27.4**	41.2	8	**29.8**	44.8
9	**20.9**	31.4	9	**21.5**	32.3	9	**25.0**	37.6	9	**27.4**	41.2	9	**29.8**	44.7
10	**20.9**	31.3	10	**21.4**	32.2	10	**25.0**	37.6	10	**27.4**	41.1	10	**29.7**	44.7
11	**20.8**	31.3	11	**21.4**	32.2	11	**25.0**	37.5	11	**27.3**	41.1	11	**29.7**	44.6
12	**20.8**	31.3	12	**21.4**	32.2	12	**24.9**	37.5	12	**27.3**	41.0	12	**29.6**	44.5
13	**20.8**	31.2	13	**21.4**	32.1	13	**24.9**	37.4	13	**27.2**	40.9	13	**29.6**	44.4
14	**20.7**	31.2	14	**21.3**	32.1	14	**24.8**	37.3	14	**27.2**	40.9	14	**29.5**	44.4
15	**20.7**	31.1	15	**21.3**	32.0	15	**24.8**	37.3	15	**27.1**	40.8	15	**29.5**	44.3
16	**20.7**	31.1	16	**21.3**	32.0	16	**24.8**	37.2	16	**27.1**	40.7	16	**29.4**	44.2
17	**20.7**	31.0	17	**21.2**	31.9	17	**24.7**	37.1	17	**27.0**	40.6	17	**29.3**	44.1
18	**20.6**	31.0	18	**21.2**	31.9	18	**24.7**	37.1	18	**27.0**	40.5	18	**29.2**	43.9
19	**20.6**	30.9	19	**21.2**	31.8	19	**24.6**	37.0	19	**26.9**	40.4	19	**29.2**	43.8
20	**20.5**	30.9	20	**21.1**	31.7	20	**24.5**	36.9	20	**26.8**	40.3	20	**29.1**	43.7
21	**20.5**	30.8	21	**21.1**	31.7	21	**24.5**	36.8	21	**26.7**	40.2	21	**29.0**	43.6
22	**20.4**	30.7	22	**21.0**	31.6	22	**24.4**	36.7	22	**26.7**	40.1	22	**28.9**	43.4
23	**20.4**	30.7	23	**21.0**	31.5	23	**24.3**	36.6	23	**26.6**	40.0	23	**28.8**	43.3
24	**20.3**	30.6	24	**20.9**	31.4	24	**24.3**	36.5	24	**26.5**	39.8	24	**28.7**	43.1
25	**20.3**	30.5	25	**20.9**	31.4	25	**24.2**	36.4	25	**26.4**	39.7	25	**28.6**	43.0
26	**20.2**	30.4	26	**20.8**	31.3	26	**24.1**	36.3	26	**26.3**	39.6	26	**28.5**	42.8
27	**20.2**	30.3	27	**20.7**	31.2	27	**24.0**	36.1	27	**26.2**	39.4	27	**28.4**	42.7
28	**20.1**	30.3	28	**20.7**	31.1	28	**24.0**	36.0	28	**26.1**	39.3	28	**28.3**	42.5
29	**20.1**	30.2	29	**20.6**	31.0	29	**23.9**	35.9	29	**26.0**	39.1	29	**28.2**	42.3
30	**20.0**	30.1	30	**20.6**	30.9	30	**23.8**	35.8	30	**25.9**	39.0	30	**28.0**	42.1
31	**20.0**	30.0	31	**20.5**	30.8	31	**23.7**	35.6	31	**25.8**	38.8	31	**27.9**	41.9
32	**19.9**	29.9	32	**20.4**	30.7	32	**23.6**	35.5	32	**25.7**	38.6	32	**27.8**	41.8
33	**19.8**	29.8	33	**20.4**	30.6	33	**23.5**	35.4	33	**25.6**	38.5	33	**27.7**	41.6
34	**19.8**	29.7	34	**20.3**	30.5	34	**23.4**	35.2	34	**25.5**	38.3	34	**27.5**	41.4
35	**19.7**	29.6	35	**20.2**	30.4	35	**23.3**	35.1	35	**25.4**	38.1	35	**27.4**	41.2
36	**19.6**	29.5	36	**20.1**	30.3	36	**23.2**	34.9	36	**25.2**	37.9	36	**27.2**	40.9
37	**19.5**	29.4	37	**20.1**	30.1	37	**23.1**	34.8	37	**25.1**	37.8	37	**27.1**	40.7
38	**19.5**	29.3	38	**20.0**	30.0	38	**23.0**	34.6	38	**25.0**	37.6	38	**26.9**	40.5
39	**19.4**	29.1	39	**19.9**	29.9	39	**22.9**	34.4	39	**24.9**	37.4	39	**26.8**	40.3
40	**19.3**	29.0	40	**19.8**	29.8	40	**22.8**	34.3	40	**24.7**	37.2	40	**26.6**	40.0

ASD	LRFD
$\Omega_c = 1.67$	$\phi_c = 0.90$

Table 4–22 (continued)

