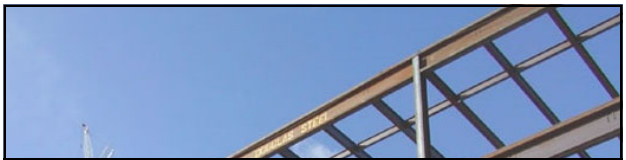
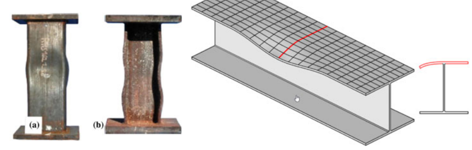


Chapter 4
Compression Members

1

Chapter 4 – Local Stability

- The strength corresponding to any overall ***buckling mode***, however, such as ***flexural buckling***, cannot be developed if the elements of the cross-section are so thin that ***local buckling occurs.***
- This type of instability is a ***localized buckling*** or ***wrinkling*** at an isolated location.



- If it occurs, the cross-section is no longer fully effective, and the member has failed.

2

Chapter 4 – Local Stability

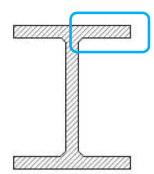
- ***I-shaped cross sections*** with thin flanges or webs are susceptible to this phenomenon, and their use should be avoided whenever possible.
- Otherwise, the compressive strength given by ***AISC Equations E3-2 and E3-3*** must be ***reduced***.
- The measure of this susceptibility is the ***width-to-thickness*** ratio of each cross-sectional element.

3

Chapter 4 – Local Stability

- Two types of elements must be considered:

  1. ***unstiffened elements,*** which are unsupported along one edge parallel to the direction of load

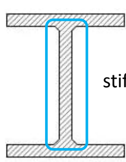


4

Chapter 4 – Local Stability

- Two types of elements must be considered:

  1. ***unstiffened elements,*** which are unsupported along one edge parallel to the direction of load
  2. ***stiffened elements,*** which are supported along both edges.



5

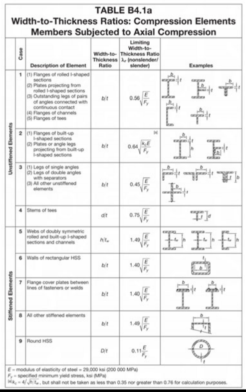
Chapter 4 – Local Stability

- For compression members, shapes are classified as ***slender*** or ***nonslender***.
- If a shape is ***slender***, its strength limit state is ***local buckling***, and the corresponding ***reduced strength must be computed***.
- The ***width-to-thickness ratio*** is given the generic symbol λ
- If λ is greater than a specific limit λ_r, the shape is ***slender***.
- If $\lambda < \lambda_r$ the shape is ***nonslender***.

6

Chapter 4 – Local Stability

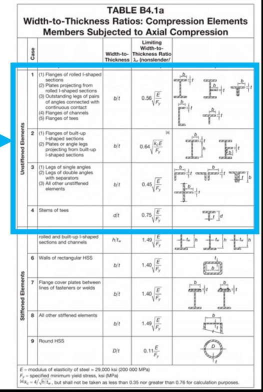
- **AISC Table B4.1a** shows the upper limit λ_r for ***nonslender*** members of various cross-sectional shapes.
- The table is divided into two parts: ***unstiffened*** elements and ***stiffened*** elements.
- For beams, a shape can be ***compact***, ***noncompact***, or ***slender***, and the limiting values are λ given in **AISC Table B4.1b**.
- We cover ***beams*** in ***Chapter 5***.



7

Chapter 4 – Local Stability

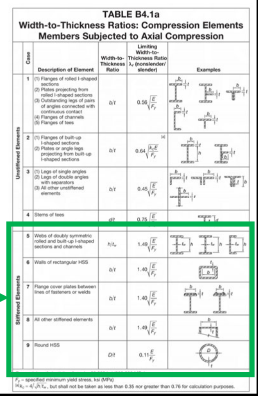
- **AISC Table B4.1a** is two parts: ***Unstiffened*** and ***Stiffened***.
- ***Unstiffened***



8

Chapter 4 – Local Stability

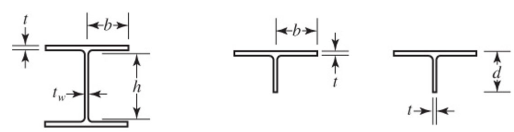
- **AISC Table B4.1a** is two parts: ***Unstiffened*** and ***Stiffened***.
- ***Unstiffened***
- ***Stiffened***



9

Chapter 4 – Local Stability

- The appropriate compression member limit, λ_r, from **AISC B4.1** is given for each case.



