Steel Design

Notation:

\( a \) = name for width dimension  
\( A \) = name for area  
\( A_b \) = area of a bolt  
\( A_e \) = effective net area found from the product of the net area \( A_n \) by the shear lag factor \( U \)  
\( A_g \) = gross area, equal to the total area ignoring any holes  
\( A_{gv} \) = gross area subjected to shear for block shear rupture  
\( A_n \) = net area, equal to the gross area subtracting any holes, as is \( A_{net} \)  
\( A_{nt} \) = net area subjected to tension for block shear rupture  
\( A_{nv} \) = net area subjected to shear for block shear rupture  
\( A_w \) = area of the web of a wide flange section  
\( AISC \) = American Institute of Steel Construction  
\( ASD \) = allowable stress design  
\( b \) = name for a (base) width  
\( b_f \) = width of the flange of a steel beam cross section  
\( B_1 \) = factor for determining \( M_u \) for combined bending and compression  
\( c \) = largest distance from the neutral axis to the top or bottom edge of a beam  
\( c_l \) = coefficient for shear stress for a rectangular bar in torsion  
\( C_b \) = lateral torsional buckling modification factor for moment in ASD & LRFD steel beam design  
\( C_c \) = column slenderness classification constant for steel column design  
\( C_m \) = modification factor accounting for combined stress in steel design  
\( C_v \) = web shear coefficient  
\( d \) = calculus symbol for differentiation  
\( d_b \) = nominal bolt diameter  
\( D \) = shorthand for dead load  
\( DL \) = shorthand for dead load  
\( e \) = eccentricity  
\( E \) = shorthand for earthquake load  
\( f_c \) = axial compressive stress  
\( f_b \) = bending stress  
\( f_p \) = bearing stress  
\( f_v \) = shear stress  
\( f_{v-max} \) = maximum shear stress  
\( f_y \) = yield stress  
\( F \) = shorthand for fluid load  
\( F_{allow(able)} \) = allowable stress  
\( F_a \) = allowable axial (compressive) stress  
\( F_b \) = allowable bending stress  
\( F_{cr} \) = flexural buckling stress  
\( F_e \) = elastic critical buckling stress  
\( F_{EXX} \) = yield strength of weld material  
\( F_n \) = nominal strength in LRFD  
\( F_p \) = allowable bearing stress  
\( F_t \) = allowable tensile stress  
\( F_u \) = ultimate stress prior to failure  
\( F_y \) = allowable shear stress  
\( F_y \) = yield strength  
\( F_{yw} \) = yield strength of web material  
\( F.S. \) = factor of safety  
\( g \) = gage spacing of staggered bolt holes  
\( G \) = relative stiffness of columns to beams in a rigid connection, as is \( \Psi \)  
\( h \) = name for a height  
\( h_c \) = height of the web of a wide flange steel section  
\( H \) = shorthand for lateral pressure load  
\( I \) = moment of inertia with respect to neutral axis bending  
\( I_{trial} \) = moment of inertia of trial section  
\( I_{req'd} \) = moment of inertia required at limiting deflection  
\( I_y \) = moment of inertia about the y axis  
\( J \) = polar moment of inertia
\( k \) = distance from outer face of W flange to the web toe of fillet
= shape factor for plastic design of steel beams

\( K \) = effective length factor for columns, as is \( k \)

\( l \) = name for length

\( \ell_b \) = length of beam in rigid joint

\( \ell_c \) = length of column in rigid joint

\( L \) = name for length or span length
= shorthand for live load

\( L_b \) = unbraced length of a steel beam

\( L_c \) = clear distance between the edge of a hole and edge of next hole or edge of the connected steel plate in the direction of the load

\( L_e \) = effective length that can buckle for column design, as is \( \ell_e \)

\( L_r \) = shorthand for live roof load
= maximum unbraced length of a steel beam in LRFD design for inelastic lateral-torsional buckling

\( L_p \) = maximum unbraced length of a steel beam in LRFD design for full plastic flexural strength

\( L' \) = length of an angle in a connector with staggered holes

\( LL \) = shorthand for live load

\( LRFD \) = load and resistance factor design

\( M \) = internal bending moment

\( M_a \) = required bending moment (ASD)

\( M_n \) = nominal flexure strength with the full section at the yield stress for LRFD beam design

\( M_{\text{max}} \) = maximum internal bending moment

\( M_{\text{max-adj}} \) = maximum bending moment adjusted to include self weight

\( M_p \) = internal bending moment when all fibers in a cross section reach the yield stress

\( M_u \) = maximum moment from factored loads for LRFD beam design

\( M_y \) = internal bending moment when the extreme fibers in a cross section reach the yield stress

\( n \) = number of bolts

\( n.a. \) = shorthand for neutral axis

\( N \) = bearing length on a wide flange steel section
= bearing type connection with threads included in shear plane

\( p \) = bolt hole spacing (pitch)

\( P \) = name for load or axial force vector

\( P_a \) = allowable axial force
= required axial force (ASD)

\( P_{\text{allowable}} \) = allowable axial force

\( P_c \) = available axial strength

\( P_{el} \) = Euler buckling strength

\( P_n \) = nominal column load capacity in LRFD steel design

\( P_r \) = required axial force

\( P_u \) = factored column load calculated from load factors in LRFD steel design

\( Q \) = first moment area about a neutral axis
= generic axial load quantity for LRFD design

\( r \) = radius of gyration

\( r_y \) = radius of gyration with respect to a y-axis

\( R \) = generic load quantity (force, shear, moment, etc.) for LRFD design
= shorthand for rain or ice load
= radius of curvature of a deformed beam

\( R_a \) = required strength (ASD)

\( R_n \) = nominal value (capacity) to be multiplied by \( \phi \) in LRFD and divided by the safety factor \( \Omega \) in ASD

\( R_u \) = factored design value for LRFD design

\( s \) = longitudinal center-to-center spacing of any two consecutive holes

\( S \) = shorthand for snow load
= section modulus
= allowable strength per length of a weld for a given size

\( S_{\text{req'd}} \) = section modulus required at allowable stress

\( S_{\text{req'd-adj}} \) = section modulus required at allowable stress when moment is adjusted to include self weight

\( SC \) = slip critical bolted connection
Steel Design

Structural design standards for steel are established by the Manual of Steel Construction published by the American Institute of Steel Construction, and uses Allowable Stress Design and Load and Factor Resistance Design. With the 13th edition, both methods are combined in one volume which provides common requirements for analyses and design and requires the application of the same set of specifications.
### Materials

American Society for Testing Materials (ASTM) is the organization responsible for material and other standards related to manufacturing. Materials meeting their standards are guaranteed to have the published strength and material properties for a designation.

- **A36** – carbon steel used for plates, angles  
  \( F_y = 36 \text{ ksi}, \ F_u = 58 \text{ ksi}, \ E = 29,000 \text{ ksi} \)
- **A572** – high strength low-alloy use for some beams  
  \( F_y = 60 \text{ ksi}, \ F_u = 75 \text{ ksi}, \ E = 29,000 \text{ ksi} \)
- **A992** – for building framing used for most beams  
  \( F_y = 50 \text{ ksi}, \ F_u = 65 \text{ ksi}, \ E = 29,000 \text{ ksi} \)
  (A572 Grade 50 has the same properties as A992)

#### ASD

\[
R_u \leq \frac{R_n}{\Omega}
\]

where  
- \( R_u \) = required strength (dead or live; force, moment or stress)  
- \( R_n \) = nominal strength specified for ASD  
- \( \Omega \) = safety factor

Factors of Safety are applied to the limit stresses for allowable stress values:

- Bending (braced, \( L_b < L_p \))  
  \( \Omega = 1.67 \)
- Bending (unbraced, \( L_p < L_b \) and \( L_b > L_r \))  
  \( \Omega = 1.67 \) (nominal moment reduces)
- Shear (beams)  
  \( \Omega = 1.5 \) or 1.67
- Shear (bolts)  
  \( \Omega = 2.00 \) (tabular nominal strength)
- Shear (welds)  
  \( \Omega = 2.00 \)

- \( L_b \) is the unbraced length between bracing points, laterally
- \( L_p \) is the limiting laterally unbraced length for the limit state of yielding
- \( L_r \) is the limiting laterally unbraced length for the limit state of inelastic lateral-torsional buckling

#### LRFD

\[
R_u \leq \phi R_n \quad \text{where} \quad R_u = \Sigma \gamma_i R_i
\]

where  
- \( \phi \) = resistance factor  
- \( \gamma \) = load factor for the type of load  
- \( R \) = load (dead or live; force, moment or stress)  
- \( R_u \) = factored load (moment or stress)  
- \( R_n \) = nominal load (ultimate capacity; force, moment or stress)

*Nominal strength* is defined as the capacity of a structure or component to resist the effects of loads, as determined by computations using specified material strengths (such as yield strength, \( F_y \), or ultimate strength, \( F_u \)) and dimensions and formulas derived from accepted principles of structural mechanics or by field tests or laboratory tests of scaled models, allowing for modeling effects and differences between laboratory and field conditions.
Factored Load Combinations

The design strength, $\phi R_n$, of each structural element or structural assembly must equal or exceed the design strength based on the ASCE-7 (2010) combinations of factored nominal loads:

\[
\begin{align*}
1.4D \\
1.2D + 1.6L + 0.5(L_r \text{ or } S \text{ or } R) \\
1.2D + 1.6(L_r \text{ or } S \text{ or } R) + (L \text{ or } 0.5W) \\
1.2D + 1.0W + L + 0.5(L_r \text{ or } S \text{ or } R) \\
1.2D + 1.0E + L + 0.2S \\
0.9D + 1.0W \\
0.9D + 1.0E
\end{align*}
\]

Criteria for Design of Beams

Allowable normal stress or normal stress from LRFD should not be exceeded:

\[
F_b \text{ or } \phi F_n \geq f_b = \frac{Mc}{I}
\]

Knowing $M$ and $F_n$, the minimum plastic section modulus fitting the limit is:

\[
Z_{reqd} \geq \frac{M_u}{F_n \Omega} \quad \left( S_{reqd} \geq \frac{M}{F_b} \right)
\]

Determining Maximum Bending Moment

Drawing $V$ and $M$ diagrams will show us the maximum values for design. Remember:

\[
\begin{align*}
V &= \Sigma (-w)dx \\
M &= \Sigma (V)dx \\
dV &= -w \\
dM &= V
\end{align*}
\]

Determining Maximum Bending Stress

For a prismatic member (constant cross section), the maximum normal stress will occur at the maximum moment.

