Steel construction: materials & beams
Steel Beam Design

- American Institute of Steel Construction
  - Manual of Steel Construction
  - ASD & LRFD
  - now combined in 13th ed. (2005)
Steel Materials

- **steel grades**
  - 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*
    - $F_{y} = 50 \text{ ksi}$ & $F_{u} = 65 \text{ ksi}$
Steel Properties

- high strength to weight ratio
- elastic limit – yield ($F_y$)
- inelastic – plastic
- ultimate strength ($F_u$)
- ductile
- strength sensitive to temperature
- can corrode
- fatigue
Structural Steel

- standard rolled shapes (W, C, L, T)
- open web joists
- plate girders
- decking
Steel Construction

• welding
• bolts
ASD Steel Design

- **bending (braced)**  \( F_b = 0.66F_y \)
- **bending (unbraced*)**  \( F_b = 0.60F_y \)
- **shear**  \( F_v = 0.40F_y \)
- **shear (bolts)**  *tabulated*
- **shear (welds)**  \( F_v = 0.30F_{\text{weld}} \)

* flanges in compression can buckle
ASD Steel Design

- braced vs. unbraced
LRFD

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

\[
R_u = \gamma_D R_D + \gamma_L R_L \leq \phi R_n
\]

\(\phi\) - resistance factor
\(\gamma\) - load factor for *(D)ead & *(L)ive load
LRFD Steel Beam Design

- limit state is yielding all across section
- outside elastic range
- load factors & resistance factors
LRFD Load Combinations

1.4(D + F)

1.2(D + F + T) + 1.6(L + H) + 0.5(L_r or S or R)

1.2D + 1.6(L_r or S or R) + (L or 0.8W)

1.2D + 1.6W + L + 0.5(L_r or S or R)

1.2D + 1.0E + L + 0.2S)

0.9D + 1.6W + 1.6H

0.9D + 1.0E + 1.6H

ASCE-7 (2002)
Beam Design Criteria (revisited)

- **strength design**
  - bending stresses predominate
  - shear stresses occur

- **serviceability**
  - limit deflection
  - stability

- **superpositioning**
  - use of beam charts
  - elastic range only!
  - “add” moment diagrams
  - “add” deflection CURVES (not maximums)
Steel Beams

- lateral stability - bracing
- local buckling – stiffen, or bigger $I_y$
Local Buckling

- steel I beams
- flange
  - buckle in direction of smaller radius of gyration
- web
  - force
  - “crippling”
Local Buckling

- flange
- web

Figure 2-5. Flange Local Bending Limit State (Beedle, L.S., Christopher, R., 1964)

Figure 2-7. Web Local Buckling Limit State (SAC Project)
Shear in Web

- panels in plate girders or webs with large shear
- buckling in compression direction
- add stiffeners

Stiffeners to prevent lateral buckling

(a) Shear Failure

(b) Shear Buckling
Steel Beams

- **bearing**
  - provide adequate area
  - prevent local yield of flange and web
LRFD - Flexure

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

- \( M_u \): 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

\( \approx 1.1 \) for an I

• plastic modulus, \( Z \)

\[ Z = \frac{M_p}{f_y} \]
LRFD - 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 = $t_w d$
LRFD - 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

\[
\sum \gamma_i R_i = M_u \leq \phi_b M_n
\]

\[
\frac{L_p}{F_y} = \frac{300 \times r_y}{\sqrt{F_y}}
\]
Compact Sections

- plastic moment can form before any buckling
- criteria

\[ \frac{b_f}{2t_f} \leq \frac{65}{\sqrt{F_y}} \]

- and

\[ \frac{h_c}{t_w} \leq \frac{640}{\sqrt{F_y}} \]
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)

$\phi_b M_{u}$, Design Moment (1 kip-ft increments)

$L_b$, 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 NOT included in $M_n$

• deflections
  – no factors are applied to the loads
  – often governs the design
Design Procedure (revisited)

1. Know \( F_{all} \) for the material or \( F_U \) for LRFD

2. Draw \( V \) & \( M \), finding \( M_{max} \)

3. Calculate \( S_{req'd} \) \( (f_b \leq F_b) \) or \( Z \)

4. Choose (economical) section from section or beam capacity charts
Beam Design (revisited)

4*. Include self weight for $M_{\text{max}}$
   - and repeat 3 & 4 if necessary

5. Consider lateral stability

Unbraced roof trusses were blown down in 1999 at this project in Moscow, Idaho.

Photo: Ken Carper
6. Evaluate shear stresses - horizontal

- \((f_v \leq F_v)\) or \((V_u \leq \phi V_n)\)

- **W and rectangles**
  \[ f_{v_{-\text{max}}} = \frac{3V}{2A} \approx \frac{V}{A_{\text{web}}} \]

- **thin walled sections**
  \[ f_{v_{-\text{max}}} = \frac{VQ}{I_b} \]
Beam Design (revisited)

7. Provide adequate bearing area at supports

\[ f_p = \frac{P}{A} \leq F_p \]
Beam Design (revisited)

8. Evaluate torsion

\( (f_v \leq F_v) \)

- **circular cross section**
  \[ f_v = \frac{T \rho}{J} \]

- **rectangular**
  \[ f_v = \frac{T}{c_1 ab^2} \]

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Beam Design (revisited)

9. Evaluate deflections – NO LOAD FACTORS

\[ y_{\text{max}}(x) = \Delta_{\text{actual}} \leq \Delta_{\text{allowable}} \]
### Load Tables & Equivalent Load

- **uniformly distributed loads**
- **equivalent “w”**

\[
M_{\text{max}} = \frac{w_{\text{equivalent}} L^2}{8}
\]

#### Load for live load deflection limit
- in RED, total in BLACK

---

**STANDARD LOAD TABLE/OPEN WEB STEEL JOISTS, K-SERIES**

Based on a Maximum Allowable Tensile Stress of 30 ksi

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- **8**
- **9**
- **10**
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**Steel Beams 37**

**Lecture 16**

**Elements of Architectural Structures**

**ARCH 614**

**S2007abn**