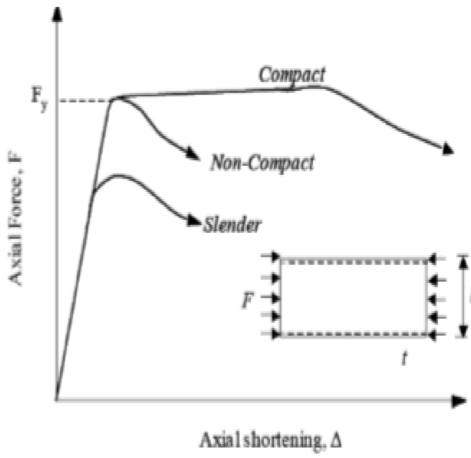
CLASSIFICATION OF SECTIONS FOR LOCAL BUCKLING

In section B4 of AISC, three possible local buckling stability parameters are defined: compact, non-compact, or slender. A compact section reaches its cross-sectional material strength, or capacity, before local buckling occurs. In a non-compact section, only apportion of the cross- sectional reaches its yield strength before local buckling occurs. In slender section the cross-sectional area doesn't yield and the strength of the member is governed by local buckling.

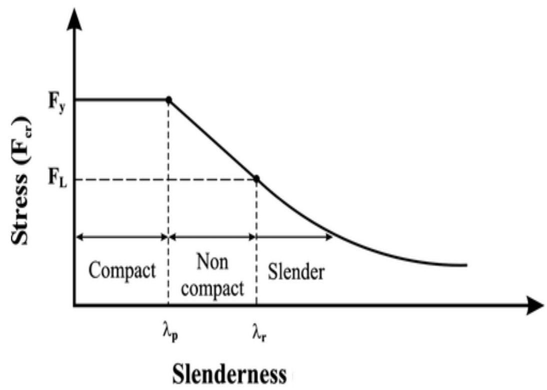
local buckling



For a section to qualify as compact its flanges must be continuously connected to the web or webs and the width-thickness ratios of its compression elements must not exceed the limiting width-thickness ratios λp from Table B4.1.

If the width thickness ratio of one or more compression elements exceeds λp, but does not exceed λr from Table B4.1, the section is non compact.

If the width-thickness ratio of any element exceeds λr, the section is referred to as a slender-element section



$\lambda = b/t$

$\lambda \geq \lambda p$ compact section

$\lambda p < \lambda \geq \lambda r$ non- compact section

$\lambda > \lambda r$ slender section

Stiffened and Unstiffened Elements

There are also two types of elements of a column section that are defined in the AISC Stiffened and Unstiffened elements.

For unstiffened elements supported along only one edge parallel to the direction of the compression force, the width shall be taken as follows

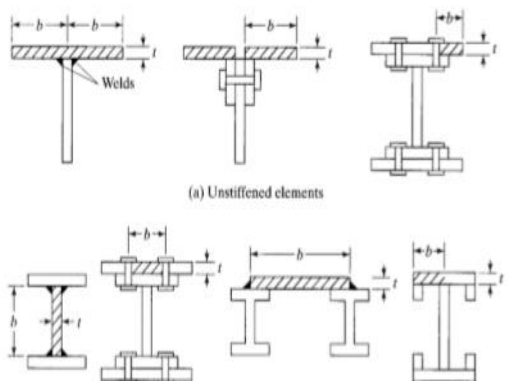
a) For flanges of I-shaped members and tees, the width b is one-half the full-flange width, bf

(b) For legs of angles and flanges of channels and zees, the width b is the full nominal dimension.