For a non-prismatic member, the stress varies with the cross section AND the moment.

Deflections

If the bending moment changes, $M(x)$ across a beam of constant material and cross section then the curvature will change:

\[
\frac{1}{R} = \frac{M(x)}{EI}
\]

The slope of the n.a. of a beam, $\theta$, will be tangent to the radius of curvature, $R$:

\[
\theta = \text{slope} = \frac{1}{EI} \int M(x)dx
\]

The equation for deflection, $y$, along a beam is:

\[
y = \frac{1}{EI} \int \theta dx = \frac{1}{EI} \int \int M(x)dx
\]
Elastic curve equations can be found in handbooks, textbooks, design manuals, etc... Computer programs can be used as well. Elastic curve equations can be superimposed ONLY if the stresses are in the elastic range.

*The deflected shape is roughly the same shape flipped as the bending moment diagram but is constrained by supports and geometry.*

**Allowable Deflection Limits**

All building codes and design codes limit deflection for beam types and damage that could happen based on service condition and severity.

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

<table>
<thead>
<tr>
<th>Use</th>
<th>LL only</th>
<th>DL+LL*</th>
</tr>
</thead>
<tbody>
<tr>
<td>Roof beams:</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Industrial (no ceiling)</td>
<td>L/180</td>
<td>L/120</td>
</tr>
<tr>
<td>Commercial</td>
<td></td>
<td></td>
</tr>
<tr>
<td>plaster ceiling</td>
<td>L/240</td>
<td>L/180</td>
</tr>
<tr>
<td>no plaster</td>
<td>L/360</td>
<td>L/240</td>
</tr>
<tr>
<td>Floor beams:</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Ordinary Usage</td>
<td>L/360</td>
<td>L/240</td>
</tr>
<tr>
<td>Roof or floor (damageable elements)</td>
<td>L/480</td>
<td></td>
</tr>
</tbody>
</table>

* IBC 2012 states that DL for steel elements shall be taken as zero

**Lateral Buckling**

With compression stresses in the top of a beam, a sudden “popping” or buckling can happen even at low stresses. In order to prevent it, we need to brace it along the top, or laterally brace it, or provide a bigger \( I_y \).

**Local Buckling in Steel Wide-flange Beams– Web Crippling or Flange Buckling**

Concentrated forces on a steel beam can cause the web to buckle (called **web crippling**). Web stiffeners under the beam loads and bearing plates at the supports reduce that tendency. Web stiffeners also prevent the web from shearing in plate girders.
The maximum support load and interior load can be determined from:

\[ P_{n_{(max-end)}} = (2.5k + N)F_{yw}t_w \]
\[ P_{n_{(interior)}} = (5k + N)F_{yw}t_w \]

where \( t_w \) = thickness of the web
\( F_{yw} \) = yield strength of the web
\( N \) = bearing length
\( k \) = dimension to fillet found in beam section tables

\( \phi = 1.00 \) (LRFD) \( \Omega = 1.50 \) (ASD)

**Beam Loads & Load Tracing**

In order to determine the loads on a beam (or girder, joist, column, frame, foundation…) we can start at the top of a structure and determine the **tributary area** that a load acts over and the beam needs to support. Loads come from material weights, people, and the environment. This area is assumed to be from half the distance to the next beam over to halfway to the next beam.

The reactions must be supported by the next lower structural element *ad infinitum*, to the ground.

**LRFD - Bending or Flexure**

For determining the flexural design strength, \( \phi_b M_n \), for resistance to pure bending (no axial load) in most flexural members where the following conditions exist, a single calculation will suffice:

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

where \( M_n \) = maximum moment from factored loads
\( \phi_b \) = resistance factor for bending = 0.9
\( M_n \) = nominal moment (ultimate capacity)
\( F_y \) = yield strength of the steel
\( Z \) = plastic section modulus

**Plastic Section Modulus**

Plastic behavior is characterized by a yield point and an increase in strain with no increase in stress.
Internal Moments and Plastic Hinges

Plastic hinges can develop when all of the material in a cross section sees the yield stress. Because all the material at that section can strain without any additional load, the member segments on either side of the hinge can rotate, possibly causing instability.

For a rectangular section:

Elastic to $f_y$: \[ M_y = \frac{I}{c} f_y = \frac{bh^2}{6} f_y = \frac{b(2c)^2}{6} f_y = \frac{2bc^2}{3} f_y \]

Fully Plastic: \[ M_{ult} or M_p = bc^2 f_y = \frac{3}{2} M_y \]

For a non-rectangular section and internal equilibrium at $\sigma_y$, the n.a. will not necessarily be at the centroid. The n.a. occurs where the $A_{tension} = A_{compression}$. The reactions occur at the centroids of the tension and compression areas.

Instability from Plastic Hinges

Shape Factor:

The ratio of the plastic moment to the elastic moment at yield:

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

- $k = 3/2$ for a rectangle
- $k = 1.1$ for an I beam

Plastic Section Modulus

\[ Z = \frac{M_p}{f_y} \quad and \quad k = Z/S \]
Design for Shear

\[ V_u \leq V_n / \Omega \text{ or } V_u \leq \phi V_n \]

The nominal shear strength is dependent on the cross section shape. Case 1: With a thick or stiff web, the shear stress is resisted by the web of a wide flange shape (with the exception of a handful of W's). Case 2: When the web is not stiff for doubly symmetric shapes, singly symmetric shapes (like channels) (excluding round high strength steel shapes), inelastic web buckling occurs. When the web is very slender, elastic web buckling occurs, reducing the capacity even more:

Case 1) For \( h/t_w \leq 2.24 \frac{E}{F_y} \)

\[ V_n = 0.6F_{yw}A_w \quad \phi_v = 1.00 \text{ (LRFD)} \quad \Omega = 1.50 \text{ (ASD)} \]

where \( h \) equals the clear distance between flanges less the fillet or corner radius for rolled shapes

\( V_n = \) nominal shear strength

\( F_{yw} = \) yield strength of the steel in the web

\( A_w = t_wd = \) area of the web

Case 2) For \( h/t_w > 2.24 \frac{E}{F_y} \)

\[ V_n = 0.6F_{yw}A_vC_v \quad \phi_v = 0.9 \text{ (LRFD)} \quad \Omega = 1.67 \text{ (ASD)} \]

where \( C_v \) is a reduction factor (1.0 or less by equation)

Design for Flexure

\[ M_a \leq M_n / \Omega \text{ or } M_a \leq \phi_b M_n \quad \phi_b = 0.90 \text{ (LRFD)} \quad \Omega = 1.67 \text{ (ASD)} \]

The nominal flexural strength \( M_n \) is the lowest value obtained according to the limit states of

1. yielding, limited at length \( L_p = 1.76r_y \frac{E}{F_y} \), where \( r_y \) is the radius of gyration in \( y \)

2. lateral-torsional buckling limited at length \( L_t \)

3. flange local buckling

4. web local buckling

Beam design charts show available moment, \( M_n/\Omega \) and \( \phi_b M_n \), for unbraced length, \( L_{br} \) of the compression flange in one-foot increments from 1 to 50 ft. for values of the bending coefficient \( C_b = 1 \). For values of \( 1 < C_b \leq 2.3 \), the required flexural strength \( M_u \) can be reduced by dividing it by \( C_b \). \( (C_b = 1 \text{ when the bending moment at any point within an unbraced length is larger than that at both ends of the length. } C_b \text{ of 1 is conservative and permitted to be used in any case. When the free end is unbraced in a cantilever or overhang, } C_b = 1 \). The full formula is provided below.

\textbf{NOTE:} the self weight is not included in determination of \( M_n/\Omega \) and \( \phi_b M_n \)
Compact Sections

For a laterally braced compact section (one for which the plastic moment can be reached before local buckling) only the limit state of yielding is applicable. For unbraced compact beams and non-compact tees and double angles, only the limit states of yielding and lateral-torsional buckling are applicable.

Compact sections meet the following criteria:

\[
\frac{b_f}{2t_f} \leq 0.38 \sqrt{\frac{E}{F_y}} \quad \text{and} \quad \frac{h_c}{t_w} \leq 3.76 \sqrt{\frac{E}{F_y}}
\]

where:

- \(b_f\) = flange width in inches
- \(t_f\) = flange thickness in inches
- \(E\) = modulus of elasticity in ksi
- \(F_y\) = minimum yield stress in ksi
- \(h_c\) = height of the web in inches
- \(t_w\) = web thickness in inches

With lateral-torsional buckling the nominal flexural strength is

\[
M_n = C_b \left[ M_p - \left( M_p - 0.7F_y S_x \left( \frac{L_p - L_p}{L_q - L_p} \right) \right) \right] \leq M_p
\]

where \(M_p = M_n = F_y Z_x\)

and \(C_b\) is a modification factor for non-uniform moment diagrams where, when both ends of the beam segment are braced:

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

\(M_{\text{max}}\) = absolute value of the maximum moment in the unbraced beam segment
\(M_A\) = absolute value of the moment at the quarter point of the unbraced beam segment
\(M_B\) = absolute value of the moment at the center point of the unbraced beam segment
\(M_C\) = absolute value of the moment at the three quarter point of the unbraced beam segment length.

