# Ex. 3.2 - Column Strength

- Calculate the design strength of W14 x 74 with length of 6 m and pinned ends. A36 steel is used.

  - **Step I.** Calculate the effective length and slenderness ratio for the problem

    $K_x = K_y = 1.0$

    $L_x = L_y = 6$ m

    Major axis slenderness ratio = $K_x L_x / r_x$ = 6000/153.4 = 39.1

    Minor axis slenderness ratio = $K_y L_y / r_y$ = 6000/63 = 95.2

  - **Step II.** Calculate the buckling strength for governing slenderness ratio

    The governing slenderness ratio is the larger of ($K_x L_x / r_x$, $K_y L_y / r_y$)

# Ex. 3.2 - Column Strength

- $K_yL_y/r_y$ is larger and the governing slenderness ratio;

$$K_yL_y/r_y < 4.71\sqrt{\frac{E}{F_y}} = 4.71\sqrt{\frac{200000}{248}} = 133.7$$

- $$F_e = \frac{\pi^2 E}{\left(K_yL_y/r_y\right)^2} = \frac{3.1416^2 \times 200000}{95.2^2} = 217.8 \quad \text{MPa}$$

- Therefore, $F_{cr} = \left(0.658^{F_y/F_e}\right)F_y = 154$ MPa
- Design column strength = $\phi_c P_n$ = 0.9 ($A_g$ $F_{cr}$) = 0.9 (14060x154)/1000= 1948.7 kN.
- Design strength of column = 1948.7 kN.

# 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.

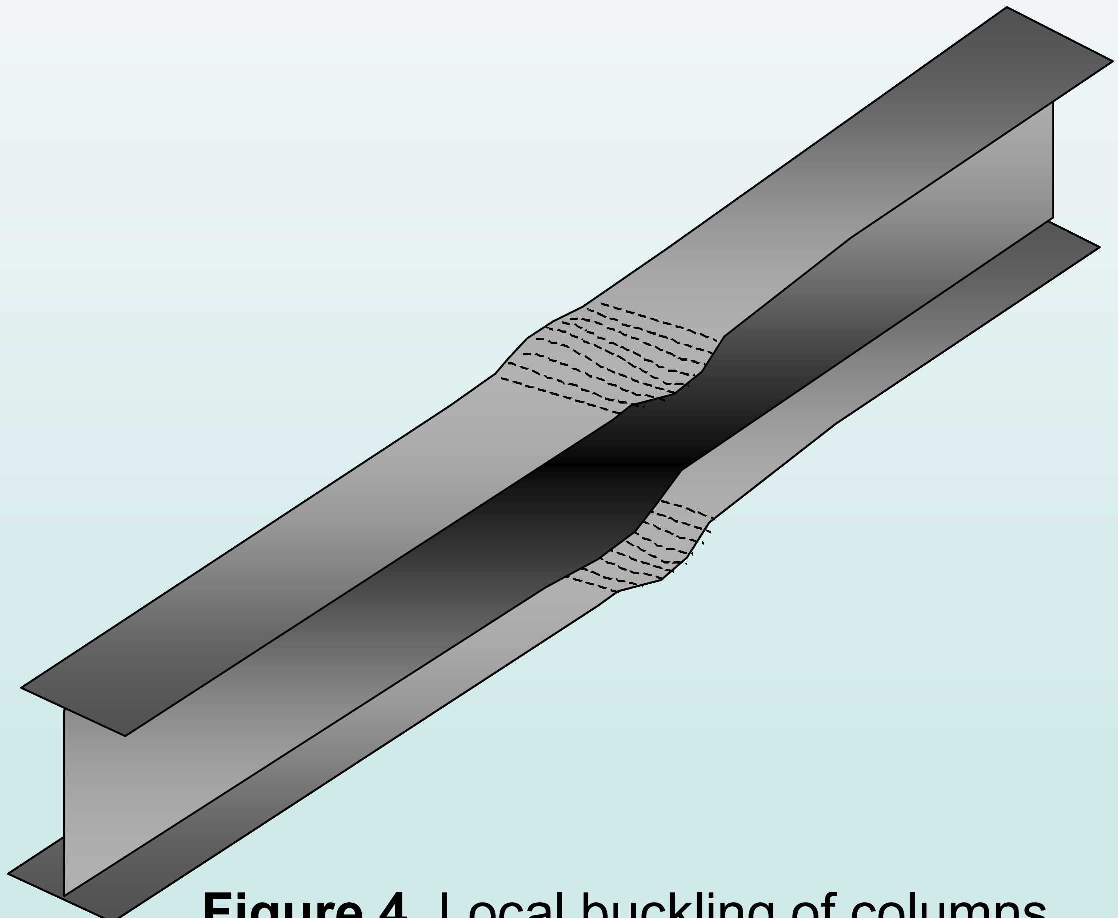
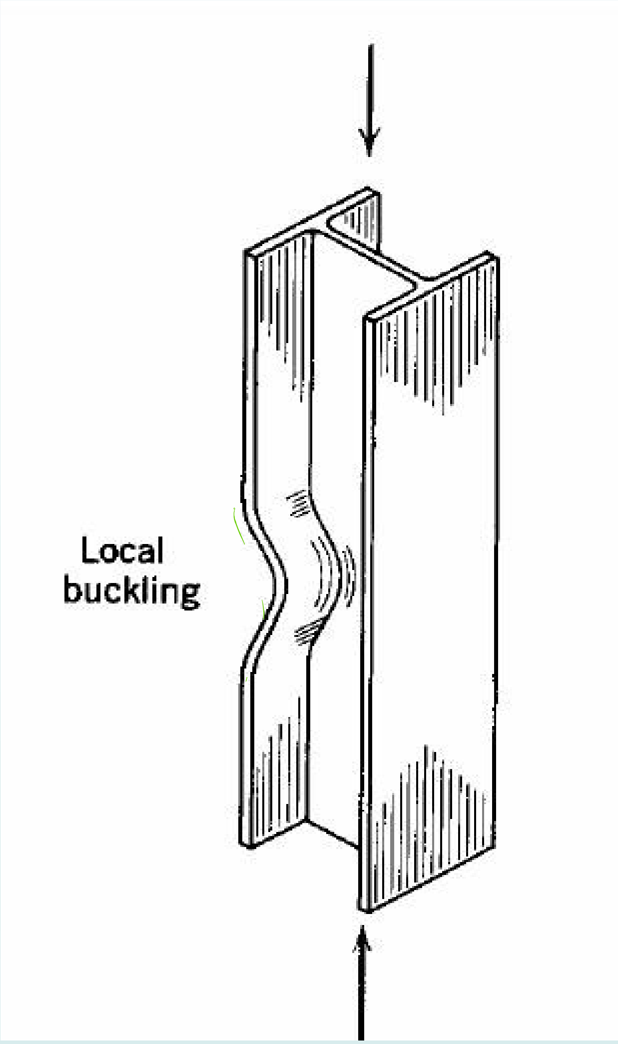