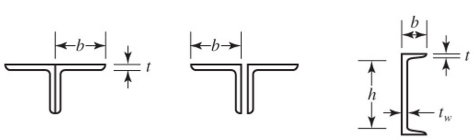
$b/t = 0.56\sqrt{E/F_y}$ $\quad$ $b/t = 0.56\sqrt{E/F_y}$ $\quad$ $d/t = 0.75\sqrt{E/F_y}$

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

10

Chapter 4 – Local Stability

- The appropriate compression member limit, λ_r, from **AISC B4.1** is given for each case.



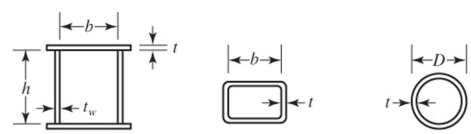
$b/t = 0.56\sqrt{E/F_y}$ $\quad$ $b/t = 0.45\sqrt{E/F_y}$ $\quad$ $b/t = 0.56\sqrt{E/F_y}$

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

11

Chapter 4 – Local Stability

- The appropriate compression member limit, λ_r, from **AISC B4.1** is given for each case.



$b/t = 1.40\sqrt{E/F_y}$ $\quad$ $b/t = 1.40\sqrt{E/F_y}$ $\quad$ $D/t = 0.11\left(E/F_y\right)$

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

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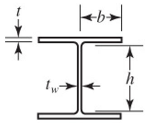
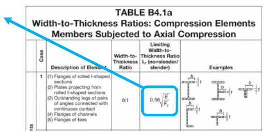
Chapter 4 – Local Stability

- For ***I shapes***, the projecting flange is considered to be an ***unstiffened*** element, and its width can be taken as half of the full nominal width.
- Using ***AISC*** notation gives: $\lambda = \dfrac{b}{t} = \dfrac{b_f/2}{t_f} = \dfrac{b_f}{2t_f}$

  where $\boldsymbol{b_f}$ and $\boldsymbol{t_f}$ are the width and thickness of the flange.

- The upper limit is: $\lambda = 0.56\sqrt{\dfrac{E}{F_y}}$





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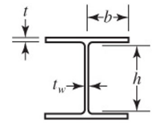
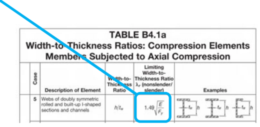
Chapter 4 – Local Stability

- For ***I shapes***, the web is a ***stiffened*** element, and its width is the distance between the roots of the flanges.
- The width-to-thickness ratio is: $\lambda = \dfrac{h}{t_w}$

  where $\boldsymbol{h}$ is the distance between the roots of the flanges, and $\boldsymbol{t_w}$ is the web thickness.

- The upper limit is: $\lambda = 1.49\sqrt{\dfrac{E}{F_y}}$

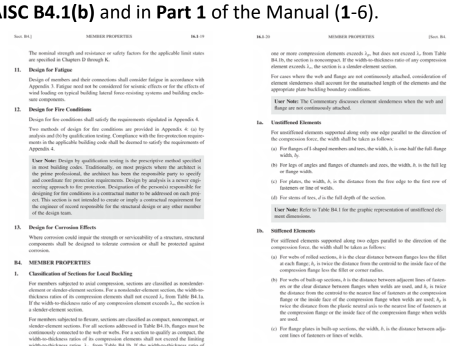




14

Chapter 4 – Local Stability

- The length of ***stiffened*** and ***unstiffened*** elements is discussed in **AISC B4.1(b)** and in **Part 1** of the Manual (**1**-6).



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Chapter 4 – Local Stability

- The length of ***stiffened*** and ***unstiffened*** elements is discussed in **AISC B4.1(b)** and in **Part 1** of the Manual (**1**-6).

1a. 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, b_f.

(b) For legs of angles and flanges of channels and zees, the width, b, is the full leg or flange width.