Available Flexural Strength Plots

Plots of the available moment for the unbraced length for wide flange sections are useful to find sections to satisfy the design criteria of \(M_a \leq M_n / \Omega\) or \(M_a \leq \phi_p M_n\). The maximum moment that can be applied on a beam (taking self weight into account), \(M_a\) or \(M_u\), can be plotted against the unbraced length, \(L_b\). The limiting length, \(L_p\) (fully plastic), is indicated by a solid dot (●), while the limiting length, \(L_q\) (for lateral torsional buckling), is indicated by an open dot (○). Solid lines indicate the most economical, while dashed lines indicate there is a lighter section that could be used. \(C_b\), which is a lateral torsional buckling modification factor for non-zero moments at the ends, is 1 for simply supported beams (0 moments at the ends). (see figure)
Design Procedure

The intent is to find the most lightweight member (which is economical) satisfying the section modulus size.

1. Determine the unbraced length to choose the limit state (yielding, lateral torsional buckling or more extreme) and the factor of safety and limiting moments. Determine the material.

2. Draw V & M, finding $V_{\text{max}}$ and $M_{\text{max}}$ for unfactored loads (ASD, $V_\text{u}$ & $M_\text{u}$) or from factored loads (LRFD, $V_\text{u}$ & $M_\text{u}$).

3. Calculate $Z_{\text{req'd}}$ when yielding is the limit state. This step is equivalent to determining if

$$f_b = \frac{M_{\text{max}}}{S} \leq F_b, \quad Z_{\text{req'd}} \geq \frac{M_{\text{max}}}{F_y} \quad \Omega$$

and

$$Z_{\text{req'd}} \geq \frac{M_n}{\phi M_n}$$

to meet the design criteria that

$$M_n \leq M_n / \Omega \quad \text{or} \quad M_n \leq \phi M_n$$

If the limit state is something other than yielding, determine the nominal moment, $M_n$, or use plots of available moment to unbraced length, $L_b$.

4. For steel: use the section charts to find a trial $Z$ and remember that the beam self weight (the second number in the section designation) will increase $Z_{\text{req'd}}$. The design charts show the lightest section within a grouping of similar $Z$'s.

**** Determine the “updated” $V_{\text{max}}$ and $M_{\text{max}}$ including the beam self weight, and verify that the updated $Z_{\text{req'd}}$ has been met. *****
5. Consider lateral stability.

6. Evaluate horizontal shear using $V_{\text{max}}$. This step is equivalent to determining if $f_v \leq F_v$ is satisfied to meet the design criteria that $V_a \leq V_n / \Omega$ or $V_a \leq \varphi V_n$

For I beams: 
$$f_{v_{\text{max}}} = \frac{3V}{2A} = \frac{V}{A_{\text{web}}} = \frac{V}{t_w d}$$

$$V_n = 0.6 F_{yw} A_w \text{ or } V_n = 0.6 F_{yw} A_v C_v$$

Others: 
$$f_{v_{\text{max}}} = \frac{VQ}{Ib}$$

7. Provide adequate bearing area at supports. This step is equivalent to determining if $f_p = \frac{P}{A} \leq F_p$ is satisfied to meet the design criteria that $P_a \leq P_n / \Omega$ or $P_a \leq \varphi P_n$

8. Evaluate shear due to torsion 
$$f_v = \frac{T_P}{J} \text{ or } \frac{T}{c_i d^2} \leq F_v \text{ (circular section or rectangular)}$$

9. Evaluate the deflection to determine if $\Delta_{\text{maxLL}} \leq \Delta_{\text{LL-allowed}}$ and/or $\Delta_{\text{maxTotal}} \leq \Delta_{\text{totalallowed}}$

**** note: when $\Delta_{\text{calculated}} > \Delta_{\text{limit}}, I_{\text{req'd}} \text{ can be found with:}$

and $Z_{\text{req'd}} \text{ will be satisfied for similar self weight} \text{ *****}$

$$I_{\text{req'd}} \geq \frac{A_{\text{low}} \Delta_{\text{trial}}}{\Delta_{\text{limit}}}$$

FOR ANY EVALUATION:

Redesign (with a new section) at any point that a stress or serviceability criteria is NOT satisfied and re-evaluate each condition until it is satisfactory.

Load Tables for Uniformly Loaded Joists & Beams

Tables exist for the common loading situation of uniformly distributed load. The tables provide the safe distributed loads based on bending and deflection limits. For specific clear spans, they provide maximum total loads and live loads for a specific deflection limits.

If the load is not uniform, an equivalent uniform load can be calculated from the maximum moment equation:

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

If the deflection limit needed is not that of the table, the design live load must be adjusted. For example:

$$w_{\text{adjusted}} = w_{\text{it-have}} \left( \frac{L / 360}{L / 400} \right) \text{ table limit}$$

Criteria for Design of Columns

If we know the loads, we can select a section that is adequate for strength & buckling.

If we know the length, we can find the limiting load satisfying strength & buckling.
Allowable Stress Design

American Institute of Steel Construction (AISC) Manual of ASD, 9th ed:

Long and slender: \[ \frac{L}{r} \geq C_c, \text{ preferably } < 200 \]

\[
F_{\text{allowable}} = \frac{F_{cr}}{F.S.} = \frac{12\pi^2 E}{23\left(\frac{KL}{r}\right)^2}
\]

The yield limit is idealized into a parabolic curve that blends into the Euler’s Formula at \( C_c \).

With \( F_y = 36 \text{ ksi} \), \( C_c = 126.1 \)
With \( F_y = 50 \text{ ksi} \), \( C_c = 107.0 \)

Short and stubby: \[ \frac{L}{r} < C_c \]

\[
F_a = \left[ 1 - \frac{\left(\frac{KL}{r}\right)^2}{2C_c^2} \right] \frac{F_y}{F.S.}
\]

with:

\[
F.S. = \frac{5}{3} + \frac{3\left(\frac{KL}{r}\right)}{8C_c} - \frac{\left(\frac{KL}{r}\right)^3}{8C_c^3}
\]

Design for Compression

American Institute of Steel Construction (AISC) Manual 14th ed:

\[
P_a \leq P_n / \Omega \text{ or } P_a \leq \phi P_n \quad \text{where } P_n = \sum \gamma_i P_i
\]

\( \gamma \) is a load factor

\( P \) is a load type

\( \phi \) is a resistance factor

\( P_n \) is the nominal load capacity (strength)

\( \phi = 0.90 \text{ (LRFD)} \quad \Omega = 1.67 \text{ (ASD)} \)

For compression \( P_n = F_{cr} A_g \)

where : \( A_g \) is the cross section area and \( F_{cr} \) is the flexural buckling stress
The flexural buckling stress, $F_{cr}$, is determined as follows:

when $\frac{KL}{r} \leq 4.71 \sqrt{\frac{E}{F_y}}$ or ($F_e \geq 0.44F_y$):

$$F_{cr} = \left[ 0.658 \frac{F_e}{F_y} \right] F_y$$

when $\frac{KL}{r} > 4.71 \sqrt{\frac{E}{F_y}}$ or ($F_e < 0.44F_y$):

$$F_{cr} = 0.877F_e$$

where $F_e$ is the elastic critical buckling stress:

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

**Design Aids**

Tables exist for the value of the flexural buckling stress based on slenderness ratio. In addition, tables are provided in the AISC Manual for Available Strength in Axial Compression based on the effective length with respect to least radius of gyration, $r_y$. If the critical effective length is about the largest radius of gyration, $r_x$, it can be turned into an effective length about the $y$ axis by dividing by the fraction $r_x/r_y$. 

*Sample AISC Table for Available Strength in Axial Compression*
**Procedure for Analysis**

1. Calculate KL/r for each axis (if necessary). The largest will govern the buckling load.
2. Find $F_a$ or $F_{cr}$ as a function of KL/r from the appropriate equation (above) or table.
3. Compute $P_{allowable} = F_a \cdot A$ or $P_n = F_{cr} \cdot A_g$
   
   *or alternatively* compute $f_c = P/A$ or $P_u/A$
4. Is the design satisfactory?
   
   - Is $P \leq P_{allowable}$ (or $P_a \leq P_n/\Omega$) or $P_u \leq \phi_c P_n$? ⇒ yes, it is; no, it is no good
   
   - or Is $f_c \leq F_a$ (or $\leq F_{cr}/\Omega$) or $\phi_c F_{cr}$? ⇒ yes, it is; no, it is no good

**Procedure for Design**

1. Guess a size by picking a section.
2. Calculate KL/r for each axis (if necessary). The largest will govern the buckling load.
3. Find $F_a$ or $F_{cr}$ as a function of KL/r from appropriate equation (above) or table.
4. Compute $P_{allowable} = F_a \cdot A$ or $P_n = F_{cr} \cdot A_g$
   
   *or alternatively* compute $f_c = P/A$ or $P_u/A$
5. Is the design satisfactory?
   
   - Is $P \leq P_{allowable}$ (or $P_a \leq P_n/\Omega$) or $P_u \leq \phi_c P_n$? yes, it is; no, pick a bigger section and go back to step 2.
   
   - Is $f_c \leq F_a$ (or $\leq F_{cr}/\Omega$) or $\phi_c F_{cr}$? yes, it is; no, pick a bigger section and go back to step 2.
6. Check design efficiency by calculating percentage of stress used:

   $$\frac{P}{P_{allowable}} \cdot 100\% \left( \frac{P_a}{P_n} \cdot 100\% \right) \text{ or } \frac{P_u}{\phi_c P_n} \cdot 100\%$$

   If value is between 90-100%, it is efficient.
   
   If values is less than 90%, *pick a smaller section and go back to step 2*.

