### Lateral-Torsional Buckling (LTB) – Uniform BM **Lateral-Torsional Buckling (LTB) –<br>Uniform BM**<br>• As soon as any portion of the cross-section reaches the yield stress  $F_y$ , the elastic LTB equation cannot be used. **ateral-Torsional Buckling<br>
niform BM**<br>
As soon as any portion of the cross-sect<br>
yield stress F<sub>y</sub>, the elastic LTB equation car<br>
• L<sub>r</sub> is the unbraced length that corresponds to a L **prsional Buckling (LTB)** –<br>**M**<br>iny portion of the cross-section reaches the<br>, the elastic LTB equation cannot be used.<br>aced length that corresponds to a LTB moment **ral-Torsional Buckling (LTB) –<br>
orm BM**<br>
oon as any portion of the cross-section reaches the<br>
stress  $F_y$ , the elastic LTB equation cannot be used.<br>
is the unbraced length that corresponds to a LTB moment<br>
=  $S_x$  (0.7F<sub>y</sub> **eral-Torsional Buckli**<br> **form BM**<br>
soon as any portion of the cross<br>
Id stress  $F_y$ , the elastic LTB equatio<br>
L<sub>r</sub> is the unbraced length that corresponds<br>  $M_r = S_x (0.7F_y)$ .<br>  $M_r$  will cause yielding of the cross-section

- ortion reaches the<br>
the mot be used.<br>
LTB moment<br>
o residual stresses.<br>
, then the elastic
- $L_r$  is the unbraced length that corresponds to a LTB moment  $(0.7F_y)$ . **Example 11 DESIDE IS DUCKLIFT CAUSE 11 DEALLY AS SOON AS SOON AS ANY portion of the cross-section reaches the vield stress F<sub>y</sub>, the elastic LTB equation cannot be used.<br>• L<sub>r</sub> is the unbraced length that corresponds to** • As soon as any portion of the cross-section reaches the yield stress  $F_y$ , the elastic LTB equation cannot be used.<br>• L<sub>r</sub> is the unbraced length that corresponds to a LTB moment  $M_r = S_x (0.7F_y)$ .<br>•  $M_r$  will cause yieldi As soon as any portion of the cross-section reachied stress  $F_y$ , the elastic LTB equation cannot be<br>  $L_r$  is the unbraced length that corresponds to a LTB mom<br>  $M_r = S_x (0.7F_y)$ .<br>
In M<sub>r</sub> will cause yielding of the cross-se
	-
- 
- **EVALUATE:** The elastic LTD equation cannot be used.<br>
 L<sub>r</sub> is the unbraced length that corresponds to a LTB moment<br>  $M_r = S_x (0.7F_y)$ .<br>
 M<sub>r</sub> will cause yielding of the cross-section due to residual stresses.<br>
 When the sponds to a LTB moment<br>section due to residual stresses.<br>ses than  $L_r$ , then the elastic<br>) is less than  $L_r$  but more<br>the LTB  $M_n$  is given by the • L<sub>r</sub> is the unbraced length that corresponds to a LTB moment<br>  $M_r = S_x (0.7F_y)$ .<br>
• M<sub>r</sub> will cause yielding of the cross-section due to residual stre:<br>
When the unbraced length is less than L<sub>r</sub>, then the e<br>
LTB Eq. cann at corresponds to a LTB moment<br>
Exercises cross-section due to residual stresses.<br>
th is less than L<sub>r</sub>, then the elastic<br>
gth (L<sub>b</sub>) is less than L<sub>r</sub> but more<br>
then the LTB M<sub>n</sub> is given by the<br>
46 B moment<br>esidual stresses.<br>hen the elastic<br>in L<sub>r</sub> but more<br>is given by the<br><sup>46</sup>  $M_r = S_x (0.7F_y)$ .<br>
•  $M_r$  will cause yielding of the cross-sect<br>
When the unbraced length is less<br>
LTB Eq. cannot be used.<br>
When the unbraced length (L<sub>b</sub>) is<br>
than the plastic length L<sub>p</sub>, then the<br>
Eq. below:

#### Lateral-Torsional Buckling – Uniform BM

• If 
$$
L_p \le L_b \le L_r
$$
, then  $M_n = \left[ M_p - (M_p - M_r) \left( \frac{L_b - L_p}{L_r - L_p} \right) \right]$ 

• This is linear interpolation between  $(L_p, M_p)$  and  $(L_r, M_r)$ )

• See Fig. 10 again.

$$
L_r = 1.95r_{ts} \frac{E}{0.7F_y} \sqrt{\frac{Jc}{S_x h_0}} \sqrt{1 + \sqrt{1 + 6.76 \left(\frac{0.7F_y}{E} \frac{S_x h_0}{Jc}\right)^2}}
$$

$$
r_{ts}^2 = \frac{\sqrt{I_y C_w}}{S_x}
$$

- For a doubly symmetric I-shape:  $c = 1$
- $h_0$  = distance between the flange centroids (mm)

## Moment Capacity of Beams Subjected to Non-uniform BM **Moment Capacity of Beams<br>
Subjected to Non-uniform BM**<br>
• As mentioned previously, the case with uniform bending<br>
moment is worst for lateral torsional buckling. **Ioment Capacity of Beams<br>
ubjected to Non-uniform BM**<br>
As mentioned previously, the case with uniform bending<br>
moment is worst for lateral torsional buckling.<br>
For cases with non-uniform bending moment, the LTB

- 
- **Moment Capacity of Beams<br>
Subjected to Non-uniform BM**<br>
 As mentioned previously, the case with uniform bending<br>
moment is worst for lateral torsional buckling.<br>
 For cases with non-uniform bending moment, the LTB<br>
mome **oment Capacity of Beams<br>ubjected to Non-uniform BM**<br>As mentioned previously, the case with uniform bending<br>moment is worst for lateral torsional buckling.<br>For cases with non-uniform bending moment, the LTB<br>moment. moment. **Subjected to Non-uniform BM**<br>
• As mentioned previously, the case with uniform bending<br>
moment is worst for lateral torsional buckling.<br>
• For cases with non-uniform bending moment, the LTB<br>
moment is greater than that f As mentioned previously, the case with uniform bending<br>noment is worst for lateral torsional buckling.<br>For cases with non-uniform bending moment, the LTB<br>noment is greater than that for the case with uniform<br>noment.<br>The A entioned previously, the case with uniform bending<br>ent is worst for lateral torsional buckling.<br>cases with non-uniform bending moment, the LTB<br>ent is greater than that for the case with uniform<br>ent.<br>ISC specification says
- -
	- $\bullet$   $\mathsf{C}_{\mathsf{b}}$  x lateral torsional buckling moment for uniform moment case.

## Moment Capacity of Beams Subjected to Non-uniform BM **is a meant Capacity of Beams<br>
Signeted to Non-uniform BM**<br>
is always greater than 1.0 for non-uniform bending<br> **Parage and the 1.0 for uniform bending moment. ent Capacity of Beams<br>
ected to Non-uniform BM**<br>
always greater than 1.0 for non-uniform bending<br>
ent.<br>
is equal to 1.0 for uniform bending moment.<br>
metimes, if you cannot calculate or figure out C<sub>b</sub>, then it can be<br>
ise