(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 the full depth of the section.

User Note: Refer to Table B4.1 for the graphic representation of unstiffened element dimensions.

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Chapter 4 – Local Stability

- The length of ***stiffened*** and ***unstiffened*** elements is discussed in **AISC B4.1(b)** and in **Part 1** of the Manual (**1**-6).

1b. 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 sections, h is the clear distance between flanges less the fillet at each flange; h_c 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 h_c 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; h_p 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 plates in built-up sections, the width, b, is the distance between adjacent lines of fasteners or lines of welds.

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Chapter 4 – Local Stability

- The length of ***stiffened*** and ***unstiffened*** elements is discussed in **AISC B4.1(b)** and in **Part 1** of the Manual (**1**-6).

(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, per Section B4.2.

(e) For flanges or webs of box sections and other stiffened elements, the width, b, is the clear distance between the elements providing stiffening.

(f) For perforated cover plates, b is the transverse distance between the nearest line of fasteners, and the net area of the plate is taken at the widest hole.

(g) For round hollow structural sections (HSS), the width shall be taken as the outside diameter, D, and the thickness, t, shall be taken as the design wall thickness, as defined in Section B4.2.

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Chapter 4 – Local Stability

- The length of ***stiffened*** and ***unstiffened*** elements is discussed in **AISC B4.1(b)** and in **Part 1** of the Manual (**1**-6).

1-6 DIMENSIONS AND PROPERTIES

example, an HSS10.000×0.500 is nominally 10 in. in diameter with a ½ in. nominal wall thickness.

Per AISC *Specification* Section B4.2, the wall thickness used in design, t_{des}, is taken as 0.93 times the nominal wall thickness for HSS conforming to ASTM A500/A500M. The rationale for this requirement is explained in the corresponding AISC *Specification* Commentary Section B4.2.

In calculating the *b/t* and *h/t* ratios in Tables 1-11 and 1-12, each outside corner radius is taken as $1.5t_{des}$ for rectangular and square HSS. This is in conformity with AISC *Specification* Section B4.1b(d). In Table 1-11, *b* is the lesser value, and *h* is the greater value of the outside dimensions. When using AISC *Specification* Table B4.1a, Case 6, with Table 1-11, *b/t* should be taken from the *h/t* column in the table. In other tabulated properties, each corner radius is taken as $2t_{des}$. In the tabulated workable flat dimensions of rectangular (and square) HSS, the outside corner radii are taken as $2.25t_{nom}$. The term workable flat refers to a reasonable flat width or depth of material for use in making connections to HSS. The workable flat dimension is provided as a reflection of current industry practice, although the tolerances of ASTM A500/A500M allow a greater maximum corner radius of $3t_{nom}$.

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Chapter 4 – Local Stability

- **Example 4-3**: A **W14 x 53** of **A992** steel (F_y = 50 *ksi* and F_u = 65 *ksi*) has a length of 15 *ft*, and the ends are pinned.

- Investigate the flange for local stability.

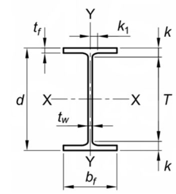
$b_f = 8.06\,in$

$t_f = 0.660\,in$

$$\frac{b_f}{2t_f} = \frac{8.06\,in}{2(0.660\,in)} = 6.11$$

$$\lambda = 0.56\sqrt{\frac{E}{F_y}} = 0.56\sqrt{\frac{29{,}000\,ksi}{50\,ksi}}$$

$$= 13.49 > 6.11 \quad \textbf{OK}$$



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Chapter 4 – Local Stability

- **Example 4-3**: A **W14 x 53** of **A992** steel (F_y = 50 *ksi* and F_u = 65 *ksi*) has a length of 15 *ft*, and the ends are pinned.

- Investigate the web for local stability.

$d = 13.9\,in$

$t_w = 0.370\,in$

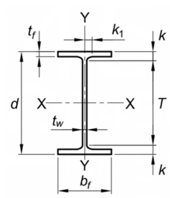
$k_{des} = 1.25\,in$

$$\frac{h}{t_w} = \frac{d - 2k_{des}}{t_w} = \frac{13.9\,in - 2(1.25\,in)}{0.370\,in}$$

$$= 30.81$$

$$\lambda = 1.49\sqrt{\frac{E}{F_y}} = 1.49\sqrt{\frac{29{,}000\,ksi}{50\,ksi}}$$

$$= 35.88 > 30.81 \quad \textbf{OK}$$



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Chapter 4 – Local Stability

- In **Example 4.3**, the width-to-thickness ratios $b_f/2t_f$ and h/t_w were computed.