**Figure 4.** Local buckling of columns

# Local Buckling Limit State

- Local buckling is another limitation that represents the instability of the cross section itself.

- If local buckling occurs, the full strength of the cross section can not be developed.



# Local Buckling Limit State

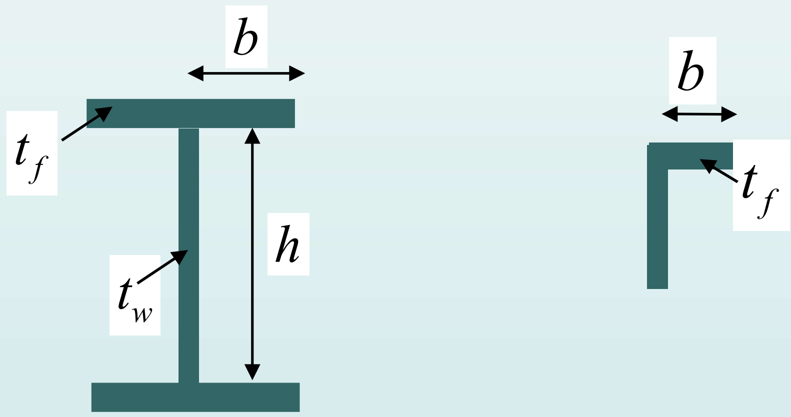
- If *local buckling* of the individual plate elements occurs, then the column may not be able to develop its buckling strength.
- Therefore, the local buckling limit state must be prevented from controlling the column strength.
- Local buckling depends on the slenderness (width-to-thickness *b/t* ratio) of the plate element and the yield stress ($F_y$) 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 provides the slenderness (b/t) limits that the individual plate elements must satisfy so that *local buckling* does not control.

