Steel Beam Design

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

Winnipeg DOT

strain hardening

Ultimate strength

Elongation to failure

Specified minimum

strain hardening range
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

LRFD Steel Beam Design

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

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)

LRFD Load Combinations

ASCE-7 (2010)

• 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)
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”

Shear in Web

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

Local Buckling

- flange
- web
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

\[ M_p = f_y A_1 d = f_y \sum A_i d_i \]

Plastic Hinge Development

- stability can be effected

Plastic Hinge Examples
Plastic Section Modulus

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

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

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

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

Compact Sections

- plastic moment can form before any buckling
- criteria
  \[
  - \frac{b_f}{2t_f} \leq 0.38 \frac{E}{F_y}
  \]
  \[
  - \frac{h_c}{t_w} \leq 3.76 \frac{E}{F_y}
  \]
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
- \( M_{\text{max}} \) - |max moment|, unbraced segment
- \( M_A \) - |moment|, 1/4 point
- \( M_B \) = |moment|, center point
- \( M_C \) = |moment|, 3/4 point

Beam Design Charts

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 unbraced length, material, design method (\( \Omega, \phi \))
2. Draw V & M, finding \( M_{\text{max}} \)
3. Calculate \( Z_{\text{req'd}} \) \( \left( f_b \leq F_b \right) \) \( \left( M_u \leq \phi_b M_n \right) \)
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

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**Beam Design (revisited)**

6. Evaluate shear stresses - horizontal

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

- $W$ and rectangles

$$f_{v-\text{max}} = \frac{3V}{2A} \approx \frac{V}{A_{\text{web}}}$$

- thin walled sections

$$f_{v-\text{max}} = \frac{VQ}{Ib}$$

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
  \( f_v = \frac{T}{c_4 ab^2} \)

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9. Evaluate deflections – NO LOAD FACTORS

\[ y_{\text{max}}(x) = \Delta_{\text{actual}} \leq \Delta_{\text{allowable}} \]

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Load Tables & Equivalent Load

- uniformly distributed loads
- equivalent "w"

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

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Steel Arches and Frames

- solid sections
- or open web

http://nisee.berkeley.edu/godden
Steel Shell and Cable Structures

Approximate Depths

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<th>Span (Feet)</th>
<th>Depth (Inches)</th>
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<tr>
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<tr>
<td>141-150</td>
<td>15</td>
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Key:
- Minimum depth
- Maximum depth
- Possible span range
- Minimum span

Ribbed domes
- Cables