Available Critical Stress for Compression Members

Kl/r	F_y = 35ksi		Kl/r	F_y = 36ksi		Kl/r	F_y = 42ksi		Kl/r	F_y = 46ksi		Kl/r	F_y = 50ksi	
	F_{cr}/Ω_c	$\phi_c F_{cr}$		F_{cr}/Ω_c	$\phi_c F_{cr}$		F_{cr}/Ω_c	$\phi_c F_{cr}$		F_{cr}/Ω_c	$\phi_c F_{cr}$		F_{cr}/Ω_c	$\phi_c F_{cr}$
	ksi	ksi		ksi	ksi		ksi	ksi		ksi	ksi		ksi	ksi
	ASD	LRFD		ASD	LRFD		ASD	LRFD		ASD	LRFD		ASD	LRFD
41	19.2	28.9	41	19.7	29.7	41	22.7	34.1	41	24.6	37.0	41	26.5	39.8
42	19.2	28.8	42	19.6	29.5	42	22.6	33.9	42	24.5	36.8	42	26.3	39.5
43	19.1	28.7	43	19.6	29.4	43	22.5	33.7	43	24.3	36.6	43	26.2	39.3
44	19.0	28.5	44	19.5	29.3	44	22.3	33.6	44	24.2	36.3	44	26.0	39.1
45	18.9	28.4	45	19.4	29.1	45	22.2	33.4	45	24.0	36.1	45	25.8	38.8
46	18.8	28.3	46	19.3	29.0	46	22.1	33.2	46	23.9	35.9	46	25.6	38.5
47	18.7	28.1	47	19.2	28.9	47	22.0	33.0	47	23.8	35.7	47	25.5	38.3
48	18.6	28.0	48	19.1	28.7	48	21.8	32.8	48	23.6	35.4	48	25.3	38.0
49	18.5	27.9	49	19.0	28.5	49	21.7	32.6	49	23.4	35.2	49	25.1	37.7
50	18.4	27.7	50	18.9	28.4	50	21.6	32.4	50	23.3	35.0	50	24.9	37.5
51	18.3	27.6	51	18.8	28.3	51	21.4	32.2	51	23.1	34.8	51	24.8	37.2
52	18.3	27.4	52	18.7	28.1	52	21.3	32.0	52	23.0	34.5	52	24.6	36.9
53	18.2	27.3	53	18.6	28.0	53	21.2	31.8	53	22.8	34.3	53	24.4	36.7
54	18.1	27.1	54	18.5	27.8	54	21.0	31.6	54	22.6	34.0	54	24.2	36.4
55	18.0	27.0	55	18.4	27.6	55	20.9	31.4	55	22.5	33.8	55	24.0	36.1
56	17.9	26.8	56	18.3	27.5	56	20.7	31.2	56	22.3	33.5	56	23.8	35.8
57	17.7	26.7	57	18.2	27.3	57	20.6	31.0	57	22.1	33.3	57	23.6	35.5
58	17.6	26.5	58	18.1	27.1	58	20.5	30.7	58	22.0	33.0	58	23.4	35.2
59	17.5	26.4	59	17.9	27.0	59	20.3	30.5	59	21.8	32.8	59	23.2	34.9
60	17.4	26.2	60	17.8	26.8	60	20.2	30.3	60	21.6	32.5	60	23.0	34.6
61	17.3	26.0	61	17.7	26.6	61	20.0	30.1	61	21.4	32.2	61	22.8	34.3
62	17.2	25.9	62	17.6	26.5	62	19.9	29.9	62	21.3	32.0	62	22.6	34.0
63	17.1	25.7	63	17.5	26.3	63	19.7	29.6	63	21.1	31.7	63	22.4	33.7
64	17.0	25.5	64	17.4	26.1	64	19.6	29.4	64	20.9	31.4	64	22.2	33.4
65	16.9	25.4	65	17.3	25.9	65	19.4	29.2	65	20.7	31.2	65	22.0	33.0
66	16.8	25.2	66	17.1	25.8	66	19.2	28.9	66	20.5	30.9	66	21.8	32.7
67	16.7	25.0	67	17.0	25.6	67	19.1	28.7	67	20.4	30.6	67	21.6	32.4
68	16.5	24.9	68	16.9	25.4	68	18.9	28.5	68	20.2	30.3	68	21.4	32.1
69	16.4	24.7	69	16.8	25.2	69	18.8	28.2	69	20.0	30.1	69	21.1	31.8
70	16.3	24.5	70	16.7	25.0	70	18.6	28.0	70	19.8	29.8	70	20.9	31.4
71	16.2	24.3	71	16.5	24.8	71	18.5	27.7	71	19.6	29.5	71	20.7	31.1
72	16.1	24.2	72	16.4	24.7	72	18.3	27.5	72	19.4	29.2	72	20.5	30.8
73	16.0	24.0	73	16.3	24.5	73	18.1	27.2	73	19.2	28.9	73	20.3	30.5
74	15.8	23.8	74	16.2	24.3	74	18.0	27.0	74	19.1	28.6	74	20.1	30.2
75	15.7	23.6	75	16.0	24.1	75	17.8	26.8	75	18.9	28.4	75	19.8	29.8
76	15.6	23.4	76	15.9	23.9	76	17.6	26.5	76	18.7	28.1	76	19.6	29.5
77	15.5	23.3	77	15.8	23.7	77	17.5	26.3	77	18.5	27.8	77	19.4	29.2
78	15.4	23.1	78	15.6	23.5	78	17.3	26.0	78	18.3	27.5	78	19.2	28.8
79	15.2	22.9	79	15.5	23.3	79	17.1	25.8	79	18.1	27.2	79	19.0	28.5
80	15.1	22.7	80	15.4	23.1	80	17.0	25.5	80	17.9	26.9	80	18.8	28.2

ASD	LRFD
$\Omega_c = 1.67$	$\phi_c = 0.90$

Design procedure of steel column according to manual tables

Data:

- Column – length
- Support conditions
- Material properties – F_y
- Applied load - P_{actual}

Required:

- Column Size

1. Enter table with height.
2. Read allowable load for each section to find the smallest adequate size.
3. **Tables assume weak axis buckling. If the strong axis controls the length must be divide by the ratio r_x/r_y**
4. Values stop in table (black line) at slenderness limit, KL/r = 200

Analysis procedure of steel column according to manual tables

Data:

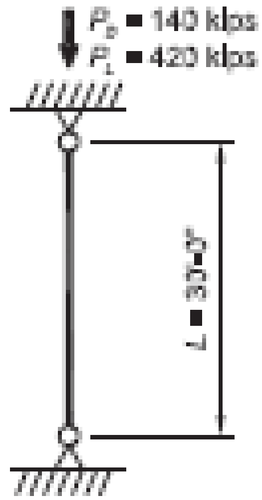
- Column – size, length
- Support conditions
- Material properties – F_y
- Applied load - P_{actual}

Required:

- $P_{actual} < P_{allowable}$

1. Calculate slenderness ratios. **largest ratio governs.**
2. In AISC Table look up $\mathbf{F_a}$ for given slenderness ratio.
3. Compute: $\mathbf{P_{allowable}} = F_a \quad A$.
4. Check column adequacy: $P_{actual} < P_{allowable}$

EX/ Given D.L = 140k, L.L = 420k & length of 30ft stand as shown. Select from W14 the required steel shapes. Assume A992 steel material.