- $C<sub>b</sub>$  is always greater than 1.0 for non-uniform bending moment.
	- $C<sub>b</sub>$  is equal to 1.0 for uniform bending moment.
- **bjected to Non-uniform BM**<br>  $\mathbf{c}_b$  is always greater than 1.0 for non-uniform bending<br>  $\mathbf{c}_c$  is equal to 1.0 for uniform bending moment.<br>
  $\mathbf{c}_b$  is equal to 1.0 for uniform bending moment.<br>
 Sometimes, if yo orm bending<br>, then it can be<br>ngly symmetric **conservatively assumed as 1.0. for Deams**<br> **is always greater than 1.0 for non-uniform bending**<br> **conservatively assumed as 1.0. for doubly and singly symmetric<br>
Sometimes, if you cannot calculate or figure out**  $C_b$ **, the** sections s always greater than 1.0 for non-uniform bending<br>nent.<br>
b is equal to 1.0 for uniform bending moment.<br>
ometimes, if you cannot calculate or figure out  $C_b$ , then it can be<br>
onservatively assumed as 1.0. for doubly and si at equal to 1.0 for uniform bending moment.<br>
equal to 1.0 for uniform bending moment.<br>
etimes, if you cannot calculate or figure out  $C_b$ , then it can be<br>
ervatively assumed as 1.0. for doubly and singly symmetric<br>
ons<br> **Example 1.0** for uniform bending moment.<br>
equal to 1.0 for uniform bending moment.<br>
etimes, if you cannot calculate or figure out  $C_b$ , then it can be<br>
ervatively assumed as 1.0. for doubly and singly symmetric<br>
ons<br>  $=\frac$ equal to 1.0 for uniform bending moment.<br>
etimes, if you cannot calculate or figure out  $C_b$ , then it can be<br>
ervatively assumed as 1.0. for doubly and singly symmetric<br>
ons<br>  $=\frac{12.5 M_{\text{max}}}{2.5 M_{\text{max}} + 3 M_A + 4 M_B + 3 M_c} < 3.$ Sometimes, it you cannot calculate or figure out  $C_b$ , then it conservatively assumed as 1.0. for doubly and singly symmetries<br>sections<br> $C_b = \frac{12.5 M_{\text{max}}}{2.5 M_{\text{max}} + 3 M_A + 4 M_B + 3 M_c} < 3.0$ <br> $M_{\text{max}}$ - magnitude of maximum b

$$
C_b = \frac{12.5 M_{\text{max}}}{2.5 M_{\text{max}} + 3 M_A + 4 M_B + 3 M_c} < 3.0
$$

- $M_A$  magnitude of bending moment at quarter point of  $L_b$
- $M_B$  magnitude of bending moment at half point of  $L_b$

 $M_c$  - magnitude of bending moment at three-quarter point of  $L_b$ 

#### Flexural Strength of Compact Sections



Moments determined between bracing points

# Other quation for Cb

$$
C_b = 1.75 + 1.05 \left(\frac{M_1}{M_2}\right) + 0.3 \left(\frac{M_1}{M_2}\right)^2 \tag{C-F1-1}
$$

where

 $M_1$  = smaller moment at end of unbraced length, kip-in. (N-mm)  $M_2$  = larger moment at end of unbraced length, kip-in. (N-mm)  $(M_1/M_2)$  is positive when moments cause reverse curvature and negative for single curvature



#### Moment Capacity of Beams Subjected to Non-uniform Bending Moments

- The moment capacity  $M_n$  for the case of non-uniform bending moment
	- $M_n = C_b \times \{M_n \text{ for the case of uniform bending moment} \} \leq M_p$
	- Important to note that the increased moment capacity for the nonuniform moment case cannot possibly be more than  $M<sub>p</sub>$ . .
	- Therefore, if the calculated values is greater than  $M_p$ , then you have to reduce it to  $M_p$

#### Moment Capacity of Beams Subjected to Non-uniform BM



54 Figure 11. Moment capacity versus Lb for non-uniform moment case  $C_b = 1.0$  means uniform BM

### Structural Design of Beams

- Steps for adequate design of beams:
	- 1) Compute the factored loads, factored moment and shear
	- 2) Determine unsupported length  $\mathsf{L}_\mathsf{b}$  and  $\mathsf{C}_\mathsf{b}$
	- 3) Select a WF shape and choose  $Z_{x}$  assuming it is a compact section with full lateral support Equivalent independent entropy to the set of the set of the set of with full lateral support<br>
	with full lateral support<br>  $Z_x = \frac{M_u}{\phi_b F_y}$   $M_u \le \phi_b M_n = 0.9 Z F_y$ <br>
	4) Check the section dimension for compactness and determine

