5.6 Selecting Sections

The objective of the selection process is, generally, to select the least cost (this is also frequently the lightest) member that satisfies the design criteria. For beams, there are multiple limit states to consider. The selection criteria can be stated as: select the lightest section such that:

- Req'd $M_n \leq$ Actual M_n,
- Req'd $V_n \leq$ Actual V_n, and
- Actual $\delta \leq$ Allowed δ.

5.7 Shear Strength Limit State

Beam shear strength must be provided to resist the anticipated applied beam shears. In steel members, the elements of the cross section that resist shear may be very slender. As a result the shear elements may be subject to the normal ranges of the buckling curve, including plastic, inelastic buckling, and elastic buckling behaviors. The distribution of elastic beam shear stress on a given cross section is determined by the following equation:

$$\tau = VQ/(Ib)$$

Where: τ is the shear stress at some point on the cross section.

- V is the shear force acting on the cross section.
- Q is the first moment of area “above” the point where of interest is.
- I is the moment of inertia of the cross section.
- b is the breadth (i.e. width), parallel to the axis of bending, of the cross section at the point of interest.

The graph of this equation over the height of a rectangular section and an "I" shaped section is shown in Figure 5-7-1. Its appear that for I-shapes bent about their major axis, it is assumed that only the web resists the shear and that the intensity of shear stress is uniform throught the depth. The design shear strength.

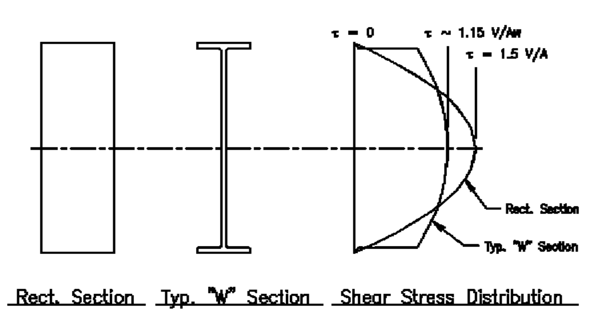

Figure 5-7-1 Shear stress distribution

For I-rolled for limit state of shear yielding of the web is (Sec. G2, pp. 64 to 65):

$$V_d = \varphi_v V_n = \varphi_v F_{yv} A_w = \varphi_v C_v (0.6F_y) A_w$$

Where: $\varphi_v = 0.90$

- F_{yv} is the shear yield strength of the steel$=0.6F_y$
- A_w is the shear are of a web. For I shaped members including channels, A_w equals the overall depth times the web thickness, d t_w.

$$V_d = \varphi_v V_n = 0.54 F_y A_w C_v$$

- C_v is a modifier that accounts for buckling behavior of the web.

$$\frac{h}{t_w} \le 1.10\sqrt{\frac{k_v E}{F_y}} \qquad C_v = 1.0$$

$$1.10\sqrt{\frac{k_v E}{F_y}} < \frac{h}{t_w} \le 1.37\sqrt{\frac{k_v E}{F_y}} \qquad C_v = \frac{1.10\sqrt{\frac{k_v E}{F_y}}}{\frac{h}{t_w}}$$

$$\frac{h}{t_w} > 1.37\sqrt{\frac{k_v E}{F_y}} \qquad C_v = \frac{1.51 E k_v}{\left(\frac{h}{t_w}\right)^2 F_y}$$

For webs without transverse stiffeners and with

$$\frac{h}{t_w} < 260 \qquad k_v = 5$$

5.8 *Deflection Limit State*

The calculations of deflection are done at service (i.e. actual) levels and for load combinations that make sense for the project and/or member under consideration. Typically, two different loadings are considered: Total load (dead plus transient loads such as live load and snow) and transient load only.
Total load deflections are important because these will have an impact on nonstructural elements that are near to or attached to the beam. The transient load deflections are important for maintaining the comfort of occupants. In the absence of more specific criteria, criteria for structures with brittle finishes (as found in code documents for years) is frequently used.
Standard American practice for buildings has been to limit service live-load deflections to approximately 1/360 of the span length.
This deflection is supposedly the largest value that ceiling joists can deflect without causing cracks in underlying plaster. The 1/360 deflection is only one of many maximum deflection values in use because of different loading situations, different engineers, and different specifications. (Table 3-23, pp. 3-211 to 3-226, for calculating moment, shear and deflection for different support conditions)

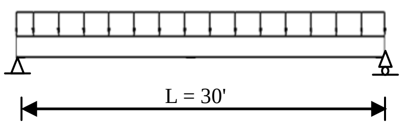

Figure 5-8: Example problem 5-3

***Example Problem* 5-3:** Select s standard W-shape of A992 Gr. 50 steel for used as simply supported beam of 30-ft span, as shown in Figure 5-8. the beam has continuous lateral supports and support a uniform service live load of 4.5 kips/ft. Max allowable live load deflection is 1.5".

Solution: - Ignore the beam weight initially then check
after a selection is made.

W_u = 1.6 L.L.= 1.6(4.5) = 7.2 kips/ft.

$M_u = ⅛\ W_uL^2$ = 810 ft-kips

Assume the shape is compact with full lateral support

$M_d = \phi_b M_{px} = \phi_b\ F_y Z_x \geq M_u$ =810 ft-kips

$Z_x \geq M_u/\phi_b\ F_y$= 216 in^3

Try W 24×84 (LRFDM p.3-16), Z_x = 224 in^3

W_u = 1.2 D.L. + 1.6 L.L.= 1.2(0.084) + 1.6(4.5) = 7.3 kips/ft.