- This is ***not necessary***, because these ratios are tabulated in the dimensions and properties table.

- In addition, shapes that are ***slender*** for compression are indicated with a footnote (***footnote c***).

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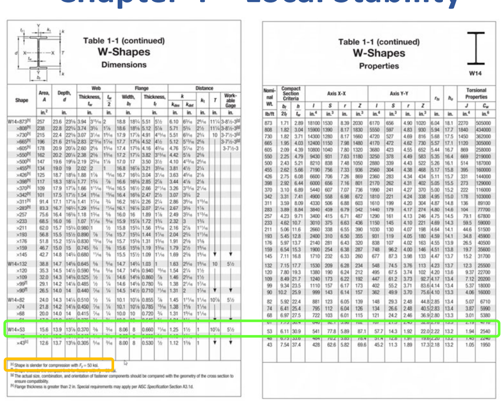
Chapter 4 – Local Stability



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Chapter 4 – Local Stability

Table 1-1 (continued)
W-Shapes
Properties
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]
82	5.92	22.4	881	123	6.05	139	148	29.3	2.48	44.8	2.85	13.4	5.07	6710
74	6.41	25.4	795	112	6.04	126	134	26.6	2.48	40.5	2.83	13.4	3.87	5990
68	6.97	27.5	722	103	6.01	115	121	24.2	2.46	36.9	2.80	13.3	3.01	5380
61	7.75	30.4	640	92.1	5.98	102	107	21.5	2.45	32.8	2.78	13.3	2.19	4710
53	6.11	30.9	541	77.8	5.89	87.1	57.7	14.3	1.92	22.0	2.22	13.2	1.94	2540
48	[illegible]	[illegible]	484	70.2	5.85	78.4	51.4	12.8	1.91	19.6	2.20	13.2	1.45	2240
43	7.54	37.4	428	62.6	5.82	69.6	45.2	11.3	1.89	17.3	2.18	13.2	1.05	1950

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Chapter 4 – Local Stability

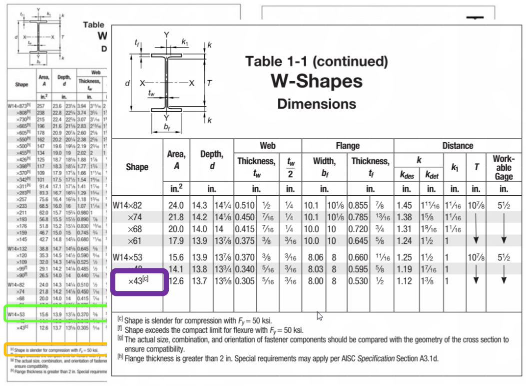

Table 1-1 (continued)
W-Shapes
Dimensions

Shape	Area, A	Depth, d		Web Thickness, t_w		Web $\frac{t_w}{2}$	Flange Width, b_f		Flange Thickness, t_f		Distance k k_{des}	Distance k k_{det}	k_1	T	Workable Gage
	in.²	in.		in.		in.	in.		in.		in.	in.	in.	in.	in.
W14×82	24.0	14.3	14¼	0.510	½	¼	10.1	10⅛	0.855	⅞	1.45	1¹¹⁄₁₆	1¹⁄₁₆	10⅞	5½
×74	21.8	14.2	14⅛	0.450	⁷⁄₁₆	¼	10.1	10⅛	0.785	¹³⁄₁₆	1.38	1⅝	1¹⁄₁₆	↓	↓
×68	20.0	14.0	14	0.415	⁷⁄₁₆	¼	10.0	10	0.720	¾	1.31	1⁹⁄₁₆	1¹⁄₁₆		
×61	17.9	13.9	13⅞	0.375	⅜	³⁄₁₆	10.0	10	0.645	⅝	1.24	1½	1		
W14×53	15.6	13.9	13⅞	0.370	⅜	³⁄₁₆	8.06	8	0.660	¹¹⁄₁₆	1.25	1½	1	10⅞	5½
[illegible]	14.1	13.8	13¾	0.340	⁵⁄₁₆	³⁄₁₆	8.03	8	0.595	⅝	1.19	1⁷⁄₁₆	1	↓	↓
×43[c]	12.6	13.7	13⅝	0.305	⁵⁄₁₆	³⁄₁₆	8.00	8	0.530	½	1.12	1⅜	1		