**Columns with Bending (Beam-Columns)**

In order to *design* an adequate section for allowable stress, we have to start somewhere:

1. Make assumptions about the limiting stress from:
   
   - buckling
   - axial stress
   - combined stress
2. See if we can find values for $r$ or $A$ or $Z$.
3. Pick a trial section based on if we think $r$ or $A$ is going to govern the section size.
4. Analyze the stresses and compare to allowable using the allowable stress method or interaction formula for eccentric columns.

5. Did the section pass the stress test?
   - If not, do you increase \( r \) or \( A \) or \( Z \)?
   - If so, is the difference really big so that you could decrease \( r \) or \( A \) or \( Z \) to make it more efficient (economical)?

6. Change the section choice and go back to step 4. Repeat until the section meets the stress criteria.

**Design for Combined Compression and Flexure:**

The interaction of compression and bending are included in the form for two conditions based on the size of the required axial force to the available axial strength. This is notated as \( P_r \) (either \( P \) from ASD or \( P_u \) from LRFD) for the axial force being supported, and \( P_c \) (either \( P_n/\Omega \) for ASD or \( \phi P_n \) for LRFD). The increased bending moment due to the P-Δ effect must be determined and used as the moment to resist.

\[
\begin{align*}
\text{For } \frac{P_r}{P_c} \geq 0.2 : & \quad \frac{P}{P_n/\Omega} + \frac{8}{9} \left( \frac{M_x}{M_{nx}/\Omega} + \frac{M_y}{M_{ny}/\Omega} \right) \leq 1.0 \\
\text{ (ASD)} & \quad \frac{P_u}{\phi P_n} + \frac{8}{9} \left( \frac{M_{ux}}{\phi_b M_{nx}} + \frac{M_{uy}}{\phi_b M_{ny}} \right) \leq 1.0 \\
\text{ (LRFD)}
\end{align*}
\]

\[
\begin{align*}
\text{For } \frac{P_r}{P_c} < 0.2 : & \quad \frac{P}{2P_n/\Omega} + \left( \frac{M_x}{M_{nx}/\Omega} + \frac{M_y}{M_{ny}/\Omega} \right) \leq 1.0 \\
\text{ (ASD)} & \quad \frac{P_u}{2\phi P_n} + \left( \frac{M_{ux}}{\phi_b M_{nx}} + \frac{M_{uy}}{\phi_b M_{ny}} \right) \leq 1.0 \\
\text{ (LRFD)}
\end{align*}
\]

where:

- for compression \( \phi_c = 0.90 \) (LRFD) \( \Omega = 1.67 \) (ASD)
- for bending \( \phi_b = 0.90 \) (LRFD) \( \Omega = 1.67 \) (ASD)

For a braced condition, the moment magnification factor \( B_1 \) is determined by \( B_1 = \frac{C_m}{1 - \alpha (P_u/P_{el})} \geq 1.0 \)

where \( C_m \) is a modification factor accounting for end conditions

When not subject to transverse loading between supports in plane of bending:

\( = 0.6 - 0.4 \) \( (M_1/M_2) \) where \( M_1 \) and \( M_2 \) are the end moments and \( M_1 < M_2 \). \( M_1/M_2 \) is positive when the member is bent in reverse curvature (moments the same direction), negative when bent in single curvature.

When there is transverse loading between the two ends of a member:

\( = 0.85 \), members with restrained (fixed) ends

\( = 1.00 \), members with unrestrained ends

\( \alpha = 1.00 \) (LRFD), 1.60 (ASD)

\( P_{el} = \frac{\pi^2 EA}{(Kl/r)^2} \)

\( P_{el} \) = Euler buckling strength
Criteria for Design of Connections

Connections must be able to transfer any axial force, shear, or moment from member to member or from beam to column.

Connections for steel are typically high strength bolts and electric arc welds. Recommended practice for ease of construction is to specified shop welding and field bolting.

Bolted and Welded Connections

The limit state for connections depends on the loads:

1. tension yielding
2. shear yielding
3. bearing yielding
4. bending yielding due to eccentric loads
5. rupture

Welds must resist shear stress. The design strengths depend on the weld materials.

Bolted Connection Design

Bolt designations signify material and type of connection where
SC: slip critical
N: bearing-type connection with bolt threads included in shear plane
X: bearing-type connection with bolt threads excluded from shear plane

A307: similar in strength to A36 steel (also known as ordinary, common or unfinished bolts)
A325: high strength bolts (Group A)
A490: high strength bolts (higher than A325) (Group B)
Bearing-type connection: no frictional resistance in the contact surfaces is assumed and slip between members occurs as the load is applied. (Load transfer through bolt only).

Slip-critical connections: bolts are torqued to a high tensile stress in the shank, resulting in a clamping force on the connected parts. (Shear resisted by clamping force). Requires inspections and is useful for structures seeing dynamic or fatigue loading. Class A indicates the faying (contact) surfaces are clean mill scale or adequate paint system, while Class B indicates blast cleaning or paint for $\mu = 0.50$.

Bolts rarely fail in bearing. The material with the hole will more likely yield first.

For the determination of the net area of a bolt hole the width is taken as $1/16"$ greater than the nominal dimension of the hole. Standard diameters for bolt holes are $1/16"$ larger than the bolt diameter. (This means the net width will be $1/8"$ larger than the bolt.)

**Design for Bolts in Bearing, Shear and Tension**

Available shear values are given by bolt type, diameter, and loading (Single or Double shear) in AISC manual tables. Available shear value for slip-critical connections are given for limit states of serviceability or strength by bolt type, hole type (standard, short-slotted, long-slotted or oversized), diameter, and loading. Available tension values are given by bolt type and diameter in AISC manual tables.

Available bearing force values are given by bolt diameter, ultimate tensile strength, $F_u$, of the connected part, and thickness of the connected part in AISC manual tables.

**For shear OR tension (same equation) in bolts:**

$$ R_u \leq R_n / \Omega \text{ or } R_u \leq \phi R_n $$

where $R_u = \Sigma \gamma_i R_i$

- single shear (or tension) $R = F_u A_b$
- double shear $R = F_n 2A_b$

where $\phi =$ the resistance factor
$F_n =$ the nominal tension or shear strength of the bolt
$A_b =$ the cross section area of the bolt
$\phi = 0.75 \text{ (LRFD)} \quad \Omega = 2.00 \text{ (ASD)}$

**For bearing of plate material at bolt holes:**

$$ R_u \leq R_n / \Omega \text{ or } R_u \leq \phi R_n $$

where $R_u = \Sigma \gamma_i R_i$

- deformation at bolt hole is a concern
  $$ R_n = 1.2L_t F_u \leq 2.4dtF_u $$
- deformation at bolt hole is not a concern
  $$ R_n = 1.5L_t F_u \leq 3.0dtF_u $$
- long slotted holes with the slot perpendicular to the load
  $$ R_n = 1.0L_t F_u \leq 2.0dtF_u $$

---

**Figure 10.11** End view-exit.
where \( R_n \) = the nominal bearing strength
\( F_u \) = specified minimum tensile strength
\( L_c \) = clear distance between the edges of the hole and the next hole or edge in the direction of the load
\( d \) = nominal bolt diameter
\( t \) = thickness of connected material

\[ \phi = 0.75 \text{ (LRFD)} \quad \Omega = 2.00 \text{ (ASD)} \]

The **minimum** edge distance from the center of the outer most bolt to the edge of a member is generally \( \frac{3}{4} \) times the bolt diameter for the sheared edge and \( \frac{1}{4} \) times the bolt diameter for the rolled or gas cut edges.

The **maximum** edge distance should not exceed 12 times the thickness of thinner member or 6 in.

Standard bolt hole spacing is 3 in. with the minimum spacing of \( \frac{2}{3} \) times the diameter of the bolt, \( d_b \). Common edge distance from the center of last hole to the edge is 1¼ in..

**Tension Member Design**

In steel tension members, there may be bolt holes that reduce the size of the cross section.

\( g \) refers to the row spacing or *gage*
\( p \) refers to the bolt spacing or *pitch*
\( s \) refers to the longitudinal spacing of two consecutive holes

**Effective Net Area:**

The smallest effective area must be determined by subtracting the bolt hole areas. With staggered holes, the shortest length must be evaluated.

A series of bolts can also transfer a portion of the tensile force, and some of the effective net areas see reduced stress.

The effective net area, \( A_e \), is determined from the net area, \( A_n \), multiplied by a shear lag factor, \( U \), which depends on the element type and connection configuration. If a portion of a connected member is not fully connected (like the leg of an angle), the unconnected part is not subject to the full stress and the shear lag factor can range from 0.6 to 1.0:

\[ A_e = A_n U \]
The staggered hole path area is determined by:

\[ A_n = A_g - A_{all\ holes} + \Omega \frac{s^2}{4g} \]

where \( t \) is the plate thickness, \( s \) is each stagger spacing, and \( g \) is the gage spacing.

**For tension elements:**

\[ R_u \leq \frac{R_n}{\Omega} \quad \text{or} \quad R_u \leq \phi R_n \]

where \( R_n = \Sigma \gamma_i R_i \)

1. yielding

\[
\phi = 0.90 \quad \text{(LRFD)} \\
\Omega = 1.67 \quad \text{(ASD)}
\]

2. rupture

\[
\phi = 0.75 \quad \text{(LRFD)} \\
\Omega = 2.00 \quad \text{(ASD)}
\]

where \( A_g = \) the gross area of the member (excluding holes)  
\( A_e = \) the effective net area (with holes, etc.)  
\( F_y = \) the yield strength of the steel  
\( F_u = \) the tensile strength of the steel (ultimate)

**Welded Connections**

Weld designations include the strength in the name, i.e. E70XX has \( F_y = 70 \text{ ksi} \). Welds are weakest in shear and are assumed to always fail in the shear mode.

The throat size, \( T \), of a fillet weld is determined trigonometry by:

\[ T = 0.707 \times \text{weld size} \]

* When the submerged arc weld process is used, welds over 3/8” will have a throat thickness of 0.11 in. larger than the formula.