(c) For plates, the width b is the distance from the free edge to the first row of fasteners or line of welds.
(d) For stems of tees, d is taken as the full nominal depth of the section

Stiffened Elements

For stiffened elements supported along two edges parallel to the direction of the compression force, the width shall be taken as follows
a) For webs of rolled or formed sections, h is the clear distance between flanges less the fillet or corner radius at each flange; hc is twice the distance from the centroid to the inside face of the compression flange less the fillet or corner radius.
(b) For webs of built-up sections, h is the distance between adjacent lines of fasteners or the clear distance between flanges when welds are used, and hc is twice the distance from the centroid to the nearest line of fasteners at the compression flange or the inside face of the compression flange when welds are used; hp is twice the distance from the plastic neutral axis to the nearest line of fasteners at the compression flange or the inside face of the compression flange when welds are used.
(c) For flange or diaphragm plate's inbuilt-up sections, the width b is the distance between adjacent lines of fasteners or lines of welds.
(d) For flanges of rectangular hollow structural sections (HSS), the width b is the clear distance between webs less the inside corner radius on each side. For webs of rectangular HSS, h is the clear distance between the flanges less the inside corner radius on each side. If the corner radius is not known, b and h shall be taken as the corresponding outside dimension minus three times the thickness. The thickness, t, shall be taken as the design wall thickness.



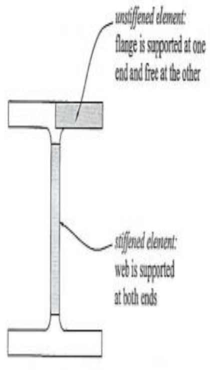
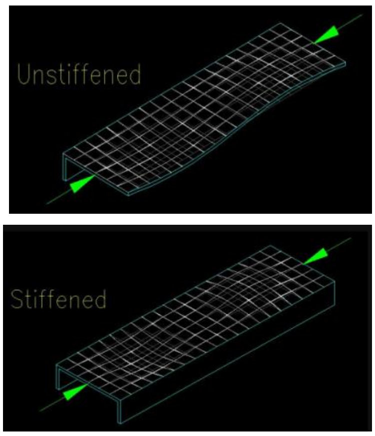
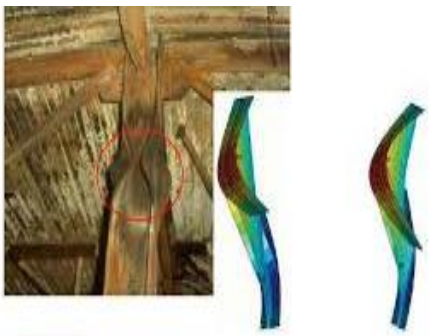


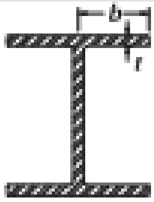
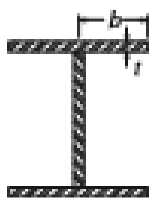
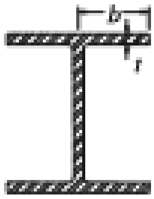
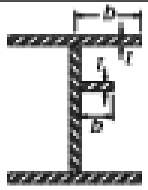
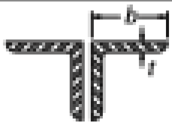
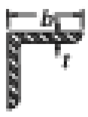
Figure 5-7 Stiffened and unstiffened elements.





Local buckling failures

TABLE B4.1
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
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	$0.54\sqrt{E/F_y}$	$0.91\sqrt{E/F_y}$	

Comp. member, also T section and Double angles without gusset plate

Built up section

Double angles with gusset plate

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

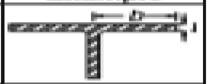
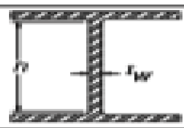
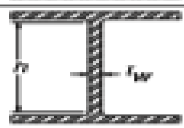
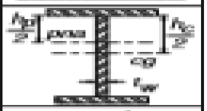
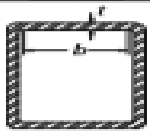
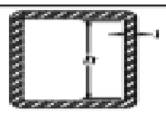
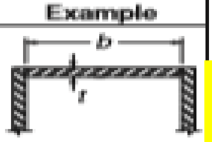
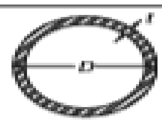
	Case	Description of Element	Width Thickness Ratio	Limiting Width-Thickness Ratios λ_p (compact)	λ_r (noncompact)	Example	
	7	Flexure in flanges of tees	b/t	$0.38\sqrt{E/F_y}$	$1.0\sqrt{E/F_y}$		
	8	Uniform compression in stems of tees	d/t	NA	$0.75\sqrt{E/F_y}$		Comp. member
Stiffened Elements	9	Flexure 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	$1.49\sqrt{E/F_y}$		Comp. member
	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	$1.12\sqrt{E/F_y}$	$1.40\sqrt{E/F_y}$		Comp. member
	13	Flexure in webs of rectangular HSS	h/t	$2.42\sqrt{E/F_y}$	$5.70\sqrt{E/F_y}$		