[c] Shape is slender for compression with $F_y = 50$ ksi.
[f] Shape exceeds the compact limit for flexure with $F_y = 50$ ksi.
[g] The actual size, combination, and orientation of fastener components should be compared with the geometry of the cross section to ensure compatibility.
[h] Flange thickness is greater than 2 in. Special requirements may apply per AISC *Specification* Section A3.1d.

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Chapter 4 – Local Stability

- It is permissible to use a cross-sectional shape that does not satisfy the width-to-thickness ratio requirements, but such a member may ***not be permitted*** to carry as large a load as one that does satisfy the requirements.
- In other words, the ***strength could be reduced*** because of ***local buckling***.
- The procedure for making this investigation is given in **AISC E7**.

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Chapter 4 – Local Stability

- The nominal strength based on the limit state of local buckling is:

$$P_n = F_n A_e \qquad \textbf{AISC Equation E7-1}$$

where $\boldsymbol{A_e}$ is a reduced ***effective*** cross-sectional area

- The reduced area of each slender element is $\boldsymbol{b_e t}$

where $\boldsymbol{t}$ is the thickness of the element, and
$\boldsymbol{b_e}$ is a reduced effective width of the element. (Note: for tees, this is $\boldsymbol{d_e}$; for webs, this is $\boldsymbol{h_e}$.)

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Chapter 4 – Local Stability

- The reduced effective area of the cross-section is the gross area minus the reduction in area.
- The reduced area of each slender element is shown in the Specification User Note in **AISC E7** as:

$$bt - b_e t = (b - b_e)t$$

- So, the ***effective net area*** of the cross-section is:

$$A_e = A_g - \sum (b - b_e)t$$

- The summation is for the case where there is more than one slender element.

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Chapter 4 – Local Stability

- In **Table B4.1a**, the notation ***b/t*** is a generic form of the width-to-thickness ratio.
- The dimension ***b*** can be a ***width*** or a ***depth***.
- The reduced effective width is obtained as follows:

$$\lambda \le \lambda_r \sqrt{\frac{F_y}{F_n}} \quad \Rightarrow \quad b_e = b \qquad \textbf{AISC Equation E7-2}$$

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Chapter 4 – Local Stability

- In **Table B4.1a**, the notation ***b/t*** is a generic form of the width-to-thickness ratio.
- The dimension ***b*** can be a ***width*** or a ***depth***.
- The reduced effective width is obtained as follows:

$$\lambda > \lambda_r \sqrt{\frac{F_y}{F_n}} \quad \Rightarrow \quad b_e = b\left(1 - c_1\sqrt{\frac{F_{el}}{F_n}}\right)\sqrt{\frac{F_{el}}{F_n}}$$

AISC Equation E7-3

where $\boldsymbol{c_1}$ is the effective width imperfection adjustment factor from **AISC Table E7.1.**

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Chapter 4 – Local Stability

- In **Table B4.1a**, the notation ***b/t*** is a generic form of the width-to-thickness ratio.
- The dimension ***b*** can be a ***width*** or a ***depth***.
- The reduced effective width is obtained as follows:

$$\lambda > \lambda_r\sqrt{\frac{F_y}{F_n}} \Rightarrow b_e = b\left(1 - c_1\sqrt{\frac{F_{el}}{F_n}}\right)\sqrt{\frac{F_{el}}{F_n}}$$

AISC Equation E7-3

$$F_{el} = \left(c_2\frac{\lambda_r}{\lambda}\right)^2 F_y$$

Elastic local buckling stress

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Chapter 4 – Local Stability

- In **Table B4.1a**, the notation ***b/t*** is a generic form of the width-to-thickness ratio.
- The dimension ***b*** can be a ***width*** or a ***depth***.
- The reduced effective width is obtained as follows:

$$\lambda > \lambda_r\sqrt{\frac{F_y}{F_n}} \Rightarrow b_e = b\left(1 - c_1\sqrt{\frac{F_{el}}{F_n}}\right)\sqrt{\frac{F_{el}}{F_n}}$$