$$
Z_x = \frac{M_u}{\phi_b \ F_y}
$$

 $M_u \leq \phi_b M_n = 0.9 Z F_y$  $M_{n} = M_{p} = ZF_{y}$ 

4) Check the section dimension for compactness and determine  $\phi_h M_n$ 

$$
\boxed{M_{u} \leq \phi_{b} M_{n}}
$$

**Ex. 4.4 – Beam Design<br>• Use Grade 50 steel to design the beam shown below. The unfactored uniformly distributed live load is equal to 4 Ex. 4.4 – Beam Design<br>• Use Grade 50 steel to design the beam shown below. The unfactored uniformly distributed live load is equal to 40<br>kN/m. There is no dead load. Lateral support is provided at x. 4.4 – Beam Design**<br>Use Grade 50 steel to design the beam shown below. The<br>unfactored uniformly distributed live load is equal to 40<br>kN/m. There is no dead load. Lateral support is provided at<br>the end reactions. Select **x. 4.4 – Beam Design Manuson State Convention Section**<br>Use Grade 50 steel to design the beam shown below. The<br>unfactored uniformly distributed live load is equal to 40<br>kN/m. There is no dead load. Lateral support is prov **x. 4.4 – Beam Design**<br>Use Grade 50 steel to design the beam shown below. The<br>unfactored uniformly distributed live load is equal to 40<br>kN/m. There is no dead load. Lateral support is provided at<br>the end reactions. Select



Ex. 4.4 – Beam Design<br>
\* Step I. Calculate the factored loads assuming a reasonable self-<br>
weight. • Step I. Calculate the factored loads assuming a reasonable selfweight.

Assume self-weight =  $w_{sw}$  = 1.46 kN/m. Dead load =  $w_D = 0 + 1.46 = 1.46$  kN/m. Live load =  $w_1$  = 40 kN/m. Ultimate load =  $w_u$  = 1.2  $w_D$  + 1.6  $w_L$  = 65.8 kN/m. Factored ultimate moment =  $M_u = w_u L^2/8 = 462.3$  kN-m. **4.4 – Beam Design**<br>
Step I. Calculate the factored loads assuming a reasonable self-<br>
weight.<br>
Assume self-weight =  $w_{sw}$  = 1.46 kN/m.<br>
Dead load =  $w_D$  = 0 + 1.46 = 1.46 kN/m.<br>
Live load =  $w_L$  = 40 kN/m.<br>
Ultimate loa red loads assuming a reasonable self-<br>= 1.46 kN/m.<br> $b = 1.46$  kN/m.<br> $V_D + 1.6$  w<sub>L</sub> = 65.8 kN/m.<br> $t = M_u = w_u L^2/8 = 462.3$  kN-m.<br>Yes  $C_b = 1.0$ <br>No Go to Step II<br>oorted length L<sub>b</sub> and C<sub>b</sub>

• Step II. Determine unsupported length  $L_b$  and  $C_b$ There is only one unsupported span with  $L_b = 7.5$  m  $C<sub>b</sub>$  = 1.14 for the parabolic bending moment diagram, See values of  $C_{\text{b}}$  shown in Table 3-1.