Weld sizes are limited by the size of the parts being put together and are given in AISC manual table J2.4 along with the allowable strength per length of fillet weld, referred to as \( S \).

The maximum size of a fillet weld:

a) can’t be greater than the material thickness if it is \( \frac{1}{4”} \) or less  
b) is permitted to be \( \frac{1}{16”} \) less than the thickness of the material if it is over \( \frac{1}{4”} \)
The minimum length of a fillet weld is 4 times the nominal size. If it is not, then the weld size used for design is ¼ the length.

Intermittent fillet welds cannot be less than four times the weld size, not to be less than 1 ½”.

For fillet welds: 
\[ R_u \leq R_n / \Omega \text{ or } R_u \leq \phi R_n \]
where 
\[ R_u = \Sigma \gamma_i R_i \]

for the weld metal: 
\[ R_n = 0.6F_{\text{exx}}Tl = Sl \]
\[ \phi = 0.75 \text{ (LRFD)} \quad \Omega = 2.00 \text{ (ASD)} \]

where:
- \( T \) is throat thickness
- \( l \) is length of the weld

For a connected part, the other limit states for the base metal, such as tension yield, tension rupture, shear yield, or shear rupture must be considered.

### Table J2.4

<table>
<thead>
<tr>
<th>Material Thickness of Thicker Part Joined (in.)</th>
<th>Minimum Size of Fillet Weld (in.)</th>
</tr>
</thead>
<tbody>
<tr>
<td>To ¼ inclusive</td>
<td>½</td>
</tr>
<tr>
<td>Over ¼ to ½</td>
<td>⅛</td>
</tr>
<tr>
<td>Over ½ to ¾</td>
<td>¼</td>
</tr>
<tr>
<td>Over ¾</td>
<td>⅛ to ⅛</td>
</tr>
</tbody>
</table>

*Leg dimension of fillet welds. Single-pass welds must be used.

American Institute of Steel Construction

<table>
<thead>
<tr>
<th>Available Strength of Fillet Welds per inch of weld (k/in)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Weld Size (in.)</td>
</tr>
<tr>
<td>⅛</td>
</tr>
<tr>
<td>¼</td>
</tr>
<tr>
<td>⅝</td>
</tr>
<tr>
<td>⅞</td>
</tr>
<tr>
<td>1</td>
</tr>
<tr>
<td>⅜</td>
</tr>
<tr>
<td>⅔</td>
</tr>
<tr>
<td>⅜</td>
</tr>
</tbody>
</table>

(not considering increase in throat with submerged arc weld process)

Framed Beam Connections

Coping is the term for cutting away part of the flange to connect a beam to another beam using welded or bolted angles.

AISC provides tables that give bolt and angle available strength knowing number of bolts, bolt type, bolt diameter, angle leg thickness, hole type and coping, and the wide flange beam being connected. For the connections the limit-state of bolt shear, bolts bearing on the angles, shear yielding of the angles, shear rupture of the angles, and block shear rupture of the angles, and bolt bearing on the beam web are considered.

Group A bolts include A325, while Group B includes A490.

There are also tables for bolted/welded double-angle connections and all-welded double-angle connections.
Sample AISC Table for Bolt and Angle Available Strength in All-Bolted Double-Angle Connections

Limiting Strength or Stability States

In addition to resisting shear and tension in bolts and shear in welds, the connected materials may be subjected to shear, bearing, tension, flexure and even prying action. Coping can significantly reduce design strengths and may require web reinforcement. All the following must be considered:

- shear yielding
- shear rupture
- block shear rupture - failure of a block at a beam as a result of shear and tension
- tension yielding
- tension rupture
- local web buckling
- lateral torsional buckling

Block Shear Strength (or Rupture):

\[ R_n = R_n^u / \Omega \quad \text{or} \quad R_n = \phi R_n^u \]

where \( R_n^u = \Sigma \gamma / R_i \)

\[ R_n = 0.6F_u A_{uv} + U_{fs} F_a A_{ml} \leq 0.6F_y A_{uv} + U_{fs} F_a A_{ml} \]

\[ \phi = 0.75 \quad \text{(LRFD)} \quad \Omega = 2.00 \quad \text{(ASD)} \]
where:
- \( A_{nv} \) is the net area subjected to shear
- \( A_{nt} \) is the net area subjected to tension
- \( A_{gv} \) is the gross area subjected to shear
- \( U_{bs} = 1.0 \) when the tensile stress is uniform (most cases)
  = 0.5 when the tensile stress is non-uniform

Gusset Plates
Gusset plates are used for truss member connections where the geometry prevents the members from coming together at the joint “point”. Members being joined are typically double angles.

Decking
Shaped, thin sheet-steel panels that span several joists or evenly spaced support behave as continuous beams. Design tables consider a “1 unit” wide strip across the supports and determine maximum bending moment and deflections in order to provide allowable loads depending on the depth of the material.

The other structural use of decking is to construct what is called a diaphragm, which is a horizontal unit tying the decking to the joists that resists forces parallel to the surface of the diaphragm.

When decking supports a concrete topping or floor, the steel-concrete construction is called composite.

Frame Columns
Because joints can rotate in frames, the effective length of the column in a frame is harder to determine. The stiffness \( (EI/L) \) of each member in a joint determines how rigid or flexible it is. To find \( k \), the relative stiffness, \( G \) or \( \Psi \), must be found for both ends, plotted on the alignment charts, and connected by a line for braced and unbraced frames.

\[
G = \Psi = \frac{\sum EI/\ell_c}{\sum EI/\ell_b}
\]

where
- \( E \) = modulus of elasticity for a member
- \( I \) = moment of inertia of for a member
- \( \ell_c \) = length of the column from center to center
- \( \ell_b \) = length of the beam from center to center

- For pinned connections we typically use a value of 10 for \( \Psi \).
- For fixed connections we typically use a value of 1 for \( \Psi \).
Braced – non-sway frame

Unbraced – sway frame

(a) Nonsway Frames

(b) Sway Frames
Example 1 (pg 330)

*Hypothetically determine the size of section required when the deflection criteria is NOT met

Example Problem 9.16 (Figures 9.76 to 9.78)

A steel beam (A572/50) is loaded as shown. Assuming a deflection requirement of $\Delta_{\text{total}} = L/240$ and a depth restriction of 18" nominal, select the most economical section. (unified ASD)

$F_y = 30$ ksi; $F_p = 20$ ksi; $E = 30 \times 10^3$ ksi $F_s = 50$ ksi

\[ F_y = 50 \text{ ksi} \]

\[ \omega = 1 \text{ k/ft} \]
Example 2

Given:

Select an ASTM A992 W-shape beam with a simple span of 35 feet. Limit the member to a maximum nominal depth of 18 in. Limit the live load deflection to \( L/360 \). The nominal loads are a uniform dead load of 0.45 kip/ft and a uniform live load of 0.75 kip/ft. Assume the beam is continuously braced. Use ASD of the Unified Design method.

Solution:

Material Properties:

<table>
<thead>
<tr>
<th>Property</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>ASTM A992</td>
<td>( F_y = 50 ) ksi</td>
</tr>
<tr>
<td>( F_n = 65 ) ksi</td>
<td></td>
</tr>
</tbody>
</table>

1. The unbraced length is 0 because it says it is fully braced.

2. Find the maximum shear and moment from unfactored loads: \( w_a = 0.450 \text{ k/ft} + 0.750 \text{ k/ft} = 1.20 \text{ k/ft} \)

\[
V_a = 1.20 \text{ k/ft}(35 \text{ ft})/2 = 21 \text{ k}
\]

\[
M_a = 1.20 \text{ k/ft}(35 \text{ ft})^2/8 = 184 \text{ k-ft}
\]

If \( M_a \leq M_n/\Omega \), the maximum moment for design is \( M_\Omega: M_{\text{max}} = 184 \text{ k-ft} \)

3. Find \( Z_{\text{reqd}} \):

\[
Z_{\text{reqd}} \geq M_{\text{max}}/F_y = 184 \text{ k-ft}(1.67)(12 \text{ in/ft})/50 \text{ ksi} = 73.75 \text{ in}^3
\]

\( F_y \) is the limit stress when fully braced)

4. Choose a trial section, and also limit the depth to 18 in as instructed:

W18 x 40 has a plastic section modulus of 78.4 in\(^3\) and is the most light weight (as indicated by the bold text) in Table 9.1

Include the self weight in the maximum values:

\[
w_{a-\text{adjusted}} = 1.20 \text{ k/ft} + 0.04 \text{ k/ft} \]

\[
V_{a-\text{adjusted}} = 1.24 \text{ k/ft}(35 \text{ ft})/2 = 21.7 \text{ k}
\]

\[
M_{a-\text{adjusted}} = 1.24 \text{ k/ft}(35 \text{ ft})^2/8 = 189.9 \text{ k-ft}
\]

\[
Z_{a-\text{reqd}} \geq 189.9 \text{ k-ft}(1.67)(12 \text{ in/ft})/50 \text{ ksi} = 76.11 \text{ in}^3
\]

And the Z we have (78.4) is larger than the Z we need (76.11), so OK.

6. Evaluate shear (is \( V_a \leq V_n/\Omega \)): \( A_w = dt_w \) so look up section properties for W18 x 40: \( d = 17.90 \text{ in} \) and \( t_w = 0.315 \text{ in} \)

\[
V_{n/\Omega} = 0.6F_yA_w/\Omega = 0.6(50 \text{ ksi})(17.90 \text{ in})(0.315 \text{ in})/1.5 = 112.8 \text{ k} \] which is much larger than 21.7 k, so OK.

