Steel Beam Design

- American Institute of Steel Construction
  - Manual of Steel Construction
  - ASD & LRFD
  - combined in 2005

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

Steel Materials

- steel grades
  - ASTM A36 – carbon
    - plates, angles
    - $F_y = 36$ ksi & $F_u = 58$ ksi
  - ASTM A572 – high strength low-alloy
    - some beams
    - $F_y = 60$ ksi & $F_u = 75$ ksi
  - ASTM A992 – for building framing
    - most beams
    - $F_y = 50$ ksi & $F_u = 65$ ksi
Structural Steel

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

Steel Construction

- welding
- bolts

Unified Steel Design

- ASD
  \[ R_a \leq \frac{R_n}{\Omega} \]
  - bending (braced) \( \Omega = 1.67 \)
  - bending (unbraced*) \( \Omega = 1.67 \)
  - shear \( \Omega = 1.5 \) or 1.67
  - shear (bolts & welds) \( \Omega = 2.00 \)
  - shear (welds) \( \Omega = 2.00 \)

* flanges in compression can buckle
**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

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**LRFD Steel Beam Design**

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

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**LRFD Load Combinations**

- 1.4D
- 1.2D + 1.6L + 0.5(L_r or S or R)
- 1.2D + 1.6(L_r or S or R) + (L or 0.5W)
- 1.2D + 1.0W + L + 0.5(L_r or S or R)
- 1.2D + 1.0E + L + 0.2S
- 0.9D + 1.0W
- 0.9D + 1.0E
  - F has same factor as D in 1-5 and 7
  - H adds with 1.6 and resists with 0.9 (permanent)

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**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

Shear in Web

- panels in plate girders or webs with large shear
- buckling in compression direction
- add stiffeners
Shear in Web
• plate girders and stiffeners

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

Plastic Hinge Examples

- stability can be effected
Plastic Section Modulus

- **shape factor, k**
  \[ k = \frac{M_p}{M_y} \]
  
  = 3/2 for a rectangle
  
  \[ \approx 1.1 \text{ for an } I \]

- **plastic modulus, }\]
  \[ Z = \frac{M_p}{f_y} \]

LRFD - Shear

\[ \Sigma \gamma_i R_i = V_u \leq \phi_v V_n = 1.0 \left( 0.6 F_{yw} A_w \right) \]

- **maximum shear**
- **resistance factor for shear** = 0.9
- **nominal shear**
- **yield strength of the steel in the web**
- **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 }\]
  \[ \Sigma \gamma_i R_i = M_u \leq \phi_b M_n \]

Compact Sections

- **plastic moment can form before any buckling**
- **criteria**
  
  - \[ - \frac{b_f}{2 t_f} \leq 0.38 \left( \frac{E}{F_y} \right) \]
  
  - \[ - \frac{h_c}{t_w} \leq 3.76 \left( \frac{E}{F_y} \right) \]

**TABLE A.3 Properties of W Shapes**
**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

**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

**Beam Design Charts**

**Design Procedure (revisited)**

1. Know unbraced length, material, design method (\( \Omega, \phi \))

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

3. Calculate \( Z_{\text{req'd}} \) \left( f_b \leq F_b \right)
   \( M_u \leq \phi_b M_n \)

4. Choose (economical) section from section or beam capacity charts
Beam Charts by $Z_x$ (pg. 250)

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<th>Designation</th>
<th>$I_x$ (in$^4$)</th>
<th>$I_y$ (in$^4$)</th>
<th>$M_y$ (kip-in)</th>
<th>$M_x$ (kip-in)</th>
<th>$r_x$ (in)</th>
<th>$r_y$ (in)</th>
<th>$X_1$ (in)</th>
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<th>$s_x$ (kip/in$^2$)</th>
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Beam Design (revisited)

4*. Include self weight for $M_{max}$

- 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

Beam Design (revisited)

6. Evaluate shear stresses - horizontal

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

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

- Thin walled sections
  \[
  f_{v_{-max}} = \frac{VQ}{Ib}
  \]

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
  \[ f_v = \frac{T}{c_1ab^2} \]

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

**Steel Arches and Frames**

- solid sections
- open web

http://nisee.berkeley.edu/godden

Freedom Steel
Steel Shell and Cable Structures

Approximate Depths

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