## Ex. 4.4 – Beam Design<br>
• Step III. Select a wide-flange shape<br>
• Compute  $Z = 462.3*10^6/(0.9*344) = 1493x10^6$  mm<sup>3</sup> **Ex. 4.4 – Beam Design<br>• Step III.** Select a wide-flange shape<br>• Compute  $Z_x = 462.3*10^6/(0.9*344) = 1493x10^6$  mm<sup>3</sup>.<br>• Select W16 x 50 steel section **4.4 – Beam Design<br>
tep III.** Select a wide-flange shape<br>
• Compute  $Z_x = 462.3*10^6/(0.9*344) = 1493x10^6$  mm<sup>3</sup>.<br>
• Select W16 x 50 steel section<br>
•  $Z_y = 1508x10^3$  mm<sup>3</sup> s<sub>5</sub> = 1327x10<sup>3</sup> mm<sup>3</sup> r<sub>y</sub> = 40 **m Design**<br>mge shape<br>/(0.9\*344) = 1493x10<sup>6</sup> mm<sup>3</sup>.<br>ection<br>S<sub>v</sub> = 1327x10<sup>3</sup> mm<sup>3</sup> r<sub>v</sub> = 40.4 mm **4.4 – Beam Design<br>
tep III.** Select a wide-flange shape<br>
• Compute  $Z_x = 462.3*10^6/(0.9*344) = 1493x10^6$  mm<sup>3</sup>.<br>
• Select W16 x 50 steel section<br>
•  $Z_x = 1508x10^3$  mm<sup>3</sup> s<sub>x</sub> = 1327x10<sup>3</sup> mm<sup>3</sup> r<sub>y</sub> = 40.4 mm<br>
•  $C_w = 610x$ **Ex. 4.4 – Beam Design<br>
• Step III.** Select a wide-flange shape<br>
• Compute  $Z_x = 462.3*10^6/(0.9*344) = 1493x10^6$  mm<sup>3</sup>.<br>
• Select W16 x 50 steel section<br>
•  $Z_x = 1508x10^3$  mm<sup>3</sup><br>
•  $S_x = 1327x10^3$  mm<sup>3</sup><br>
•  $T_y = 40.4$  mm

- - .
	-

**4.4 – Beam Design**<br> **tep III.** Select a wide-flange shape<br>
• Compute  $Z_x = 462.3*10^6/(0.9*344) = 1493x10^6$  mm<sup>3</sup>.<br>
• Select W16 x 50 steel section<br>
•  $Z_x = 1508x10^3$  mm<sup>3</sup>  $S_x = 1327x10^3$  mm<sup>3</sup>  $r_y = 40.4$  mm<br>
•  $C_w = 610x1$ **nge shape**<br>
(0.9\*344) = 1493x10<sup>6</sup> mm<sup>3</sup>.<br>
ction<br>
S<sub>x</sub> = 1327x10<sup>3</sup> mm<sup>3</sup> r<sub>y</sub> = 40.4 mm<br>
I<sub>y</sub> = 15.5x10<sup>6</sup> mm<sup>4</sup> J = 0.63x10<sup>6</sup> mm<sup>4</sup><br>
(40.4 $\sqrt{200000/344}$  = 1.71 m

•  $C_w = 610x10^9$  mm<sup>6</sup>  $I_v = 15.5x10^6$  mm<sup>4</sup>  $J = 0.63x10^6$  mm<sup>4</sup>

 $r_y = 40.4 \text{ mm}$ <br> $J = 0.63 \times 10^6 \text{ mm}^4$  $T_y = 40.4$  mm<br>J = 0.63x10<sup>6</sup> mm<sup>4</sup>

• 
$$
L_p = 1.76r_y \sqrt{E/F_y} = \frac{1.76 \times 40.4 \sqrt{200000/344}}{1000} = 1.71 m
$$
  
\n•  $L_r = 1.95r_{ts} \frac{E}{0.7F_y} \sqrt{\frac{Jc}{S_x h_0}} \sqrt{1 + \sqrt{1 + 6.76 \left(\frac{0.7F_y S_x h_0}{E} \right)^2}}$ 

• 
$$
r_{ts} = \sqrt{\frac{\sqrt{I_y C_w}}{S_x}} = \sqrt{\frac{\sqrt{15.5 \times 10^6 \times 610 \times 10^9}}{1327 \times 10^3}} = 48.1 \text{ mm}
$$

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# Ex. 4.4 – Beam Design<br>  $\bullet$   $h_0 = D - T_F = 414 - 16 = 398 \text{ mm}$