$M_u = ⅛\ W_uL^2$ = 821.4 ft-kips

$Z_{x,\ req} = M_u/\phi_b\ F_y$= 219 in^3 < 224 in^3 OK

This shape is compact (noncompact shape are marked as such table)

$V_u = ½\ W_uL$ = 110 kips

$\lambda_w = 21/0.47 = 44.68 < \lambda_{pv} = 2.45\sqrt{\dfrac{E}{F_y}} = 59$

$\phi_b\ V_n = \phi_b\ 0.6F_yA_w$ = 0.9*0.6*50*(24.1*0.47) = 306 kips $>V_u$ OK

$\Delta_{L.L} = \dfrac{5W_{LL}L^4}{384EI_x} = 1.19in < 1.5$ (p. 3-211) OK

***Example Problem* 5-4:** Select s standard W18×?-shape of A992 Gr. 50 steel for used as simply supported beam of 30-ft span, as shown in Figure 5-9. The beam supports a two equal concentrated service live and dead load of 24 and 10 kips/ft, respectively, at one-third and two-third. The beam is supported laterally at the points of load application. Max allowable live load deflection is 1.3".

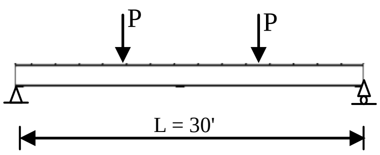

Figure: Example problem 5-4

Solution: - Ignore the beam weight initially and assume that $L_b < L_p$. Then one can use LRFDM Max. factored uniform loads Tables but first enter to the LRFD Table of Concentrated Load Equivalents on p.(3-208):

- Equivalent uniform load = 2.667 P_u (Table 3-22a, p. 3-208)

- Required factored uniform load:

$$P_u = 1.2(10) + 1.6(24) = 50.4$$
$$W_u = 2.667\ P_u = 135 \text{ kips}$$

- Enter factored uniform loads Table for F_y=50 ksi and $W_u \geq$ 135 kips

- 1st trial – W 18×71: W_u = 146 kips > 135 kips (p.3-58)
  L_b= 10' , L_p= 6' and L_r = 19.6' (p.3-16)
  Since $L_p < L_b < L_r$

$$\phi_b M_n = C_b[\phi_b M_{px} - BF(L_b - L_p) \leq \phi_b M_{px}$$

C_b = 1…. (Table 3-1, p. 3-10) , $\phi_b M_{px}$= 548 kip-ft , BF= 15.7 …… p.(3-16)

M_d =485.2 kip-ft

$$M_{u,\ max} = (50.4*10) + \frac{0.071*35^2}{8} = 511.3 \text{ kip-ft} > M_d \quad \text{Not. Ok.}$$

- 2nd trial – W 18×76: W_u = 163 kips > 135 kips (p.4-87)
  L_b= 10' , L_p= 9.22' and L_r = 27.1'

$$\phi_b M_n = C_b[\phi_b M_{px} - BF(L_b - L_p) \leq \phi_b M_{px}$$

C_b = 1…. (Table 3-1, p. 3-10) , $\phi_b M_{px}$= 611 kip-ft , BF= 12.8 …… p.(3-16)

M_d =601 kip-ft

$$M_{u,\ max} = (50.4*10) + \frac{0.076*35^2}{8} = 512 \text{ kip-ft} < M_d \text{ ….. OK}$$

Use W 18×76

- Check for shear requirement: $V_u = P_u$ = 1.2(10) +1.6(24) = 50.4 kips
  $\lambda_w = 15.5/0.425 = 36.47 < \lambda_{pv} = 2.45\sqrt{\frac{E}{F_y}} = 59$
  $\phi_b V_n = \phi_b\ 0.6 F_y A_w$ = 0.9*0.6*50*(15.5*0.425) = 177.86 kips >V_u OK.

- Check for live load deflection: M_{LL} = 24*10 = 240 kip-ft
  The max. deflection is (See p. 3-7):

$$\Delta_{\text{max. L.L (@ mid span)}} = \frac{M_{LL} L^2}{C_1 I_x} = \frac{240(30)^2}{158(1330)} 1.03 in < 1.3 \quad \text{OK.}$$

<u>Example Problem</u> 5-5: Select standard W-shape of A992 Gr. 50 steel for used as framed girder of 35-ft span, as shown in Figure 5-10, using LRFD Beam Design Moment Charts. it is supported a two equal concentrated service, which produce a required moment of 440 kip-ft in the center between the two loads. The beam is supported laterally at the points of load application.

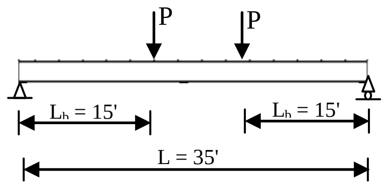

Figure : Example problem 5-5

Solution: - LRFD Beam Design Moment Charts (p.4-113 to p.4-166) can be used for $L_p < L_b \leq L_r$ and $C_b = 1.0$

For this load condition, $C_b = 1.0$ (the moment is uniform between the two loads. Since the 15' is longest unbraced length, one can expected that $L_p < L_b < L_r$.

With total span of 35' and M_u=440 kip-ft., assume weight of beam 70 ibs/ft

$$M_{u,\ total} = 440 + \left(1.2 * \frac{0.07 * 35^2}{8}\right) = 453 \text{ kip-ft.}$$

Enter the chart with L_b =15' and M_u=453 kip-ft, any beam listed above and to the right of intersecting point satisfies the design moment requirement. The solid portion of curves indicated the most economical section by weight, while the dashed portion of curves indicated ranges in which a lighter weight beam will satisfy the loading conditions.
For our example: Use W21×68 (p. 3-121)

$$M_d = 457 \text{ kip-ft} > M_u = 453 \text{ kip-ft}$$