7. Evaluate the deflection with respect to the limit stated of \( L/360 \) for the live load. (If we knew the total load limit we would check that as well). The moment of inertia for the W18 x 40 is needed. \( I_x = 612 \text{ in}^4 \)

\[
\Delta_{\text{live load limit}} = 35 \text{ ft}(12 \text{ in/ft})/360 = 1.17 \text{ in}
\]

\[
\Delta = 5wL^4/384EI = 5(0.75 \text{ k/ft})(35 \text{ ft})^4(12 \text{ in/ft})^3/384(29 \times 10^3 \text{ ksi})(612 \text{ in}^4) = 1.42 \text{ in}
\]

This is TOO BIG (not less than the limit.

Find the moment of inertia needed:

\[
l_{\text{reqd}} \geq 1.42 \text{ in}(612 \text{ in}^4)/(1.17 \text{ in}) = 742.8 \text{ in}^4
\]

From the Listing of W Shapes in Descending order of \( Z_x \) for Beam Design, a W21 x 44 is larger (by \( I_x = 843 \text{ in}^4 \)), and the most light weight, but it is too deep!. In the next group up, the W16 x 57 works (\( I_x = 758 \text{ in}^4 \), but we aren’t sure there is a more economical section. Then W18x50, W12x72, and W18x55, so we stop because the W18x50 has the smallest weight with the depth restriction. (In order: \( I_x = 800, 597, \) and \( 890 \text{ in}^4 \))

Choose a W18 x 50
Example 3
For the same beam and loading of Example 1, select the most economical beam using Load and Resistance Factor Design (LRFD) with the 18" depth restriction. Assume the distributed load is dead load, and the point load is live load.

\[ F_y = 50 \text{ ksi} \] \[ E = 30 \times 10^3 \text{ ksi} \]

\[ (1.6)20k = 32k \]
\[ (1.2)1k/ft = 1.2k/ft \]
\[ +32.8k \]
\[ -32.8k \]
\[ +16k \]
\[ -16k \]

1. To find \( V_{u\text{-max}} \) and \( M_{u\text{-max}} \), factor the loads, construct a new load diagram, shear diagram and bending moment diagram.

2. To satisfy \( M_u \leq \phi M_N \), we find \( M_u = \frac{M_N}{\phi} = \frac{341.6k-ft}{0.9} = 379.6k-ft \) and solve for \( Z \) needed:

\[ Z = \frac{M_u}{F_y} = \frac{379.6k-ft}{50kksi} = 91.9in^3 \]

Choose a trial section from the Listing of W Shapes in Descending Order of \( Z \) by selecting the bold section at the top of the grouping satisfying our \( Z \) and depth requirement – \( W_{16x50} \) is the lightest with \( Z = 95.4 \text{ in}^3 \).

(W21 x 44 is the lightest without the depth requirement.) Include the additional self weight (dead load) and find the maximum shear and bending moment:

\[ V_{u\text{-adjusted}} = 32.8k + \frac{1.2(50\text{ lb/ft})28\text{ ft}}{2(1000\text{ lb/ft})} = 33.64k \]
\[ M_{u\text{-adjusted}} = 341.6k-ft + \frac{1.2(50\text{ lb/ft})(28\text{ ft})^2}{8(1000\text{ lb/ft})} = 347.5k-ft \]
\[ Z_{\text{web}} \geq \frac{M_u}{\phi F_y} = \frac{347.5k-ft}{0.9(50kksi)} = 92.7in^3 \], so \( Z \) (have) of 95.4 \text{ in}^3 is greater than the \( Z \) (needed).

3. Check the shear capacity to satisfy \( V_u \leq \phi V_n \): \( A_{web} = dt_w \) and \( d = 17.99 \text{ in.}, t_w = 0.355 \text{ in.} \) for the W18x50

\[ \phi V_n = \phi_k 0.6F_{yw}A_w = 1.0(0.6)50ksi(16.26in)(0.380in) = 185.4k \]

So \( 33.64k \leq 185.4 \text{ k} \) \text{ OK}

4. Calculate the deflection from the unfactored loads, including the self-weight now because it is known, and satisfy the deflection criteria of \( \Delta_{\text{LL}} \leq \Delta_{\text{LL-limit}} \) and \( \Delta_{\text{total}} \leq \Delta_{\text{total-limit}} \). (This is identical to what is done in Example 1.) \( I_k = 800 \text{ in}^3 \) for the W18x50

\[ \Delta_{\text{total-limit}} = L/240 = 1.4 \text{ in.}, \text{ say } \Delta_{\text{LL}} = L/360 = 0.93 \text{ in} \]

\[ \Delta_{\text{total}} = \frac{PL^2}{48EI} + \frac{5WL^4}{384EI} = \frac{20k(28\text{ ft})^3(12\text{ in/ft})^3}{48(29x10^3\text{ ksi})659\text{in}^4} + \frac{5(1.050\text{ ksi})(28\text{ ft})^3(12\text{ in/ft})^3}{384(29x10^3\text{ ksi})659\text{in}^4} = 0.827 + 0.760 = 1.59\text{in} \]

But 1.59 in. \( \geq \, 1.4 \text{ in.}, \text{ while } 0.827 \text{ in.} \leq 0.93 \text{ in. NOT OK} \)

Choose a larger section: \( I_{\text{web}} \geq \frac{\Delta_{\text{total-big}}}{\Delta_{\text{trial}}} \)

\( I_{\text{trial}} = \frac{1.59\text{in}}{1.4\text{in}}(659\text{in}^4) = 748.4\text{in}^4 \)

Next most lightweight section in \( Z \) Listing is W18x50 with \( I = 800 \text{ in}^4 \). It has the same self weight, so there is no need to recalculate and check the strength in shear and bending. \( \therefore \) **FINAL SELECTION IS W18x50**
Example 4
A steel beam with a 20 ft span is designed to be simply supported at the ends on columns and to carry a floor system made with open-web steel joists at 4 ft on center. The joists span 28 feet and frame into the beam from one side only and have a self weight of 8.5 lb/ft. Use A992 (grade 50) steel and select the most economical wide-flange section for the beam with LRFD design. Floor loads are 50 psf LL and 14.5 psf DL.
Example 5
Select a A992 W shape flexural member ($F_y = 50$ ksi, $F_u = 65$ ksi) for a beam with distributed loads of 825 lb/ft (dead) and 1300 lb/ft (live) and a live point load at midspan of 3 k using the Available Moment tables. The beam is simply supported, 20 feet long, and braced at the ends and midpoint only ($L_b = 10$ ft.) The beam is a roof beam for an institution without plaster ceilings. (LRFD)

SOLUTION:

To use the Available Moment tables, the maximum moment required is plotted against the unbraced length. The first solid line with capacity or unbraced length above what is needed is the most economical.

DESIGN LOADS (load factors applied on figure):

$$M_x = \frac{4}{3} + \frac{3.07}{\beta} \left( \frac{40 \text{ ft}^2}{10} \right) + 4.8k(10 \text{ ft}) = 662 \text{ k-ft} \quad V_{un} = \frac{wL + P}{2} = 3.07 \frac{40 \text{ k-ft}}{20 \text{ ft}} + 4.8k = 66.2k$$

Plotting 662 k-ft vs. 10 ft lands just on the capacity of the W21x83, but it is dashed (and not the most economical) AND we need to consider the contribution of self weight to the total moment. Choose a trial section of W24 x 76. Include the new dead load:

$$M_x^{\text{adjusted}} = 662k + \frac{1.2(76\text{ k-ft})(20 \text{ ft})^2}{2(1000\text{ k-ft})} = 680.2k \quad V_{n}^{\text{adjusted}} = 66.2k + 1.2(0.076 \frac{k}{\beta})(20 \text{ ft}) = 68.0k$$

Replot 680.2 k-ft vs. 10 ft, which lands above the capacity of the W21x83. We can’t look up because the chart ends, but we can look for that capacity with a longer unbraced length. This leads us to a **W24 x 84** as the most economical. (With the additional self weight of 84 - 76 lb/ft = 8 lb/ft, the increase in the factored moment is only 1.92 k-ft; therefore, it is still OK.)

Evaluate the shear capacity:

$$\phi V_n = \phi V_{n,\text{min}} A_y = 1.0(0.6)50\text{ksi}(24.1\text{in})0.47\text{in} = 338.4k \quad \text{so yes, 68 k} \leq 338.4k \quad \text{OK}$$

Evaluate the deflection with respect to the limits of L/240 for live (unfactored) load and L/180 for total (unfactored) load:

$$\frac{Pb}{6EI} + \frac{wL^3}{24EI} \leq \frac{3k(10\text{ ft})^2(3\cdot20-10\text{ ft})(12\text{ in})^3}{6(30\times10^6\text{ k-ft}^3/\text{in}^3)} + \frac{(2.209)(20\text{ ft})^4(12\text{ in})^3}{24(30\times10^6\text{ k-ft}^3/\text{in}^3)} = 0.06 + 0.36 = 0.42\text{in}$$

So, $\Delta_{\text{all}}\leq\Delta_{\text{all-limit}}$ and $\Delta_{\text{total}}\leq\Delta_{\text{total-limit}}$

0.06 in. $\leq$ 1 in. and 0.42 in. $\leq$ 1.33 in.

(This section is so big to accommodate the large bending moment at the cantilever support that it deflects very little.)

∴ **FINAL SELECTION IS W24x84**

---

**Table 3-10**

<table>
<thead>
<tr>
<th>W Shapes</th>
<th>Available Moment vs. Unbraced Length</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
</tr>
</tbody>
</table>
Example 6

Select the most economical joist for the 40 ft grid structure with floors and a flat roof. The roof loads are 10 lb/ft² dead load and 20 lb/ft² live load. The floor loads are 30 lb/ft² dead load 100 lb/ft² live load. (Live load deflection limit for the roof is L/240, while the floor is L/360). Use the (LRFD) K and LH series charts provided.