**11.4** - **Beam Design**  
\n• 
$$
h_0 = D - T_F = 414 - 16 = 398 \text{ mm}
$$
  
\n•  $\sqrt{1 + 6.76 \left(\frac{0.7F_y S_x h_0}{E} \right)^2} = \sqrt{1 + 6.76 \left(\frac{0.7 \times 344}{200000} \frac{1327 \times 10^3 \times 398}{0.63 \times 10^6 \times 1} \right)^2} = 2.81$   
\n•  $L_r = 1.95 \times \frac{48.1}{1000} \times \frac{200000}{0.7 \times 344} \sqrt{\frac{0.63 \times 10^6 \times 1}{1327 \times 10^3 \times 398}} \sqrt{1 + 2.81} = 5.26 \text{ m}$   
\n•  $L_b > L_r$   
\n $M_n = C_b \sqrt{\frac{\pi^2 E I_y}{L_h^2} \left( GJ + \frac{\pi^2 E C_w}{L_h^2} \right)}$ 

$$
L_{\rm b} > L_{\rm r}
$$
  
\n
$$
M_{\rm n} = C_{\rm b} \sqrt{\frac{\pi^2 EI_y}{L_b^2} \left( GJ + \frac{\pi^2 EC_w}{L_b^2} \right)}
$$
  
\n= 1.14  $\sqrt{\frac{\pi^2 \times 200 \times 15.5 \times 10^6}{7500^2} \left( 77 \times 0.63 \times 10^6 + \frac{\pi^2 \times 200 \times 610 \times 10^9}{7500^2} \right)}$   
\n= 222 × 10<sup>3</sup> kN.mm = 222 kN.m  $\langle M_{\rm p} = \frac{1508 \times 10^3 \times 344}{10^6} = 518.8 kN.m$ 

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## $x. 4.4 - \text{Beam Design}$ **<br>• Step IV.** Check if section is adequate<br>•  $M_u > \phi M_n$  Not OK<br>• Step V. Try a larger section. **4 - Beam Design<br>V.** Check if section is adequate<br> $>_{\phi M_n}$  Not OK<br>Y. Try a larger section.<br>Pr few trials select W16 x 67  $\phi M_n = 497.7 > M_u$  OK **1.4 – Beam Design<br>
• Step IV.** Check if section is adequate<br>
•  $M_u > \phi M_n$  Not OK<br>
• Step V. Try a larger section.<br>
• After few trials select W16 x 67  $\phi M_n = 497.7 > M_u$  OK **4.4 - Beam Design<br>
tep IV.** Check if section is adequate<br>
•  $M_u > \phi M_n$  Not OK<br> **tep V.** Try a larger section.<br>
• After few trials select W16 x 67  $\phi M_n = 497.7 > M_u$  OK<br>
tep VI. Check for local buckling. Ex.  $4.4 -$  Beam Design<br>
• Step IV. Check if section is adequate<br>
•  $M_{\rm u} > \phi M_{\rm n}$  Not OK