AISC Equation E7-3

$$c_2 = \frac{1-\sqrt{1-4c_1}}{2c_1}$$

The factor c_2 is also tabulated in **AISC Table E7-1.**

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Chapter 4 – Local Stability

- In **Table B4.1a**, the notation ***b/t*** is a generic form of the width-to-thickness ratio.
- The dimension ***b*** can be a ***width*** or a ***depth***.
- The reduced effective width is obtained as follows:

TABLE E7.1
Effective Width Imperfection Adjustment Factors, c_1 and c_2

Case	Slender Element	c_1	c_2
(a)	Stiffened elements except walls of square and rectangular HSS	0.18	1.31
(b)	Walls of square and rectangular HSS	0.20	1.38
(c)	All other elements	0.22	1.49

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Chapter 4 – Local Stability

- **Example 4-4**: A **HSS 6 x 3 x ⅛** of **A500 Grade C** steel (***F_y*** = 50 *ksi* and ***F_u*** = 62 *ksi*) has a length of 10 *ft*, and the ends are pinned. Compute the flexural buckling strength.
- From **Table 1-11: Rectangular HHS**

$r_y = 1.21 in$, $h/t = 48.7$, $b/t = 22.9$, $A_g = 2.00 in^2$

$$L_c = (10\,ft)(12\,in/ft) = 120\ in$$

$$\frac{L_c}{r} = \frac{120\,in}{1.27\,in} = 94.49 < 200 \quad \textbf{OK}$$

$$4.71\sqrt{\frac{E}{F_y}} = 4.71\sqrt{\frac{29{,}000\,ksi}{50\,ksi}} = 113.4$$

$$\frac{L_c}{r} \le 4.71\sqrt{\frac{E}{F_y}} \Rightarrow F_n = \left(0.658^{F_y/F_e}\right)F_y$$

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Chapter 4 – Local Stability

- **Example 4-4**: A **HSS 6 x 3 x ⅛** of **A500 Grade C** steel (***F_y*** = 50 *ksi* and ***F_u*** = 62 *ksi*) has a length of 10 *ft*, and the ends are pinned. Compute the flexural buckling strength.

$$F_e = \frac{\pi^2 E}{(L_c/r)^2} = \frac{\pi^2(29{,}000\,ksi)}{(94.49)^2} = 32.06\,ksi$$

$$F_n = \left(0.658^{50ksi/32.06ksi}\right)(50\,ksi) = 26.03\,ksi$$

$$P_n = F_n A_g = 26.03\,ksi\left(2.00\,in^2\right) = \boxed{52.06\,k}$$

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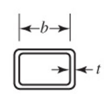
Chapter 4 – Local Stability

- **Example 4-4**: A **HSS 6 x 3 x ⅛** of **A500 Grade C** steel (***F_y*** = 50 *ksi* and ***F_u*** = 62 *ksi*) has a length of 10 *ft*, and the ends are pinned. Compute the flexural buckling strength.
- Check width-to-thickness ratios: $h/t = 48.7$ $b/t = 22.9$
- From **AISC Table B4.1a**, Case 6, the upper limit for nonslender elements is



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

$$\lambda_r = 1.40\sqrt{\frac{E}{F_y}} = 1.40\sqrt{\frac{29{,}000\,ksi}{50\,ksi}} = 33.72$$

$$h/t = 48.7 > 33.72$$

- Since $\lambda > \lambda_r$ the larger dimension element is ***slender,*** and the ***local buckling strength*** must be computed.