# Local Buckling Limit State

- Local buckling can be prevented by limiting the width to thickness ratio known as "λ" to an upper limit $\lambda_r$

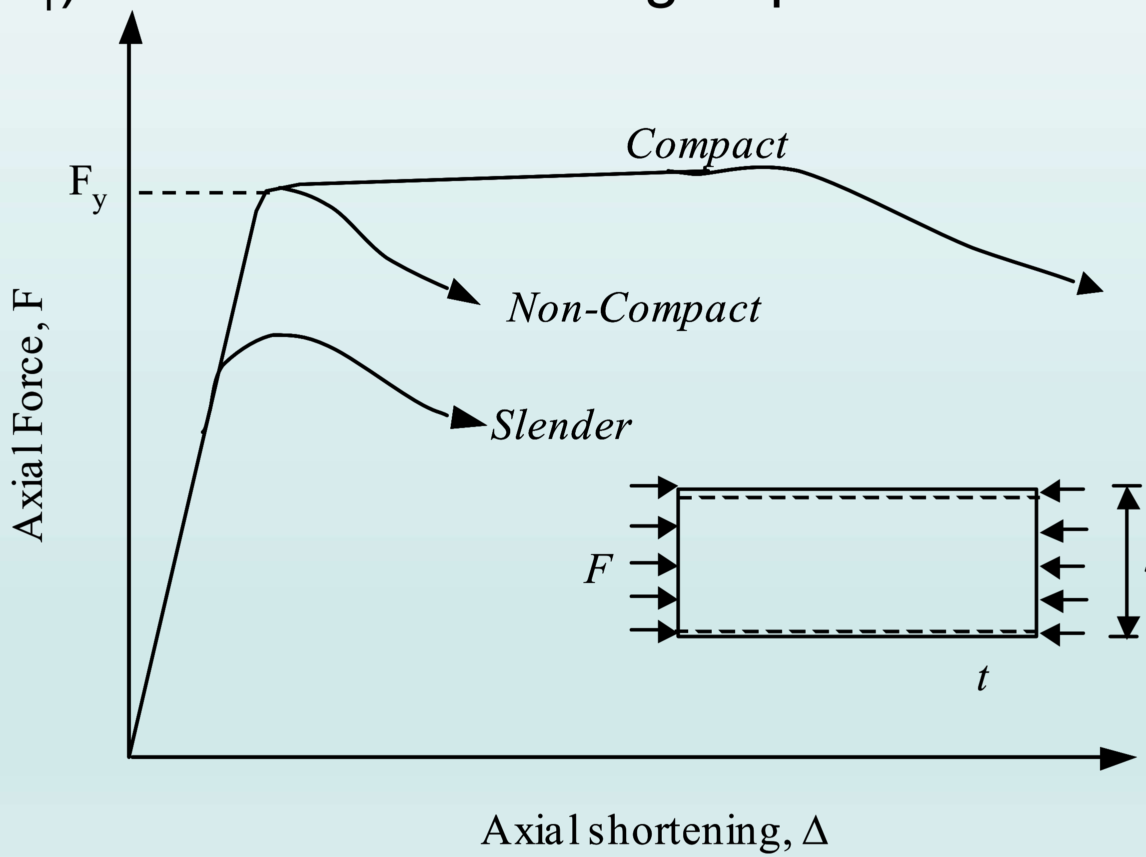
$$\lambda = \frac{b}{t_f} \leq 0.56\sqrt{\frac{E}{F_y}}$$

$$\lambda = \frac{h}{t_w} \leq 1.49\sqrt{\frac{E}{F_y}}$$



# Local Buckling Limit State

- The AISC specification provides two slenderness limits ($\lambda_p$ and $\lambda_r$) for the local buckling of plate elements.



# Local Buckling Limit State

- If the slenderness ratio (b/t) of the plate element is greater than $\lambda_r$ then it *is slender*. It will locally buckle in the elastic range *before* reaching $F_y$
- If the slenderness ratio (b/t) of the plate element is less than $\lambda_r$ but greater than $\lambda_p$, then it is *non-compact*. It will locally buckle *immediately* after reaching $F_y$
- If the slenderness ratio (b/t) of the plate element is less than $\lambda_p$, then the element is *compact*. It will locally buckle *much after* reaching $F_y$

- If all the plate elements of a cross-section are compact, then the section is *compact*.
  - If any one plate element is non-compact, then the cross-section is non-compact
  - If any one plate element is slender, then the cross-section is slender.

# Local Buckling Limit State

- Cross section can be classified as "*compact*", "*non compact*" or "*slender*" sections based on their width to thickness ratios
- If the cross-section does not satisfy local buckling requirements its critical buckling stress $F_{cr}$ shall be reduced
- If $\lambda \geq \lambda_r$ then the section is slender, a reduction factor for capacity shall be computed from

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- It is not recommended to use slender sections for columns.