- Solution:

1- The required compressive loading:

LRFD
Pu = 1.2 D.L + 1.6 L.L = 840kips

2- From AISC specification, select from table C.C2.1, → k=1.0.

For cases of same bracing on both axes, the smaller (ry) will govern, so (kL)y = 30ft.

From column tables search for available load that equal or exceed the required compressive load given above…represented by section W14x132..page

LRFD
ØPn = 893 > 840kips

F_y = 50 ksi

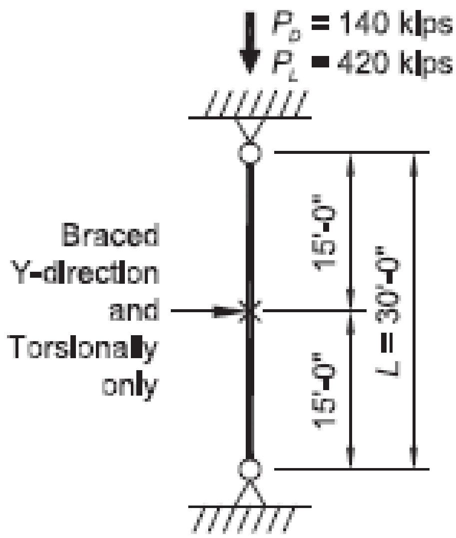
Table 4-1 (continued)
Available Strength in Axial Compression, kips
W-Shapes

W14

Shape	W14×											
lb/ft	145		132		120		109		99		90	
Design	P_n/Ω_c ASD	$\phi_c P_n$ LRFD	P_n/Ω_c ASD	$\phi_c P_n$ LRFD	P_n/Ω_c ASD	$\phi_c P_n$ LRFD	P_n/Ω_c ASD	$\phi_c P_n$ LRFD	P_n/Ω_c ASD	$\phi_c P_n$ LRFD	P_n/Ω_c ASD	$\phi_c P_n$ LRFD
0	1280	1920	1160	1750	1060	1590	958	1440	871	1310	793	1190
6	1250	1880	1130	1700	1030	1550	932	1400	848	1270	772	1160
7	1240	1860	1120	1680	1020	1530	923	1390	839	1260	764	1150
8	1230	1840	1110	1660	1010	1510	913	1370	830	1250	755	1140
9	1210	1820	1090	1640	994	1490	901	1350	819	1230	745	1120
10	1200	1800	1080	1620	980	1470	888	1340	807	1210	735	1100
11	1180	1770	1060	1600	965	1450	874	1310	794	1190	723	1090
12	1160	1750	1040	1570	948	1430	859	1290	780	1170	710	1070
13	1140	1720	1020	1540	931	1400	843	1270	766	1150	697	1050
14	1120	1690	1000	1510	912	1370	826	1240	750	1130	682	1030
15	1100	1650	982	1480	892	1340	808	1210	733	1100	667	1000
16	1080	1620	960	1440	872	1310	789	1190	716	1080	652	979
17	1060	1590	937	1410	850	1280	770	1160	698	1050	635	955
18	1030	1550	913	1370	828	1240	750	1130	680	1020	619	930
19	1010	1510	888	1330	805	1210	729	1100	661	994	601	903
20	980	1470	862	1300	782	1180	708	1060	642	964	583	877
22	927	1390	810	1220	734	1100	664	998	602	904	547	822
24	872	1310	756	1140	685	1030	620	931	561	843	509	766
26	816	1230	702	1060	635	955	574	863	519	781	472	709
28	759	1140	648	974	586	880	529	795	478	719	434	653
30	703	1060	594	893	537	807	485	729	438	658	397	597

Effective length, KL (ft), with respect to least radius of gyration, r_y

EX: Redesign the column from example given above, assuming the column is laterally braced about the y-y axis and torsionally braced at the midpoint.



Solution:

1- The required compressive loading is:

LRFD
Pu = 1.2(140 kips) + 1.6(420 kips) = 840 kips

2- From AISC Specification, table C.C2,1 for a pinned-pinned condition,→ K = 1.0.

Because the unbraced lengths differ in the two axes, select the member using the y-y axis then verify the strength in the x-x axis.

Enter AISC Manual Table 4-1 with a y-y axis effective length, KLy, of 15ft and proceed across the table until reaching a shape with an available strength that equals or exceeds the required strength. Try a W14x90. A 15 ft long W14x90 provides an available strength in the y-y direction of:

LRFD
ØcPn = 1,000 kips

The rx /ry ratio for this column, shown at the bottom of AISC Manual Table 4-1page 4-13, is 1.66. The equivalent y-y axis effective length for strong axis buckling is computed as:

$$\frac{kL}{rx/ry} = \frac{30ft}{1.66} = 18.1\ ft$$

Table 4-1 (continued)
Available Strength in Axial Compression, kips
W-Shapes

F_y = 50 ksi — W14

Shape	W14×											
lb/ft	145		132		120		109		99		90	
Design	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
0	1280	1920	1160	1750	1060	1590	958	1440	871	1310	793	1190
6	1250	1880	1130	1700	1030	1550	932	1400	848	1270	772	1160
7	1240	1860	1120	1680	1020	1530	923	1390	839	1260	764	1150
8	1230	1840	1110	1660	1010	1510	913	1370	830	1250	755	1140
9	1210	1820	1090	1640	994	1490	901	1350	819	1230	745	1120
10	1200	1800	1080	1620	980	1470	888	1340	807	1210	735	1100
11	1180	1770	1060	1600	965	1450	874	1310	794	1190	723	1090
12	1160	1750	1040	1570	948	1430	859	1290	780	1170	710	1070
13	1140	1720	1020	1540	931	1400	843	1270	766	1150	697	1050
14	1120	1690	1000	1510	912	1370	826	1240	750	1130	682	1030
15	1100	1650	982	1480	892	1340	808	1210	733	1100	667	1000
16	1080	1620	960	1440	872	1310	789	1190	716	1080	652	979
17	1060	1590	937	1410	850	1280	770	1160	698	1050	635	955
18	1030	1550	913	1370	828	1240	750	1130	680	1020	618	929
19	1010	1510	888	1330	805	1210	729	1100	661	994	601	903
20	980	1470	862	1300	782	1180	708	1060	642	964	583	877
22	927	1390	810	1220	734	1100	664	998	602	904	547	822
24	872	1310	756	1140	685	1030	620	931	561	843	509	766
26	816	1230	702	1060	635	955	574	863	519	781	472	709
28	759	1140	648	974	586	880	529	796	478	719	434	653
30	703	1060	594	893	537	807	485	729	438	658	397	597

Effective length, KL (ft), with respect to least radius of gyration, r_y

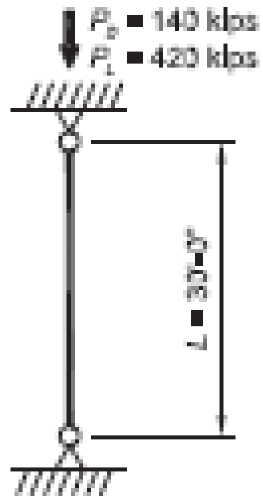
From AISC Manual Table 4-1page 4-13, the available strength of a W14x90 with an effective length of 18 ft is:

LRFD
ØcPn = 929 kips > 840 kips

The available compressive strength is governed by the x-x axis flexural buckling limit state.

The available strengths of the columns described in the above examples are easily selected directly from the AISC Manual Tables. The available strengths can also be verified by hand calculations, as shown in the following examples.

EX: Calculate the available strength of a W14x132 column with unbraced lengths of 30 ft in both axes. Use A992 steel material



Solution:

From AISC Manual Table 2-4, the material properties are as follows:

1- Sp, dim, & properties:

steel	Fy	Fu	sec	Ag	d	tw	bf	tf	L
A992	50	65	W14x132	38.8					30ft

rx	Ix	ry	Iy	rx/ry	kL	D.L	L.L		
6.28		3.76			30				

2- Slenderness Check:

From AISC Specification Commentary Table C-C2-1, for a pinned-pinned condition, K = 1.0.

Because the unbraced length is the same for both axes, the y-y axis will govern.

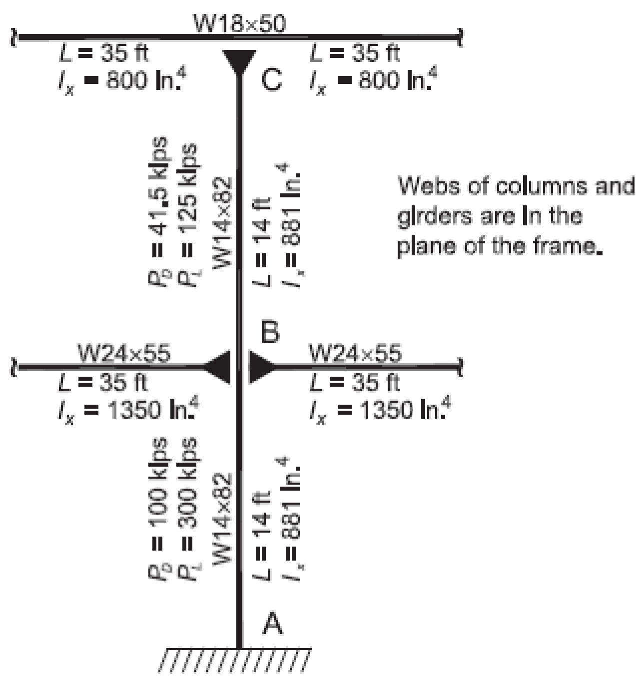
$$\frac{kL}{ry} = \frac{1x\ 30x12}{3.76} = 95.7$$

For F_y = 50 ksi, the available critical stresses, $\phi_c F_{cr}$ and F_{cr}/Ω_c for KL/r = 95.7 are interpolated from AISC Manual Table 4-22 as follows:

LRFD
$\phi_c F_{cr}$ = 23.0 ksi, $\phi_c P_n = A_g \times \phi_c F_{cr}$ = 38.8 in^2 x 23.0 ksi = 892 kips > 840 kips … O.K.

Note that the calculated values are approximately equal to the tabulated values.

EX: The member sizes shown for the moment frame illustrated here (side sway uninhibited in the plane of the frame) have been determined to be adequate for lateral loads. The material for both the column and the girders is ASTM A992. The loads shown at each level are the accumulated dead loads and live loads at that story. The column is fixed at the base about the x-x axis of the column.
Determine if the column is adequate to support the gravity loads shown. Assume the column is continuously supported in the transverse direction (the y-y axis of the column).