(Top values are maximum total factored load in lb/ft, while the lower (lighter) values are maximum (unfactored) live load for a deflection of L/360)
Example 6 (continued)

Shaded areas indicate the bridging requirements.

### STANDARD LOADABLE TABLE FOR LONGSPAN STEEL JOISTS, LH-SERIES

<table>
<thead>
<tr>
<th>Joint Designation</th>
<th>Approx. Wt in Lbs/Per Linear Ft (Joists Only)</th>
<th>Depth in Inches</th>
<th>SAFE LOAD* in Lbs. Between</th>
<th>CLEAR SPAN IN FEET</th>
</tr>
</thead>
<tbody>
<tr>
<td>20LH02</td>
<td>10  20</td>
<td>16950</td>
<td>22.24</td>
<td>34  41  48  55  62</td>
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<tr>
<td>20LH03</td>
<td>11  20</td>
<td>18000</td>
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<td>12  20</td>
<td>22050</td>
<td>22.40</td>
<td>34  41  48  55  62</td>
</tr>
<tr>
<td>20LH05</td>
<td>14  20</td>
<td>23700</td>
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</tr>
<tr>
<td>20LH06</td>
<td>15  20</td>
<td>31650</td>
<td>22.59</td>
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<td>20LH07</td>
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<td>20LH09</td>
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<td>22.76</td>
<td>34  41  48  55  62</td>
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<td>20LH10</td>
<td>23  20</td>
<td>41100</td>
<td>22.88</td>
<td>34  41  48  55  62</td>
</tr>
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<td>24LH03</td>
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<td>17250</td>
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</tr>
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<td>24LH04</td>
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<td>34  41  48  55  62</td>
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<td>24LH05</td>
<td>13  24</td>
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</tr>
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<td>24LH06</td>
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<td>24LH12</td>
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<td>24LH13</td>
<td>21  24</td>
<td>58800</td>
<td>23.15</td>
<td>34  41  48  55  62</td>
</tr>
</tbody>
</table>

(Top values are maximum total factored load in lb/ft, while the lower (lighter) values are maximum (unfactored) live load for a deflection of L/360)
Example 7 (LRFD)

**EXAMPLE 5.1 Open-Web Steel Joist Design**

A fully exposed roof system for a commercial building, spanning 35 ft, located in Muncie, Indiana, in an urban environment.

IBC specifies a 20 psf snow live load for Muncie, Indiana, home of Ball State University. Table 1.3 indicates the snow exposure factor: $C_s = 0.9$. Table 1.4 indicates the snow thermal factor: $C_t = 1.0$. Table 1.7 indicates an occupancy importance factor (for Category II): $I_o = 1.0$. Fig. 1.2 indicates the ground snow load: $p_g = 20$ psf

\[ P_s = 0.7(0.9)(1.0)20 \text{ psf} = 13.9 \text{ psf} \]

**A typical roof construction might consist of:**

- Membrane roofing: 1.0 psf
- 4 in. average tapered rigid insulation: 6.0 psf
- Steel deck (2–4 ft span): 1.0 psf

Estimated joist weight:

- 35 ft span would be a minimum 18 in. joist
- An average 18 in. joist weight = 9.0 plf
- Spaced @ 4 ft-0 in. o.c. 9.0 plf/4 ft = 2.3 psf
- Ceiling suspension system: 1.0 psf
- 1/2 in. gypsum ceiling: 2.0 psf

Mechanical system estimates should also be included; the heavy sprinkler/drain piping running parallel to a joist or pair of joists is especially critical.

- Miscellaneous ductwork/electrical = 1.0 psf
- Total dead load: $14.3 \text{ psf} \times 4 \text{ ft o.c.} = 57.2 \text{ plf}$
- Total live load: $13.9 \text{ psf} \times 4 \text{ ft o.c.} = 55.6 \text{ plf}$
- Total factored live snow load + dead load = $1.2(55.6) + 1.6 (57.2) = 158.2 \text{ plf}$

Use joist load tables to select the best section:

- At 35 ft, 18K3 joists carry 223 plf TFL and 77 plf LL
- LL: deflection controls and the weight is 6.4 psf.

At least on the surface, this is the best choice, but depending upon the need to integrate mechanical systems into the joist space, a 20K3 at 6.5 plf or even a 22K4 at 7.3 plf which is both deeper and heavier than the previous selection may be best:

![Diagram of open-web steel joist design](image_url)

### STANDARD LOAD TABLE FOR OPEN WEB STEEL JOISTS, K-SERIES

Based on a 50 ksf Maximum Yield Strength - Loads Shown in Pounds Per Linear Foot (plf)

<table>
<thead>
<tr>
<th>Joist Designation</th>
<th>18K3</th>
<th>18K4</th>
<th>18K5</th>
<th>18K6</th>
<th>18K7</th>
<th>18K10</th>
<th>20K3</th>
<th>20K4</th>
<th>20K5</th>
<th>20K6</th>
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<th>22K4</th>
<th>22K5</th>
<th>22K9</th>
<th>22K10</th>
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<tbody>
<tr>
<td>Depth (in.)</td>
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<td>18</td>
<td>18</td>
<td>18</td>
<td>18</td>
<td>18</td>
<td>20</td>
<td>20</td>
<td>20</td>
<td>20</td>
<td>20</td>
<td>20</td>
<td>22</td>
<td>22</td>
<td>22</td>
<td>22</td>
<td>22</td>
<td>22</td>
</tr>
<tr>
<td>Approx. Wt. (lbs/ft)</td>
<td>6.4</td>
<td>7.2</td>
<td>7.7</td>
<td>8.4</td>
<td>8.9</td>
<td>10.1</td>
<td>11.6</td>
<td>6.5</td>
<td>7.2</td>
<td>7.7</td>
<td>8.4</td>
<td>8.9</td>
<td>10.1</td>
<td>11.6</td>
<td>7.3</td>
<td>7.7</td>
<td>8.5</td>
<td>9.0</td>
</tr>
<tr>
<td>Span (ft)</td>
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<td>34</td>
<td>34</td>
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<td>34</td>
<td>34</td>
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<td>210</td>
<td>137</td>
<td>163</td>
<td>186</td>
<td>219</td>
</tr>
</tbody>
</table>
Example 8

A floor with multiple bays is to be supported by open-web steel joists spaced at 3 ft. on center and spanning 30 ft. having a dead load of 70 lb/ft$^2$ and a live load of 100 lb/ft$^2$. The joists are supported on joist girders spanning 30 ft. with 3 ft.-long panel points (shown). Determine the member forces at the location shown in a horizontal chord and the maximum force in a web member for an interior girder. Use factored loads. Assume a self weight for the open-web joists of 12 lb/ft, and the self weight for the joist girder of 35 lb/ft.
Example 9
A floor is to be supported by trusses spaced at 5 ft. on center and spanning 60 ft. having a dead load of 53 lb/ft² and a live load of 100 lb/ft². With 3 ft.-long panel points, the depth is assumed to be 3 ft with a span-to-depth ratio of 20. With 6 ft.-long panel points, the depth is assumed to be 6 ft with a span-to-depth ratio of 10. Determine the maximum force in a horizontal chord and the maximum force in a web member. Use factored loads. Assume a self weight of 40 lb/ft².

<table>
<thead>
<tr>
<th>Table 7.2 Computation of Truss Joint Loads</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>area loads</strong></td>
</tr>
<tr>
<td>Truss</td>
</tr>
<tr>
<td></td>
</tr>
<tr>
<td>3 ft deep</td>
</tr>
<tr>
<td>6 ft deep</td>
</tr>
</tbody>
</table>

**FBD 1** for 3 ft deep truss

**FBD 2** of cut just to the left of midspan

**FBD 3** of cut just to right of left support

**FBD 4** for 6 ft deep truss

**FBD 5** of cut just to the left of midspan

**FBD 6** of cut just to right of left support

**FBD 1**: Maximum web force will be in the end diagonal (just like maximum shear in a beam)

\[ \Sigma F_y = 10P_1 - 0.5P_1 - F_{AB}\sin 45^\circ = 0 \]

\[ F_{AB} = 9.5P_1/\sin 45^\circ = 9.5(3.49 k)/0.707 = 46.9 k \]

**FBD 2**: Maximum chord force (top or bottom) will be at midspan

\[ \Sigma M_0 = -9.5P_1(27^\circ) + P_2(24^\circ) + P_2(21^\circ) + P_2(18^\circ) + P_2(15^\circ) \]

\[ + P_1(12^\circ) + P_1(9^\circ) + P_1(6^\circ) + P_1(3^\circ) + T_1(3^\circ) = 0 \]

\[ T_1 = P_1(148.5^\circ)/30 = (3.49 k)(49.5) = 172.8 k \]

\[ \Sigma F_y = 10P_1 - 9.5P_1 - D_1\sin 45^\circ = 0 \]

\[ D_1 = 0.5(3.49 k)/0.707 = 2.5 k \text{ (minimum near midspan)} \]

\[ \Sigma F_x = -C_1 + T_1 + D_1\cos 45^\circ = 0 \]

\[ C_1 = 174.5 k \]

**FBD 3**: Maximum web force will be in the end diagonal

\[ \Sigma F_y = 5P_2 - 0.5P_2 - F_{AB}\sin 45^\circ = 0 \]

\[ F_{AB} = 4.5P_2/\sin 45^\circ = 4.5(7 k)/0.707 = 44.5 k \]

**FBD 5**: Maximum chord (top or bottom) force will be at midspan

\[ \Sigma M_0 = -4.5P_2(24^\circ) + P_2(18^\circ) + P_2(12^\circ) + P_2(6^\circ) + T_2(6^\circ) = 0 \]

\[ T_2 = P_2(72^\circ)/60 = (7 k)(12) = 84 k \]

\[ \Sigma F_y = 5P_2 - 4.5P_2 - D_2\sin 45^\circ = 0 \]

\[ D_2 = 0.5(7 k)/0.707 = 4.9 k \text{ (minimum near midspan)} \]

\[ \Sigma F_x = -C_2 + T_2 + D_2\cos 45^\circ = 0 \]

\[ C_2 = 87.5 k \]
Example 10 (pg 367) + LRFD

Example Problem 10.10 (Figure 10.41)

A 24-ft.-tall, A572 grade 50, steel column (W14×82) with an $F_y = 50$ ksi has pins at both ends. Its weak axis is braced at midheight, but the column is free to buckle the full 24 ft. in the strong direction. Determine the safe load capacity for this column, using ASD and LRFD.

