Steel Beam Design

- American Institute of Steel Construction
  - Manual of Steel Construction
  - ASD & LRFD
  - combined in 13th ed.

Steel Materials

- smelt iron ore
- add alloying elements
- heat treatments
- iron, carbon
- microstructure

Steel Materials

- cast into billets
- hot rolled
- cold formed
- residual stress
- corrosion-resistant “weathering” steels
- stainless
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

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
Steel Construction

- fire proofing
  - cementicious spray
  - encasement in gypsum
  - intumescent – expands with heat
  - sprinkler system

Unified Steel Design

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

* flanges in compression can buckle

Unified 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**

\[ f_y = 50 \text{ksi} \]
\[ \varepsilon_y = 0.001724 \]

**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

- **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 \( I_y \)**

Compliance: ASCE-7 (2010)
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

Shear in Web

- web

- plate girders and stiffeners
Steel Beams

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

LRFD - Flexure

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

- \( M_u \) - 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

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 (compact shapes)**

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

- \(V_u\) - maximum shear
- \(\phi_v\) - resistance factor for shear = 1.0
- \(V_n\) - nominal shear
- \(F_{yw}\) - yield strength of the steel in the web
- \(A_w\) - area of the web = \(t_wd\)

**Compact Sections**

- plastic moment can form before any buckling
- criteria
  - \[-\frac{b_f}{2t_f} \leq 0.38 \sqrt{\frac{E}{F_y}}\]
  - \[\text{and} \] \[-\frac{h_c}{t_w} \leq 3.76 \sqrt{\frac{E}{F_y}}\]

**LRFD - Flexure Design**

- limit states for beam failure
  1. yielding \(L_p = 1.76\gamma_y \sqrt[4]{\frac{F_y}{E}}\)
  2. lateral-torsional buckling*
  3. flange local buckling
  4. web local buckling
- minimum \(M_n\) governs

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

**Lateral Torsional Buckling**

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

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

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

<table>
<thead>
<tr>
<th>Unbraced Length (in.)</th>
<th>Available Moment (kip-ft)</th>
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<tbody>
<tr>
<td>12</td>
<td>64</td>
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<tr>
<td>14</td>
<td>84</td>
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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 $S_{\text{req'd}}$ ($M_a \leq M_n/\Omega$) or $Z$
   ($M_u \leq \phi_b M_n$)
4. Choose (economical) section from section or beam capacity charts

Beam Charts by $S_x$ (Appendix A)

<table>
<thead>
<tr>
<th>Allowable Stress Design—Selected Beam Shapes</th>
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<tbody>
<tr>
<td>$S_x$</td>
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<tr>
<td>-------</td>
</tr>
<tr>
<td>410</td>
</tr>
<tr>
<td>459</td>
</tr>
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Table 31
Listing of W Shapes in Descending Order of $S_x$ for Beam Design.
Beam Design (revisited)

4*. Include self weight for $M_{max}$
- it’s dead load
- 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
- $(V_a \leq V_n/\Omega)$ or $(V_a \leq \phi V_n)$
  - rectangles and W’s
    $$f_{v,max} = \frac{3V}{2A} \approx \frac{V}{A_{web}}$$
    $$V_n = 0.6 \ F_{yw} A_w$$
  - general
    $$f_{v,max} = \frac{VQ}{I_b}$$
Beam Design (revisited)

8. Evaluate torsion

\[ f_v \leq F_v \]

- circular cross section
  \[ f_v = \frac{T \rho}{J} \]
- rectangular
  \[ f_v = \frac{T}{c_1 ab^2} \]

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

Sloped Beams

- stairs & roofs
- projected live load
- dead load over length
- perpendicular load to beam:
  \[ w_{\perp} = w \cdot \cos \alpha \]
- equivalent distributed load:
  \[ w_{\text{adj.}} = \frac{w}{\cos \alpha} \]
Steel Arches and Frames

• solid sections
  or open web

Steel Shell and Cable Structures

Approximate Depths