Solution:

1- Sp., dim., & properties:

steel	Fy	Fu	sec	Ix	sec	Ix	sec	Ag	Ix
A992	50	65	W18x50	800	W24x55	1,350	W14x82	24	882

2- Loading:

LRFD
Pu = 1.2(41.5) + 1.6(125) = 250 kips

3- Effective Length Factor:

Determine G_{top} and G_{bottom}:

$$G_{top} = \tau \frac{\sum (E_c I_c / L_c)}{\sum (E_g I_g / L_g)}$$

$$= (1.00) \frac{29,000 \text{ ksi} \left(\dfrac{881 \text{ in.}^4}{14.0 \text{ ft}} \right)}{2(29,000 \text{ ksi}) \left(\dfrac{800 \text{ in.}^4}{35.0 \text{ ft}} \right)}$$

$$= 1.38$$

$$G_{bottom} = \tau \frac{\sum (E_c I_c / L_c)}{\sum (E_g I_g / L_g)}$$

$$= (1.00) \frac{2(29,000 \text{ ksi}) \left(\dfrac{881 \text{ in.}^4}{14.0 \text{ ft}} \right)}{2(29,000 \text{ ksi}) \left(\dfrac{1,350 \text{ in.}^4}{35.0 \text{ ft}} \right)}$$

$$= 1.63$$

From the alignment chart, AISC Specification Commentary Figure C-A-7.2, (monograph) K is slightly less than 1.5; therefore use K = 1.5. Because the column available strength tables are based on the KL about the y-y axis, the equivalent effective column length of the upper segment for use in the table is:

$$KL = \frac{(KL)_x}{\left(\dfrac{r_x}{r_y} \right)}$$

$$= \frac{1.5(14.0 \text{ ft})}{2.44}$$

$$= 8.61 \text{ ft}$$

Take the available strength of the W14x82 from AISC Manual Table 4-1. At KL = 9 ft, the available strength in axial compression is:

LRFD
ØcPn = 940 kips > 250 kips O.K

Column A-B:

LRFD
Pu = 1.2(100 kips) + 1.6(300 kips) = 600 kips

Effective Length Factor

G_{bottom} = 1.0 (fixed) from AISC Specification Commentary Appendix 7, Section 7.2.

$$Gtop = \frac{\Sigma(\frac{EIc}{Lc})}{\Sigma(\frac{EIg}{Lg})} = \frac{\frac{2(29{,}000 \text{ x } 881)}{14}}{\frac{2(29{,}000 \text{ x } 1{,}350)}{35}} = 1.63$$

From the alignment chart, AISC Specification Commentary Figure C-A-7.2, (monograph) K is approximately 1.40. Because the column available strength tables are based on the KL about the y-y axis, the effective column length of the lower segment for use in the table is:

$$KL = \frac{(KL)_x}{\left(\frac{r_x}{r_y}\right)}$$

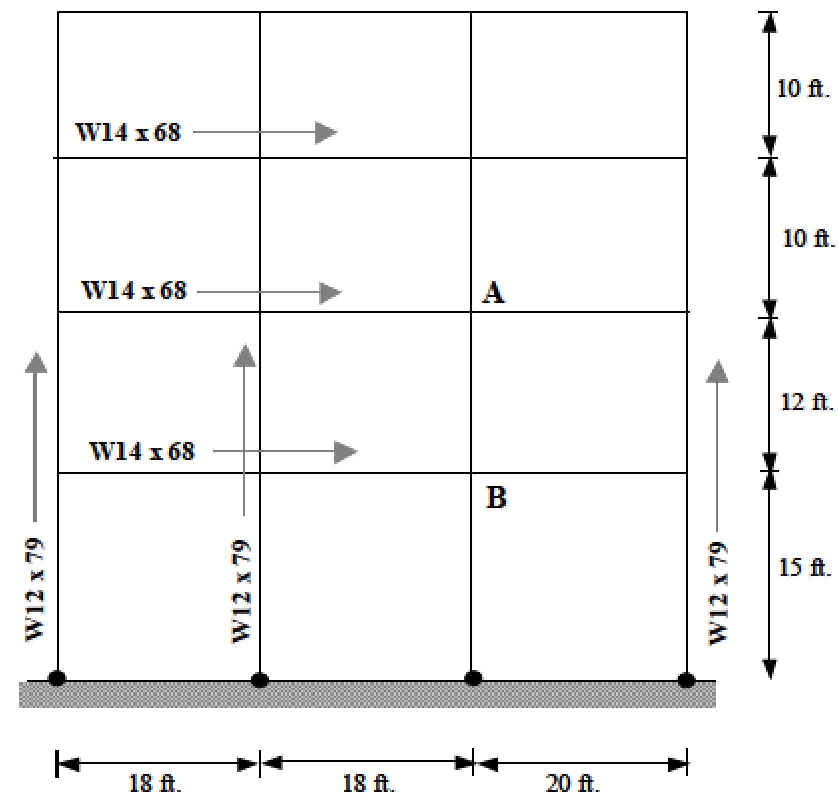
$$= \frac{1.40(14.0 \text{ ft})}{2.44}$$

$$= 8.03 \text{ ft}$$

Take the available strength of the W14x82 from AISC Manual Table 4-1. At KL = 9 ft, (conservative) the available strength in axial compression is:

LRFD
ØcPn = 940 kips > 600 kips O.K

EX: Calculate the effective length factor for the W12 x 79 column AB of the frame shown below. Assume that the column is oriented in such a way that major axis bending occurs in the plane of the frame. Assume that the columns are braced at each story level for out-of-plane buckling. Assume that the same column section is used for the stories above and below.



Solution:

1- Identify the frame type and calculate Lx, Ly, kx, and ky if possible.
It is an unbraced (side sway prevented) frame.
Lx = Ly = 12 ft.
ky = 1.0
kx depends on boundary conditions, which involve restraints due to beams and columns connected to the ends of column AB.
Need to calculate kx using alignment charts.

2- Calculate Kx

sec	Ix	sec	Ix
W 12 x 79	425	W14x68	753

GA = =1.021

GB= = 0.83

From Alignment Chart →Using GA and GB→ Kx = 1.3

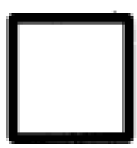
EX: Determine the design compressive strength for a pin-ended ($HSS8\times8\times3/8$) column of ASTM A500, Grade B steel with an unbraced length of 35-ft.