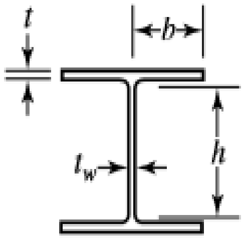
TABLE B4.1 (cont.)
Limiting Width-Thickness Ratios for Compression Elements

Case	Description of Element	Width Thickness Ratio	Limiting Width-Thickness Ratios λ_p (compact)	λ_r (noncompact)	Example	
14	Uniform compression in all other stiffened elements	b/t	NA	$1.49\sqrt{E/F_y}$		Comp. member
15	Circular hollow sections					
	In uniform compression	D/t	NA	$0.11\,E/F_y$		
	In flexure	D/t	$0.07\,E/F_y$	$0.31\,E/F_y$		

[a] $k_c = \frac{4}{\sqrt{h/t_w}}$, but shall not be taken less than 0.35 nor greater than 0.76 for calculation purposes. (See Cases 2 and 4)

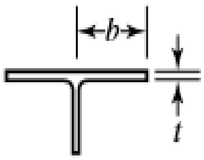
[b] $F_L = 0.7F_y$ for minor-axis bending, major axis bending of slender-web built-up I-shaped members, and major axis bending of compact and noncompact web built-up I-shaped members with $S_{xt}/S_{xc} \ge 0.7$; $F_L = F_yS_{xt}/S_{xc} \ge 0.5F_y$ for major-axis bending of compact and noncompact web built-up I-shaped members with $S_{xt}/S_{xc} < 0.7$. (See Case 2)

Summary of Table B 4.1 for compression member only

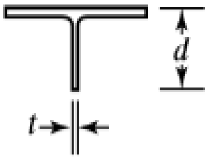


$b/t \le 0.56\sqrt{E/F_y}$

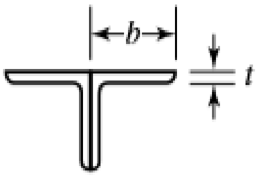
$h/t_w \le 1.49\sqrt{E/F_y}$



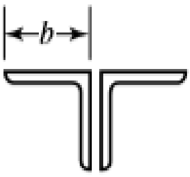
$b/t \le 0.56\sqrt{E/F_y}$



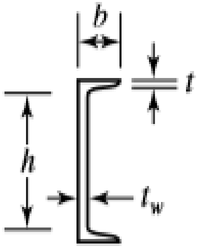
$d/t \le 0.75\sqrt{E/F_y}$



$b/t \le 0.56\sqrt{E/F_y}$

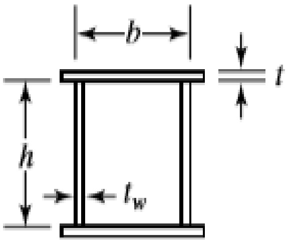


$b/t \le 0.45\sqrt{E/F_y}$



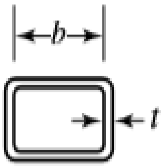
$b/t \le 0.56\sqrt{E/F_y}$

$h/t_w \le 1.49\sqrt{E/F_y}$

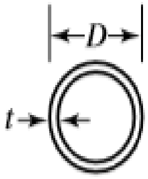


$b/t \le 1.40\sqrt{E/F_y}$

$h/t_w \le 1.40\sqrt{E/F_y}$



$b/t \le 1.40\sqrt{E/F_y}$



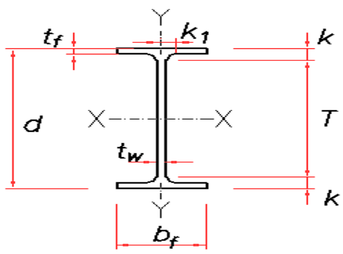
$D/t \le 0.11\, E/F_y$

Example

Check the local buckling stability for W10x 30, W12x50, W16x50, use A992 steel material

