LRFD design of steel beams
Load and Resistance Factor Design

- loads on structures are
  - not constant
  - can be more influential on failure
  - happen more or less often
  - UNCERTAINTY

\[ \sum \gamma_i R_i \leq \phi R_n \]

- \( \phi \) - resistance factor
- \( \gamma \) - load factors for types of loads (\( R \))
- \( R_n \) – nominal strength
Load Types

- $D = \text{dead load}$
- $L = \text{live load}$
- $L_r = \text{live roof load}$
- $W = \text{wind load}$
- $S = \text{snow load}$
- $E = \text{earthquake load}$
- $R = \text{rainwater load or ice water load}$

Figure 1.13  Wind loads on a structure.
Load Combinations

• “summation” means AND (combine)

- $1.4(D + F)$
- $1.2(D + F + T) + 1.6(L + H) + 0.5(L_r \text{ or } S \text{ or } R)$
- $1.2D + 1.6(L_r \text{ or } S \text{ or } R) + (L \text{ or } 0.8W)$
- $1.2D + 1.6W + L + 0.5(L_r \text{ or } S \text{ or } R)$
- $1.2D + 1.0E + L + 0.2S$
- $0.9D + 1.6W + 1.6H$
- $0.9D + 1.0E + 1.6H$
Steel Materials

- **ASTM A36 – carbon**
  - plates, angles
  - $F_y = 36 \text{ ksi}$ & $F_u = 58 \text{ ksi}$
- **ASTM A572 – high strength low-alloy**
  - some beams
  - $F_y = 60 \text{ ksi}$ & $F_u = 75 \text{ ksi}$
- **ASTM A992 – for building framing**
  - most beams
  - $F_y = 50 \text{ ksi}$ & $F_u = 65 \text{ ksi}$
Flexure

- limit is in plastic stress range

$$\sum \gamma_i R_i = M_u \leq \phi_b M_n = 0.9 F_y Z$$

- $M_u$ - maximum moment
- $\phi_b$ - resistance factor for bending = 0.9
- $M_n$ - nominal moment (ultimate capacity)
- $F_y$ - yield strength of the steel
- $Z$ - plastic section modulus*
**Internal Moments - at yield**

- material hasn’t failed

\[
M_y = \frac{I}{c} f_y = \frac{bh^2}{6} f_y
\]

\[
= \frac{b(2c)^2}{6} f_y = \frac{2bc^2}{3} f_y
\]
Internal Moments - ALL at yield

- all parts reach yield
- plastic hinge forms
- ultimate moment
- $A_{\text{tension}} = A_{\text{compression}}$

\[ M_p = bc^2 f_y = \frac{3}{2} M_y \]
n.a. of Section at Plastic Hinge

- cannot guarantee at centroid
- $f_y A_1 = f_y A_2$
- moment found from yield stress times moment area

$$M_p = f_y A_1 d = f_y \sum_{n.a} A_i d_i$$
Plastic Hinge Development

(a) $M < M_Y$

(b) $M = M_Y$

(c) $M > M_Y$

(d) $M = M_p$
Plastic Hinge Examples

- stability can be effected
Plastic Section Modulus

- **shape factor,** $k$

  
  $$
  k = \frac{M_p}{M_y}
  $$

  = 3/2 for a rectangle

  ≈ 1.1 for an I

- **plastic modulus,** $Z$

  
  $$
  Z = \frac{M_p}{f_y}
  $$
Shear

\[ \sum \gamma_i R_i = V_u \leq \phi_v V_n = 0.9(0.6F_{yw} A_w) \]

- \( V_u \) - maximum shear
- \( \phi_v \) - resistance factor for shear = 0.9
- \( V_n \) - nominal shear
- \( F_{yw} \) - yield strength of the steel in the web
- \( A_w \) - area of the web = \( t_w d \)
Flexure Design

- limit states for beam failure
  1. yielding
  2. lateral-torsional buckling*
  3. flange local buckling
  4. web local buckling

- minimum $M_n$ governs

$$\sum \gamma_i R_i = M_u \leq \phi_b M_n$$
Lateral Torsional Buckling

\[ M_n = C_b \left[ \text{moment based on lateral buckling} \right] \leq M_p \]

\[ C_b = \frac{12.5M_{\text{max}}}{2.5M_{\text{max}} + 2M_A + 4M_B + 3M_C} \]

- \( C_b = \) modification factor
- \( M_{\text{max}} = |\text{max moment}|, \) unbraced segment
- \( M_A = |\text{moment}|, \) 1/4 point
- \( M_B = |\text{moment}|, \) center point
- \( M_C = |\text{moment}|, \) 3/4 point
Beam Design Charts

Beam Design Moments ($\phi_b=0.9, C_b=1.0, F_y=50$ ksi)

$\phi B M_d$, Design Moment (1 kip-ft increments)

$L_d$, Unbraced Length (0.5 ft increments)
Charts & Deflections

• beam charts
  – solid line is most economical
  – dashed indicates there is another more economical section
  – self weight is included in $M_n$

• deflections
  – no factors are applied to the loads
  – often governs the design