Solution:

1- Spec., Dimensions & Properties:

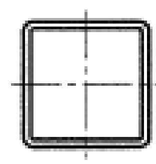
Steel	Fy	Fu	Sec	A	ry = rx	ky = kx	L	kLy	KL/r
A500	46			10.4	3.1	1	35	35	135.5

b / t	
19.9	



HSS16–HSS8

Table 1–12
Square HSS
Dimensions and Properties



Shape	Design Wall Thickness, t	Nominal Wt.	Area, A	b/t	h/t	I	S	r	Z	Workable Flat	Torsion J	Torsion C	Surface Area
	in.	lb/ft	in.^{2}			in.^{4}	in.^{3}	in.	in.^{3}	in.	in.^{4}	in.^{3}	ft^2/ft
HSS16×16×5/8	0.581	127.00	35.0	24.5	24.5	1370	171	6.25	200	13 9/16	2170	276	5.17
×1/2	0.465	103.00	28.3	31.4	31.4	1130	141	6.31	164	13 3/4	1770	224	5.20
×3/8	0.349	78.45	21.5	42.8	42.8	873	109	6.37	126	14 5/16	1350	171	5.23
×5/16	0.291	65.82	18.1	52.0	52.0	739	92.3	6.39	106	14 5/8	1140	144	5.25
HSS8×8×5/8	0.581	59.11	16.4	10.8	10.8	146	36.5	2.99	44.7	5 11/16	244	63.2	2.50
×1/2	0.465	48.72	13.5	14.2	14.2	125	31.2	3.04	37.5	5 3/4	204	52.4	2.53
×3/8	0.349	37.61	10.4	19.9	19.9	100	24.9	3.10	29.4	6 5/16	160	40.7	2.57
×5/16	0.291	31.79	8.76	24.5	24.5	85.6	21.4	3.13	25.1	6 5/8	136	34.5	2.58
×1/4	0.233	25.79	7.10	31.3	31.3	70.7	17.7	3.15	20.5	6 7/8	111	28.1	2.60
×3/16	0.174	19.61	5.37	43.0	43.0	54.4	13.6	3.18	15.7	7 3/16	84.5	21.3	2.62
×1/8	0.116	13.25	3.62	66.0	66.0	37.4	9.34	3.21	10.7	7 11/16	57.3	14.4	2.63

Note: For compactness criteria, refer to the end of Table 1-12.

Table 4–4 (continued)
Available Strength in Axial Compression, kips
Square HSS

HSS9–HSS8 $F_y = 46$ ksi

Shape	HSS9×9×				HSS8×8×							
	3/16[c]		1/8[c]		5/8		1/2		3/8		5/16	
t_{design}, in.	0.174		0.116		0.581		0.465		0.349		0.291	
Wt/ft	22.2		15.0		59.1		48.7		37.6		31.8	
Design	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
0	134	201	64.3	96.7	451	678	371	557	286	429	241	362
6	132	198	63.8	95.8	434	652	357	537	275	414	233	350
7	131	197	63.6	95.5	428	643	352	529	272	409	230	345
8	131	196	63.3	95.2	421	632	347	521	268	402	226	340
Effective l… 27	94.8	143	52.1	78.3	204	307	173	260	137	206	117	176
28	91.7	138	51.1	76.8	193	289	163	245	130	195	111	167
29	88.5	133	50.1	75.2	181	272	154	231	122	184	105	158
30	85.0	128	49.0	73.6	169	255	145	217	115	173	98.9	149
32	77.4	116	46.7	70.1	149	224	127	191	102	153	87.3	131
34	70.1	105	44.1	66.3	132	198	113	169	89.9	135	77.3	116
36	63.0	94.7	41.4	62.2	118	177	100	151	80.2	121	69.0	104
38	56.6	85.0	38.3	57.6	106	159	90.1	135	72.0	108	61.9	93.0
40	51.0	76.7	34.9	52.5	95.3	143	81.3	122	65.0	97.6	55.9	83.9
Properties												
A_g (in.^{2})	6.06		4.09		16.4		13.5		10.4		8.76	
$I_x = I_y$ (in.^{4})	78.2		53.5		146		125		100		85.6	
$r_x = r_y$ (in.^{4})	3.59		3.62		2.99		3.04		3.10		3.13	

ASD	LRFD	
$\Omega_c = 1.67$	$\phi_c = 0.90$	[c] Shape is slender for compression with $F_y = 46$ ksi.

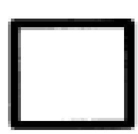
From AISC **Table 4-22**, with *KL/r* =135.5, and $F_y = 46$ ksi obtained *(ØcFcr)*=**12.3** ksi; therefore, the design strength is:

Table 4–22 (continued)
Available Critical Stress for Compression Members

F_y = 35ksi			F_y = 36ksi			F_y = 42ksi			F_y = 46ksi			F_y = 50ksi		
$\frac{Kl}{r}$	F_{cr}/Ω_c ksi ASD	$\phi_c F_{cr}$ ksi LRFD	$\frac{Kl}{r}$	F_{cr}/Ω_c ksi ASD	$\phi_c F_{cr}$ ksi LRFD	$\frac{Kl}{r}$	F_{cr}/Ω_c ksi ASD	$\phi_c F_{cr}$ ksi LRFD	$\frac{Kl}{r}$	F_{cr}/Ω_c ksi ASD	$\phi_c F_{cr}$ ksi LRFD	$\frac{Kl}{r}$	F_{cr}/Ω_c ksi ASD	$\phi_c F_{cr}$ ksi LRFD
121	9.91	14.9	121	10.0	15.0	121	10.2	15.4	121	10.3	15.4	121	10.3	15.4
122	9.79	14.7	122	9.85	14.8	122	10.1	15.2	122	10.1	15.2	122	10.1	15.2
123	9.67	14.5	123	9.72	14.6	123	9.93	14.9	123	9.94	14.9	123	9.94	14.9
124	9.55	14.3	124	9.59	14.4	124	9.78	14.7	124	9.78	14.7	124	9.78	14.7
125	9.43	14.2	125	9.47	14.2	125	9.62	14.5	125	9.62	14.5	125	9.62	14.5
126	9.31	14.0	126	9.35	14.0	126	9.47	14.2	126	9.47	14.2	126	9.47	14.2
127	9.19	13.8	127	9.22	13.9	127	9.32	14.0	127	9.32	14.0	127	9.32	14.0
128	9.07	13.6	128	9.10	13.7	128	9.17	13.8	128	9.17	13.8	128	9.17	13.8
129	8.95	13.4	129	8.98	13.5	129	9.03	13.6	129	9.03	13.6	129	9.03	13.6
130	8.83	13.3	130	8.86	13.3	130	8.89	13.4	130	8.89	13.4	130	8.89	13.4
131	8.71	13.1	131	8.73	13.1	131	8.76	13.2	131	8.76	13.2	131	8.76	13.2
132	8.60	12.9	132	8.61	12.9	132	8.63	13.0	132	8.63	13.0	132	8.63	13.0
133	8.48	12.7	133	8.49	12.8	133	8.50	12.8	133	8.50	12.8	133	8.50	12.8
134	8.37	12.6	134	8.37	12.6	134	8.37	12.6	134	8.37	12.6	134	8.37	12.6
135	8.25	12.4	135	8.25	12.4	135	8.25	12.4	135	8.25	12.4	135	8.25	12.4
136	8.13	12.2	136	8.13	12.2	136	8.13	12.2	136	8.13	12.2	136	8.13	12.2
137	8.01	12.0	137	8.01	12.0	137	8.01	12.0	137	8.01	12.0	137	8.01	12.0

So:
$\text{Ø}_c P_n = (\text{Ø}_c F_{cr}) A_g = (12.3)(10.4) = 128$ kips

Alternatively, the design strength could be obtained directly from the AISC column load Table 4-4. Enter the table with KL = 35-ft and obtain $\text{Ø}_c P_n$ = 128 kips.



HSS9–HSS8

Table 4–4 (continued)
Available Strength in Axial Compression, kips
Square HSS

F_y = 46 ksi

Shape		HSS9×9×				HSS8×8×							
		3/16[c]		1/8[c]		5/8		1/2		3/8		5/16	
t_{design}, in.		0.174		0.116		0.581		0.465		0.349		0.291	
Wt/ft		22.2		15.0		59.1		48.7		37.6		31.8	
Design		P_n/Ω_c ASD	$\phi_c P_n$ LRFD	P_n/Ω_c ASD	$\phi_c P_n$ LRFD	P_n/Ω_c ASD	$\phi_c P_n$ LRFD	P_n/Ω_c ASD	$\phi_c P_n$ LRFD	P_n/Ω_c ASD	$\phi_c P_n$ LRFD	P_n/Ω_c ASD	$\phi_c P_n$ LRFD
	0	134	201	64.3	96.7	451	678	371	557	286	429	241	362

Effective length	26	97.8	147	53.0	79.7	216	325	183	275	144	217	123	186
	27	94.8	143	52.1	78.3	204	307	173	260	137	206	117	176
	28	91.7	138	51.1	76.8	193	289	163	245	130	195	111	167
	29	88.5	133	50.1	75.2	181	272	154	231	122	184	105	158
	30	85.0	128	49.0	73.6	169	255	145	217	115	173	98.9	149
	32	77.4	116	46.7	70.1	149	224	127	191	102	153	87.3	131
	34	70.1	105	44.1	66.3	132	198	113	169	89.9	135	77.3	116
	36	63.0	94.7	41.4	62.2	118	177	100	151	80.2	121	69.0	104
	38	56.6	85.0	38.3	57.6	106	159	90.1	135	72.0	108	61.9	93.0
	40	51.0	76.7	34.9	52.5	95.3	143	81.3	122	65.0	97.6	55.9	83.9
Properties													
A_g (in.^{2})		6.06		4.09		16.4		13.5		10.4		8.76	
$I_x = I_y$ (in.^{4})		78.2		53.5		146		125		100		85.6	
$r_x = r_y$ (in.^{4})		3.59		3.62		2.99		3.04		3.10		3.13	

ASD	LRFD
$\Omega_c = 1.67$	$\phi_c = 0.90$

[c] Shape is slender for compression with $F_y = 46$ ksi.

Alternate Check:

$$\frac{b}{t} \leq 1.4\sqrt{\frac{E}{Fy}} = 19.9 < 35.1 \;\; O.K$$

Now determine the flexural buckling stress, Fcr:

$$4.71\sqrt{\frac{E}{Fy}} = 4.71\sqrt{\frac{29000}{46}} = 118.2$$

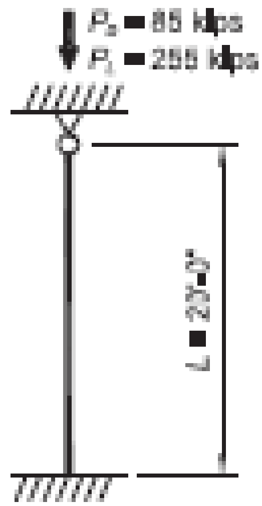
Since $\frac{kL}{ry} = 135.5 > 118.2 \rightarrow \; use\; Eq.(E7.3) for\; Fe$:

$$Fe = \frac{\pi^2 E}{(kl/r)^2} = 15.6\; ksi$$

Fcr = 0.877 Fe = 0.877(15.6) = 13.7 ksi

Øc Pn = Øc Fcr Ag = 0.9 (13.7)(10.4)= 128 kips

EX: Given an ASTM A500 Grade B compression member with a length of 20 ft, to support a dead load of 85 kips and live load of 255 kips in axial compression. The base is fixed and the top is pinned
Select, a suitable rectangular HSS.