Section	bf/2tf	h/tw
W10x 30	**5.70**	**29.5**
W12x50	**6.31**	**26.8**
W16x50	**5.61**	**37.1**

from manual chapter 1



Steel	fy	fu
A992	50	65

-1-W10x 30 For flange

λr = $0.56\sqrt{E/fy}$

$0.56\sqrt{29000/50}$

5.70 < 13.486 ……. ok

For web

λr = $1.49\sqrt{E/fy}$

$1.49\sqrt{29000/50}$

29.5 < 35.88 ……. ok

Local instability is not a problem for W10x30

-2- W12x50 For flange

λr = $0.56\sqrt{E/fy}$

$0.56\sqrt{29000/50}$

6.31 < 13.486 ……. ok

For web

λr = $1.49\sqrt{E/fy}$

$1.49 \times \sqrt{29000/50}$

26.8< 35.88 ……. ok

Local instability is not a problem for W12x50

-3-W16x50^c For flange

λr = $0.56\sqrt{E/fy}$

$0.56\sqrt{29000/50}$

5.61 < 13.486 ……. ok

For web

λr = $1.49\sqrt{E/fy}$

$1.49\sqrt{29000/50}$

37.4 > 35.88 ……. Slender web

Local instability is a problem for W16x50

(Slender section)

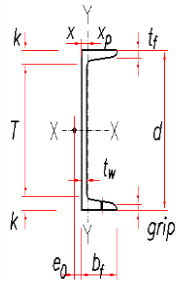
Note

When you ^c mark above the section that mean the section is slender according to fy of the section
W 16x50 ^c (c shape is slender for compression with fy =50)
We can observed that every section have ^c mark is slender section directly and other sections with no ^c mark are not slender section

Example

Check the local buckling stability for C12x 30, use A36 steel material

Steel	fy	fu
A36	36	58



Section	d	bf	tf	tw	k	T	
C12x 30	**12**	**3.17**	**0.5**	**0.51**	**1 $^{1/8}$**	**9 $^{3/4}$**	**from manual chapter 1**

For flange

λ =b/t = 3.17/0.5=6.34

λr = $0.56\sqrt{29000/36}$

=15.89

6.34>15.89 …..ok

For web

λ =h/tw=T/tw

h = d-2k

=12-2x (1 $^{1/8}$) = 9.75 =T=(d-2k)

= 0.51/9.75 =19.12

λr = $1.49\sqrt{29000/36}$

= 42.29

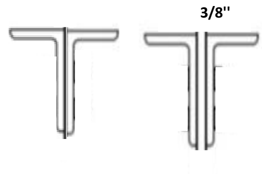
9.75 <42.29 …ok

Local instability is not a problem for C12x30

Example

Check the local buckling stability for 2L6x4x1/2 without gusset plate LLBB and 2L6x4x1/2 with 3/8'' gusset plate LLBB, use A36 steel material

Steel	fy	fu
A36	36	58



Section	b	t	
2L 6x4x1/2	4	0.5	without gusset plate
2 L 6x4x1/2	4	0.5	with3/8 gusset plate

λ =b/t = 4/0.5=8

λr = $0.45\sqrt{29000/36}$

8 < 12.7 …ok Local instability is not a problem for double angle with gusset plate

λ =b/t = 4/0.5=8

λr = $0.56\sqrt{29000/36}$

8 < 15.89 …ok Local instability is not a problem for double angle without gusset plate