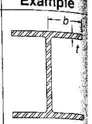
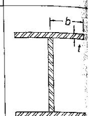
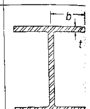
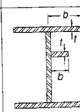
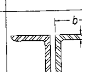
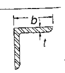
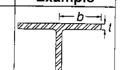
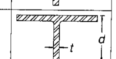
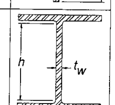
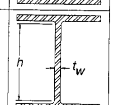
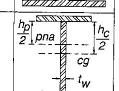
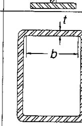
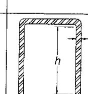
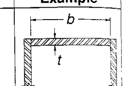
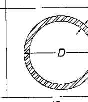
# Local Buckling Limit State

- The slenderness limits $\lambda_p$ and $\lambda_r$ for various plate elements with different boundary conditions are given in the AISC Manual.
- Note that the slenderness limits ($\lambda_p$ and $\lambda_r$) and the definition of plate slenderness (b/t) ratio depend upon the boundary conditions for the plate.
  - If the plate is supported along *two edges* parallel to the direction of compression force, then it is a *stiffened* element. For example, the webs of W shapes
  - If the plate is supported along only *one edge* parallel to the direction of the compression force, then it is an *unstiffened* element. Ex., the flanges of W shapes.
- The local buckling limit state can be prevented from controlling the column strength by using sections that are compact and non-compact.
- Avoid slender sections

# Local Buckling Limit State



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

| | Case | Description of Element | Width Thick-ness Ratio | Limiting Width-Thickness Ratios: $\lambda_p$ (compact) | Limiting Width-Thickness Ratios: $\lambda_r$ (noncompact) | Example |
|---|---|---|---|---|---|---|
| Unstiffened Elements | 1 | Flexure in flanges of rolled I-shaped sections and channels | $b/t$ | $0.38\sqrt{E/F_y}$ | $1.0\sqrt{E/F_y}$ |  |
| Unstiffened Elements | 2 | Flexure in flanges of doubly and singly symmetric I-shaped built-up sections | $b/t$ | $0.38\sqrt{E/F_y}$ | $0.95\sqrt{k_cE/F_L}$ [a],[b] |  |
| Unstiffened Elements | 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 | $0.56\sqrt{E/F_y}$ |  |
| Unstiffened Elements | 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 | $0.64\sqrt{k_cE/F_y}$ [a] |  |
| Unstiffened Elements | 5 | Uniform compression in legs of single angles, legs of double angles with separators, and all other unstiffened elements | $b/t$ | NA | $0.45\sqrt{E/F_y}$ |  |
| Unstiffened Elements | 6 | Flexure in legs of single angles | $b/t$ | $0.54\sqrt{E/F_y}$ | $0.91\sqrt{E/F_y}$ |  |
| Unstiffened Elements | 7 | Flexure in flanges of tees | $b/t$ | $0.38\sqrt{E/F_y}$ | $1.0\sqrt{E/F_y}$ |  |
| Unstiffened Elements | 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 | $h/t_w$ | $3.76\sqrt{E/F_y}$ | $5.70\sqrt{E/F_y}$ |  |
| Stiffened Elements | 10 | Uniform compression in webs of doubly symmetric I-shaped sections | $h/t_w$ | NA | $1.49\sqrt{E/F_y}$ |  |
| Stiffened Elements | 11 | Flexure in webs of singly-symmetric I-shaped sections | $h_c/t_w$ | $\dfrac{\frac{h_c}{h_p}\sqrt{\frac{E}{F_y}}}{\left(0.54\frac{M_p}{M_y}-0.09\right)^2} \le \lambda_r$ | $5.70\sqrt{E/F_y}$ |   |
| Stiffened Elements | 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 | $b/t$ | $1.12\sqrt{E/F_y}$ | $1.40\sqrt{E/F_y}$ |  |
| Stiffened Elements | 13 | Flexure in webs of rectangular HSS | $h/t$ | $2.42\sqrt{E/F_y}$ | $5.70\sqrt{E/F_y}$ |  |
| Stiffened Elements | 14 | Uniform compression in all other stiffened elements | $b/t$ | NA | $1.49\sqrt{E/F_y}$ |  |
| Stiffened Elements | 15 | Circular hollow sections<br>In uniform compression<br>In flexure | <br>$D/t$<br>$D/t$ | <br>NA<br>$0.07E/F_y$ | <br>$0.11E/F_y$<br>$0.31E/F_y$ |   |

[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)



**B5. FABRICATION, ERECTION AND QUALITY CONTROL**

Shop drawings, fabrication, shop painting, erection, and *quality control* shall meet the requirements stipulated in Chapter M, Fabrication, Erection, and Quality Control.

**B6. EVALUATION OF EXISTING STRUCTURES**

Provisions for the evaluation of existing structures are presented in Appendix 5, Evaluation of Existing Structures.