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Solution:

1- Specification, dimensions & properties: From AISC *Manual* Table 2-4;

steel	Fy	Fu	PD	PL	L	Kx=ky	KL		
A500 GB	46	58	85	255	20				

2- Loading:

LRFD
Pu = 1.2PD + 1.6PL = 510kips

3- Using table Solution:

From AISC Specification Commentary Table C.C2.1, for a fixed-pinned condition, K = 0.8.

(KL)x = (KL)y = 0.8(20.0 ft) = 16.0 ft

Enter AISC Manual Table 4-3 for rectangular sections or AISC Manual Table 4-4 for square sections. Try an HSS12x10x3/8.

From AISC Manual Table 4-3, the available strength in axial compression is:

LRFD
Øc Pn = 518 kips > 510 kips ….O.K

4- Using equations calculations:

From AISC Manual Table 1-11, the geometric properties are as follows:

section	A	rx	ry	t_{dos}			
HSS12x10x⅜	14.6	4.61	4.04	0.394			

Slenderness Check:

Note: According to AISC Specification Section B4.1b, if the corner radius is not known, b and h shall be taken as the outside dimension minus three times the design wall thickness. This is generally a conservative assumption.

Calculate b/t of the most slender wall:

$$\lambda = \frac{h}{t} = \frac{12-3(0.349)}{0.349} = 31.4$$

Determine the wall limiting slenderness ratio, λr, from AISC Specification Table B4.1a Case 6:

$$\lambda r = 1.4\sqrt{\frac{E}{Fy}} = \sqrt{\frac{29000}{46}} = 35.2$$

$\because \lambda < \lambda r \longrightarrow \text{therefore, the section does not contain slender elements.}$

Because ry < rx and (KL)x = (KL)y, ry will govern the available strength.

$$\frac{kLy}{ry} = \frac{0.8\ x\ 20\ x\ 12}{4.01} = 47.9$$

$$4.4.71\sqrt{\frac{E}{Fy}} = 4.71\sqrt{\frac{29000}{46}} = 118 \geq$$

$47.9, \text{therefore, use AISC } \textit{Specification} \text{ Equation E3} - 2$

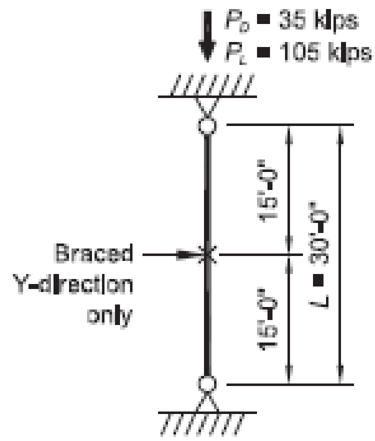
$$Fe = \frac{\pi^2 E}{(\frac{kL}{ry})} = 125ksi \qquad Fcr = \left((0.658)^{\frac{Fy}{Fe}}\right) Fy = 39.4ksi$$

Pn = Fcr x Ag = 39.4 x 14.6 = 575kips

From AISC Specification Section E1, the available compressive strength is:

LRFD Øc = 0.9
Øc Pn = 0.9 (575) = 518k > 510k

EX: Given an ASTM A53 Grade B Pipe compression member with a length of 30 ft to support a dead load of 35 kips and live load of 105 kips in axial compression. The column is pin-connected at the ends in both axes and braced at the midpoint in the y-y direction.
Select the suitable pipe section.



Solution:

1- Specification, dimensions & properties:

steel	Fy	Fu	PD	PL	Lu	L
A53GB	35	60	35	105	15	30

2- Loading:

LRFD
Pu = 1.2 x 35 +1.6 x 105 = 210kips

3- Using tables:

From AISC Specification Commentary Table C-C2-1, for a pinned-pinned condition, K = 1.0.
Therefore, (KL)x = 30.0 ft and (KL)y = 15.0 ft. Buckling about the x-x axis controls.
Enter AISC Manual Table 4-6 with a KL of 30 ft and proceed across the table until reaching the lightest section with sufficient available strength to support the required strength.
Try a 10-in. Standard Pipe.
From AISC Manual Table 4-6, the available strength in axial compression is:

LRFD
Øc Pn = 222kips > 210kips O.K

The available strength can be easily determined by using the tables of the AISC Manual. Available strength values can be verified by hand calculations, as follows.

4- Using equations calculations:

From AISC Manual Table 1-14, the geometric properties are as follows:

sec	Ag	r	$\lambda = \frac{D}{t}$	
Pipe 10 standard	11.5	3.68	31.6	

No Pipes shown in AISC Manual Table 4-6 are slender at 35 ksi, so no local buckling check is required; however, some round HSS are slender at higher steel strengths. The following calculations illustrate the required check.

Limiting Width-to-Thickness Ratio:

$$\lambda r = 0.11\ \frac{E}{Fy}\ from\ AISC\ specifi.table\ B4.1a\ case\ 9$$

$$\lambda r = 0.11\ \frac{29000}{35} = 91.1$$

$\lambda < \ \lambda r; therefore\ the\ pipe\ section\ is\ not\ slender.$

Finding critical stress Fcr;

$$\frac{kL}{r} = \frac{30\ x\ 12}{3.68} = 97.8$$

$$4.71\ \sqrt{\frac{E}{Fy}} = 4.71\ \sqrt{\frac{29000}{35}} = 136 \geq 97.8; (therefore\ AISC\ spec.apply\ E3-2).$$

$$\therefore\ Fe = \frac{\pi^2\ E}{(\frac{kL}{r})^2} = 29.9ksi$$

So ; $Fcr = \left((0.658)^{\frac{Fy}{Fe}}\right)\ Fy = 21.4ksi$

Nominal Compressive Strength:

Pn = Fcr x Ag = 21.4 x 11.5 = 246kips

From AISC Specification Section E1, the available compressive strength is:

LRFD Øc = 0.9
Øc Pn = 0.9 x 246 = 221k > 210k O.K

Thanks for listening, reading & Seriousness