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Chapter 4 – Local Stability

- **Example 4-4**: A **HSS 6 x 3 x ⅛** of **A500 Grade C** steel (***F$_y$*** = 50 *ksi* and ***F$_u$*** = 62 *ksi*) has a length of 10 *ft*, and the ends are pinned. Compute the flexural buckling strength.
- Although the limiting width-to-thickness ratio is labeled ***b/t*** in the table, it applies to ***h/t*** as well.
- Determine ***h$_e$*** for the 6-inch side:

$$\lambda \le \lambda_r \sqrt{F_y/F_n} \quad \Rightarrow \quad h_e = h \qquad \textbf{AISC Equation E7-2}$$

$$\lambda_r \sqrt{F_y/F_n} = 33.72\sqrt{50ksi/26.03ksi} = 46.73$$

$$\lambda = h/t = 48.7 > 46.73 \qquad \textbf{AISC Equation E7-3}$$

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Chapter 4 – Local Stability

- **Example 4-4**: A **HSS 6 x 3 x ⅛** of **A500 Grade C** steel (***F$_y$*** = 50 *ksi* and ***F$_u$*** = 62 *ksi*) has a length of 10 *ft*, and the ends are pinned. Compute the flexural buckling strength.
- Determine ***h$_e$*** using **AISC Equation E7-3**

$$h_e = h\left(1 - c_1\sqrt{F_{el}/F_n}\right)\sqrt{F_{el}/F_n}$$

- From **Table E7.1, Case b**: $c_1 = 0.20$ $c_2 = 1.38$

$$F_{el} = \left(c_2 \frac{\lambda_r}{\lambda}\right)^2 F_y = \left(1.38\left(\frac{33.72}{48.7}\right)\right)^2 50ksi = 45.64ksi$$

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Chapter 4 – Local Stability

- **Example 4-4**: A **HSS 6 x 3 x ⅛** of **A500 Grade C** steel (***F$_y$*** = 50 *ksi* and ***F$_u$*** = 62 *ksi*) has a length of 10 *ft*, and the ends are pinned. Compute the flexural buckling strength.
- From **AISC B4.1(b)** and the discussion in **Part 1** of the **Manual** (**1**-6), the unreduced length of the 6-inch side between the corner radii is:

$$h = 6in - 3t = 6in - 3(0.116in) = 5.652in \qquad \boxed{t = 0.116in}$$

$$h_e = h\left(1 - c_1\sqrt{F_{el}/F_n}\right)\sqrt{F_{el}/F_n}$$

$$= 5.652in\left(1 - 0.20\sqrt{45.64ksi/26.03ksi}\right)\sqrt{45.64ksi/26.03ksi}$$

$$= 5.502in$$

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Chapter 4 – Local Stability

- **Example 4-4**: A **HSS 6 x 3 x ⅛** of **A500 Grade C** steel (***F$_y$*** = 50 *ksi* and ***F$_u$*** = 62 *ksi*) has a length of 10 *ft*, and the ends are pinned. Compute the flexural buckling strength.
- The effective cross-sectional area is: $\boxed{A_g = 2.00in^2}$

$$A_e = A_g - \sum(h - h_e)t = 2.00in^2 - 2(5.652in - 5.502in)(0.116in)$$

Two 6 in. sides

$$= 1.965in^2$$

$$P_n = F_n A_e = (26.03ksi)1.965in^2 = \boxed{51.15k}$$

- Since this is less than the flexural buckling strength of 52.06 *k*, ***local buckling*** controls.

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Chapter 4 – Local Stability

- **Example 4-4**: A **HSS 6 x 3 x ⅛** of **A500 Grade C** steel (***F$_y$*** = 50 *ksi* and ***F$_u$*** = 62 *ksi*) has a length of 10 *ft*, and the ends are pinned. Compute the flexural buckling strength.
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$$A_e = A_g - \sum(h - h_e)t = 2.00in^2 - 2(5.652in - 5.502in)(0.116in)$$

Two 6 in. sides

$$= 1.965in^2$$

$$P_n = F_n A_e = (26.03ksi)1.965in^2 = \boxed{51.15k}$$

- If a cross-section has ***slender*** element(s), the local flexural buckling strength is calculated, and it will be the ***final answer***.

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Chapter 4 – Local Stability

- **Example 4-4**: A **HSS 6 x 3 x ⅛** of **A500 Grade C** steel (***F$_y$*** = 50 *ksi* and ***F$_u$*** = 62 *ksi*) has a length of 10 *ft*, and the ends are pinned. Compute the flexural buckling strength.
- ***LRFD*** design strength is: $\phi_c P_n = 0.90(51.15k) = \boxed{46.04k}$
- ***ASD*** allowable strength is: $P_n/\Omega = 51.15/1.67 = \boxed{30.63k}$

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Let’s work on some problems

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Any questions?

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