Steel Construction

- standard rolled shapes
- open web joists
- plate girders
- decking

Steel
- cast iron – wrought iron - steel
- cables
- columns
- beams
- trusses
- frames

Steel Construction
- welding
- bolts
Steel Construction

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

Steel Materials

• high strength to weight ratio
• ductile
• beam size often limited by deflection
• column size limited by slenderness

Steel Beams

• types
  – manufactured shapes

  castellated

Steel Beams

• types
  – wide flange

Figure 3.22 Stress-strain diagram for mild steel (A36) with key points highlighted.
Steel Beams

- **types**
  - open web joists (manufactured trusses)

Steel Beams

- **types (more)**
  - plate girder
  - decking

Steel Beams

- **lateral stability - bracing**
- **local buckling - stiffen**
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

Steel Beams

• end conditions
  • a) away from connection - full section effective
  • b) high shear – only web effective
Steel Beams

- end conditions
  - c) bolt holes – less material
  - d) local web buckling

Steel Beams

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

Steel Design – Open Web Joists

- SJI: www.steeljoist.com
- Vulcraft: www.vulcraft.com
  - K Series (Standard)
    - 8-30" deep, spans 8-50 ft
  - LH Series (Long span)
    - 18-48" deep, spans 25-96 ft
  - DLH (Deep Long Spans)
    - 52-72" deep, spans 89-144 ft
  - SLH (Long spans with high strength steel)
    - pitched top chord
    - 80-120" deep, spans 111-240 ft
Steel Beam Design

• **American Institute of Steel Construction**
  
  – 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 Beam Design

• **AISC: 14th ed.**
  
  – combined ASD & LRFD
  
  in one volume in 2005
Unified Steel Design

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

* flanges in compression can buckle

LRFD Steel Beam Design

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

Load Types

- **D** = dead load
- **L** = live load
- **L_r** = live roof load
- **W** = wind load
- **S** = snow load
- **E** = earthquake load
- **R** = rainwater load or ice water load
- **T** = effect of material & temperature
**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)

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**Pure 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*

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**Internal Moments - at yield**

- material hasn’t failed

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

\[ = \frac{b(2c)^2}{6} f_y = \frac{2bc^2}{3} f_y \]

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**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
• \( f_y \cdot A_1 = f_y \cdot A_2 \)
• 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 Examples

• stability can be effected

Plastic Hinge Development

\[
\begin{align*}
(\alpha) M < M_y \\
(\beta) \lambda = M_y \\
(\gamma) M > M_y
\end{align*}
\]

Plastic Section Modulus

• shape factor, \( k \)

\[
k = \frac{M_p}{M_y}
\]

= 3/2 for a rectangle

\[
k \approx 1.1 \text{ for an } I
\]

• plastic modulus, \( Z \)

\[
Z = \frac{M_p}{f_y}
\]
**Shear**

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

- \( V_u \) - 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 = \( t_wd \)

**Flexure Design**

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

- **minimum \( M_n \) governs**

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

**Compact Sections**

- **plastic moment can form before any buckling**

- **criteria**

\[ - \frac{b_f}{2t_f} \leq 0.38 \sqrt{\frac{E}{F_y}} \]

\[ - \text{and} \quad \frac{h_c}{t_w} \leq 3.76 \sqrt{\frac{E}{F_y}} \]

**Lateral Torsional Buckling**

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

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

- \( C_b \) - 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

Deflection Limits

- based on service condition
- no “impairment” to serviceability
- avoid ponding
- $L/360$ due to live load for beams & girders supporting plaster (service)

Steel Arches and Frames

- solid sections
  - or open web

Steel Shell and Cable Structures
Approximate Depths

Unified Column Design

- limit states for failure
  \[ P_a \leq \frac{P_n}{\phi} \]
  \[ P_u \leq \phi_c P_n \]

  \[ \phi_c = 0.90 \]
  \[ P_n = F_{cr} A_g \]

  \[ F_{cr} = \begin{cases} 
  0.658 \frac{F_y}{E} & \text{for } \frac{KL}{r} \leq 4.71 \sqrt{\frac{E}{F_y}} \\
  0.877 F_e & \text{for } \frac{KL}{r} > 4.71 \sqrt{\frac{E}{F_y}} 
  \end{cases} \]

  \[ F_e = \frac{\pi^2 E}{(KL/r)^2} \]

Procedure for Analysis

1. calculate \( KL/r \)
   - biggest of \( KL/r \) with respect to x axes and y axis

2. find \( F_{cr} \) from appropriate equation
   - tables are available

3. compute \( P_n = F_{cr} A_g \)
   - or find \( f_c = P_u/A \) or \( P_u/A \)

4. is \( P_a \leq \frac{P_n}{\Omega} \) or \( P_u \leq \phi_P n \)?
   - yes: ok
   - no: insufficient capacity and no good
Procedure for Design

1. guess a size (pick a section)
2. calculate KL/r
   - biggest of KL/r with respect to x axes and y axis
3. find $F_{cr}$ from appropriate equations
   - or find a table
4. compute $P_n = F_{cr}A_g$
   - or find $f_c = P_a/A$ or $P_u/A$

Procedure for Design (cont’d)

5. is $P_a \leq P_n/\Omega$ or $P_u \leq \phi P_n$?
   - yes: ok
   - no: pick a bigger section and go back to step 2.
6. check design efficiency
   - percentage of stress $= \frac{P_r}{P_c} \cdot 100\%$
   - if between 90-100%: good
   - if < 90%: pick a smaller section and go back to step 2.

Column Tables

<table>
<thead>
<tr>
<th>$F_y = 50$ ksi</th>
<th>Available Strength in Axial Compression, kips</th>
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<tr>
<td>W Shapes</td>
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<tr>
<td>Shpk</td>
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<td>$F_e/\Omega_e$</td>
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<td>11</td>
<td>739</td>
</tr>
</tbody>
</table>

Beam-Column Design

- moment magnification ($P-\Delta$)

$$M_u = B_1 M_{\text{max - factored}}$$

$$B_1 = \frac{C_m}{1 - (P_u/P_{e1})}$$

$C_m$ – modification factor for end conditions

- 0.6 – 0.4($M_u/M_0$) or
- 0.85 restrained, 1.00 unrestrained

$$P_{e1} = \text{Euler buckling strength} = \frac{\pi^2 EA}{(Kl/r_f)^2}$$
Beam-Column Design

- LRFD (Unified) Steel
  - for $\frac{P_r}{P_c} \geq 0.2$:
    \[
    \frac{P_u}{\phi_c P_n} + \frac{8}{9} \left( \frac{M_{ux}}{\phi_b M_{nx}} + \frac{M_{uy}}{\phi_b M_{ny}} \right) \leq 1.0
    \]
  - for $\frac{P_r}{P_c} < 0.2$:
    \[
    \frac{P_u}{2\phi_c P_n} + \frac{M_{ux}}{\phi_b M_{nx}} + \frac{M_{uy}}{\phi_b M_{ny}} \leq 1.0
    \]

$P_r$ is required, $P_c$ is capacity

$\phi_c$ - resistance factor for compression = 0.9

$\phi_b$ - resistance factor for bending = 0.9

Construction Supervision

- proper grade material
  - high strength bolts
- quality welds
- proper bolted conditions (ex. sc)
- fabrication and erection of steel frame connection details