- $M_{\text{u}}$  >  $\phi M_{\text{n}}$  Not OK
- -

**4.4 – Beam Design<br>
• Step IV.** Check if section is adequate<br>
•  $M_u > \phi M_n$  Not OK<br>
• Step V. Try a larger section.<br>
• After few trials select W16 x 67  $\phi M_n = 497.7 > M_u$  OK<br>
• Step VI. Check for local buckling.<br>  $\lambda = B_f / 2T_f =$ **4.4 – Beam Designship of the Beam Section**<br>
Step IV. Check if section is adequate<br>
• M<sub>u</sub> >  $\phi$ M<sub>n</sub> Not OK<br>
Step V. Try a larger section.<br>
• After few trials select W16 x 67  $\phi$ M<sub>n</sub> = 4<br>
Step VI. Check for local buckli **4 - Beam Design**<br>
W. Check if section is adequate<br>
y. PoM<sub>n</sub> Not OK<br>
W. Try a larger section.<br>
ter few trials select W16 x 67  $\phi M_n = 497.7 > M_u$  OK<br>
VI. Check for local buckling.<br>  $2T_f = 7.7$ ; Corresponding  $\lambda_p = 0.38$  (E/F  $> M_u$  **OK**<br>  $)0.5 = 9.19$ <br>  $)0.5 = 90.5$ Therefore,  $\lambda < \lambda_p$  - compact flange **Cam Design<br>
section is adequate**<br>
Not OK<br>
er section.<br>
elect W16 x 67  $\phi M_n = 497.7 > M_u$  <u>OK</u><br>
r local buckling.<br>
Corresponding  $\lambda_p = 0.38$  (E/F<sub>y</sub>)<sup>0.5</sup> = 9.19<br>
- compact flange<br>
Corresponding  $\lambda_p = 3.76$  (E/F<sub>y</sub>)<sup>0.5</sup> **Step IV.** Check if section is adequate<br>
•  $M_u > \phi M_n$  Not OK<br> **Step V.** Try a larger section.<br>
• After few trials select W16 x 67  $\phi M_n = 2$ <br> **Step VI.** Check for local buckling.<br>  $\lambda = B_f / 2T_f = 7.7$ ; Corresponding  $\lambda_p = 0.38$ Check if section is adequate<br>  $\phi M_n$  Not OK<br>
Try a larger section.<br>
few trials select W16 x 67  $\phi M_n = 497.7 > M_u$  OK<br>
Check for local buckling.<br>  $T_f = 7.7$ ; Corresponding  $\lambda_p = 0.38$  (E/F<sub>y</sub>)<sup>0.5</sup> = 9.19<br>  $\lambda, \lambda < \lambda_p$  - compa > M<sub>u</sub> <u>OK</u><br>  $y^{0.5} = 9.19$ <br>  $y^{0.5} = 90.5$ Therefore,  $\lambda < \lambda_p$  - compact web of a dequate<br>  $6 \times 67$   $\phi M_n = 497.7 > M_u$  **OK**<br>
uckling.<br>
uckling.<br>
inding  $\lambda_p = 0.38$  (E/F<sub>y</sub>)<sup>0.5</sup> = 9.19<br>
it flange<br>
inding  $\lambda_p = 3.76$  (E/F<sub>y</sub>)<sup>0.5</sup> = 90.5<br>
- compact web<br>
- OK!<br>
the method for designing beams and •  $M_u > \phi M_n$  Not OK<br> **Step V.** Try a larger section.<br>
• After few trials select W16 x 67  $\phi M_n = 497.7 > M_u$  **OK**<br> **Step VI.** Check for local buckling.<br>  $\lambda = B_f / 2T_f = 7.7$ ; Corresponding  $\lambda_p = 0.38$  (E/F<sub>y</sub>)<sup>0.5</sup> = 9.19<br>
There **Step V.** Try a larger section.<br>
• After few trials select W16 x 67  $\phi M_n = 497.7 > M_u$  OK<br> **Step VI.** Check for local buckling.<br>  $\lambda = B_f / 2T_f = 7.7$ ; Corresponding  $\lambda_p = 0.38$  (E/F<sub>y</sub>)<sup>0.5</sup> = 9.19<br>
Therefore,  $\lambda < \lambda_p$  - compa • After few trials select W16 x 67  $\phi$ M<sub>n</sub> = 497.7 > M<sub>u</sub> **OM**<br>• **Step VI.** Check for local buckling.<br> $\lambda = B_f / 2T_f = 7.7$ ; Corresponding  $\lambda_p = 0.38$  (E/F<sub>y</sub>)<sup>0.5</sup> = 9.19<br>Therefore,  $\lambda < \lambda_p$  - compact flange<br> $\lambda = H/T_w = 35.9$ ; Step VI. Check for local buckling.<br>  $\lambda = B_f / 2T_f = 7.7$ ; Corresponding  $\lambda_p = 0.38$  (E/F<sub>y</sub>)<sup>0.5</sup> = 9<br>
Therefore,  $\lambda < \lambda_p$  - compact flange<br>  $\lambda = H/T_w = 35.9$ ; Corresponding  $\lambda_p = 3.76$  (E/F<sub>y</sub>)<sup>0.5</sup> = 9<br>
Therefore,  $\lambda < \lambda_p$  for local buckling.<br>
Corresponding  $\lambda_p = 0.38$  (E/F<sub>y</sub>)<sup>0.5</sup> = 9.19<br>
Corresponding  $\lambda_p = 3.76$  (E/F<sub>y</sub>)<sup>0.5</sup> = 90.5<br>
- compact web<br>
- OK!<br>
constrates the method for designing beams and<br>
1.0)<br>
can be obtained from Tables t

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