Figure 10.41  (a) Strong axis buckling.  
(b) Weak axis buckling.

Example 11 (pg 371) + chart method

Example Problem 10.14: Design of Steel Columns (Figure 10.48)

Select the most economical W12 × column 18' in height to support an axial load of 600 kips using A572 grade 50 steel. Assume that the column is hinged at the top but fixed at the base. Use LRFD assuming that the load is a dead load (factor of 1.4)

ALSO: Select the W12 column using the Available Strength charts.
Example 12

Given:

Redesign the column from Example E.1a assuming the column is laterally braced about the y-y axis and torsionally braced at the midpoint. Use both ASD and LRFD. $F_y = 50$ ksi.

(Not using Available Strength charts)

Solution:

ASD:

1. $P_a = 140\, k + 420\, k = 560\, k$

2. The effective length in the weak (y-y) axis is 15 ft, while the effective length in the strong (x-x) axis is 30 ft. ($K = 1, KL = 1 \times 30\, ft$).
   To find $kL/r_x$ and $kL/r_y$ we can assume or choose values from the wide flange charts. $r_y$'s range from 1 to 3 in., while $r_x$'s range from 3 to 14 inches. Let's try $r_y = 2$ in and $r_x = 9$ in. (something in the W21 range, say.)
   
   $kL/r_y \approx 15\, ft(12\, in/ft)/2\, in. = 90 \leftarrow \text{GOVERNS (is larger)}$
   
   $kL/r_x \approx 30\, ft(12\, in/ft)/9\, in. = 40$

3. Find a section with sufficient area (which then will give us “real” values for $r_x$ and $r_y$):
   
   If $P_a \leq P_n/\Omega$, and $P_n = F_{cr}A$, we can find $A \geq P_a\Omega/F_{cr}$ with $\Omega = 1.67$
   
   The tables provided have $\phi F_{cr}$, so we can get $F_{cr}$ by dividing by $\phi = 0.9$
   
   $\phi F_{cr}$ for 90 is 24.9 ksi, $F_{cr} = 24.9\, ksi/0.9 = 27.67\, ksi$ so $A \geq 560\, k(1.67)/27.67\, ksi = 33.8\, in^2$

4. Choose a trial section, and find the effective lengths and associated available strength, $F_{cr}$:
   
   Looking from the smallest sections, the W14's are the first with a big enough area:
   
   Try a W14 x 120 ($A = 35.3\, in^2$) with $r_y = 3.74$ in and $r_x = 6.24$ in.: $kL/r_y = 48.1$ and $kL/r_x = 57.7$ (GOVERNS)
   
   $\phi F_{cr}$ for 58 is 35.2 ksi, $F_{cr} = 39.1\, ksi$ so $A \geq 560\, k(1.67)/39.1\, ksi = 23.9\, in^2$
   
   Choose a W14 x 90 (Choosing a W14 x 82 would make $kL/r_x = 59.5$, and $A_{req'd} = 24.3\, in^2$, which is more than 24.1 in²)

LRFD:

1. $P_u = 1.2(140\, k) + 1.6(420\, k) = 840\, k$

2. The effective length in the weak (y-y) axis is 15 ft, while the effective length in the strong (x-x) axis is 30 ft. ($K = 1, KL = 1 \times 30\, ft$).
   To find $kL/r_x$ and $kL/r_y$ we can assume or choose values from the wide flange charts. $r_y$'s range from 1 to 3 in., while $r_x$'s range from 3 to 14 inches. Let's try $r_y = 2$ in and $r_x = 9$ in. (something in the W21 range, say.)
   
   $kL/r_y \approx 15\, ft(12\, in/ft)/2\, in. = 90 \leftarrow \text{GOVERNS (is larger)}$
   
   $kL/r_x \approx 30\, ft(12\, in/ft)/9\, in. = 40$

3. Find a section with sufficient area (which then will give us “real” values for $r_x$ and $r_y$):
   
   If $P_a \leq \phi P_n$, and $P_n = \phi F_{cr}A$, we can find $A \geq P_a/\phi F_{cr}$ with $\phi = 0.9$
   
   $\phi F_{cr}$ for 90 is 24.9 ksi, so $A \geq 840\, k/24.9\, ksi = 33.7\, in^2$

4. Choose a trial section, and find the effective lengths and associated available strength, $\phi F_{cr}$:
   
   Looking from the smallest sections, the W14's are the first with a big enough area:
   
   Try a W14 x 120 ($A = 35.3\, in^2$) with $r_y = 3.74$ in and $r_x = 6.24$ in.: $kL/r_y = 48.1$ and $kL/r_x = 57.7$ (GOVERNS)
   
   $\phi F_{cr}$ for 58 is 35.2 ksi, $F_{cr} = 39.1\, ksi$ so $A \geq 840\, k(1.67)/39.1\, ksi = 23.9\, in^2$
   
   Choose a W14 x 90 (Choosing a W14 x 82 would make $kL/r_x = 59.5$, and $A_{req'd} = 24.3\, in^2$, which is more than 24.1 in²)
Example 13

For the building frame shown in Fig. 6-20, determine the effective column length factor, $K$, the slenderness ratio, $KL/r$ for each column. Assume the columns buckle and the beams bend about their strong axis.

![Building frame diagram](image)

**Solution:**

The diagonal bracing prevents sideways of the first story columns only.

\[
G_A = 1.0 \text{ (fixed support)} \quad G_b = G_c = 10.0 \text{ (pinned support)}
\]

\[
G_n = \frac{285}{448} = 0.63 \quad G_k = \frac{285 + 204}{448 + 340} = 0.87
\]

\[
G_p = \frac{285}{15} = 1.91 \quad G_G = G_M = \frac{204}{340} = 0.90
\]

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<tr>
<th>Column</th>
<th>$G_{Top}$</th>
<th>$G_{Bot}$</th>
<th>$K$</th>
<th>$KL/r$</th>
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<td>AD</td>
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<td>1.0</td>
<td>0.76</td>
<td>Braced 0.76(15)(12)/5.25 = 26.1</td>
</tr>
<tr>
<td>BE</td>
<td>0.87</td>
<td>10.0</td>
<td>0.85</td>
<td>Braced 0.85(15)(12)/5.25 = 29.1</td>
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<tr>
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<td>10.0</td>
<td>0.90</td>
<td>Braced 0.90(15)(12)/5.25 = 30.9</td>
</tr>
<tr>
<td>EG</td>
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<td>0.87</td>
<td>1.29</td>
<td>Unbraced 1.29(12)(12)/5.17 = 35.9</td>
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<tr>
<td>FH</td>
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<td>1.91</td>
<td>1.43</td>
<td>Unbraced 1.43(12)(12)/5.17 = 39.8</td>
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</tbody>
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*Table 6-1: Column effective length factors and slenderness ratios for Example 6-1.*
Example 14
Investigate the acceptability of a W16 x 67 used as a beam-column under the unfactored loading shown in the figure. It is A992 steel ($F_y = 50$ ksi). Assume 25% of the load is dead load with 75% live load.

SOLUTION:

DESIGN LOADS (shown on figure):

Axial load = $1.2(0.25)(350k) + 1.6(0.75)(350k) = 525k$

Moment at joint = $1.2(0.25)(60k-ft) + 1.6(0.75)(60k-ft) = 90k-ft$

Determine column capacity and fraction to choose the appropriate interaction equation:

\[
\frac{kL}{r_e} = \frac{15f_b(12\pi^2)}{6.96 \text{in}} = 25.9 \quad \text{and} \quad \frac{kL}{r_e} = \frac{15f_b(12\pi^2)}{2.46 \text{in}} = 73 \quad (\text{governs})
\]

There is no bending about the $y$ axis, so that term will not have any values.

Determine the bending moment capacity in the $x$ direction:

The unbraced length to use the full plastic moment ($L_p$) is listed as 8.69 ft, and we are over that so of we don’t want to determine it from formula, we can find the beam in the Available Moment vs. Unbraced Length tables. The value of $\phi M_n$ at $L_b = 15$ ft is 422 k-ft.

Determine the magnification factor when $M_1 = 0$, $M_2 = 90$ k-ft:

\[
C_p = 0.6 - 0.4 \frac{M_1}{M_2} = 0.6 - \frac{90k}{600.85k} = 0.6 \leq 1.0
\]

\[
P_{pl} = \frac{\pi^2 EA}{(K/r)^2} = \frac{\pi^2 (30 \times 10^3 \text{ ksi}) 19.7 \text{ in}^2}{(25.9)^2} = 8,695.4k
\]

\[
B_i = \frac{C_p}{1 - \alpha (P_i/P_n)} = \frac{0.6}{1 - (1.0)(0.87)(525k/8695.4k)} = 0.64 \geq 1.0 \quad \text{This is NOT OK. (and outside error tolerance). The section should be larger.)}
\]

Finally, determine the interaction value:

\[
\frac{P_n}{\phi M_n} + \frac{8}{9} \left( \frac{M_{yy}}{\phi M_{yy}} + \frac{M_{xx}}{\phi M_{xx}} \right) = 0.87 + \frac{8}{9} \left( \frac{90k}{422k} \right) = 1.06 \leq 1.0
\]
Example 15

10.9 Determine the maximum load carrying capacity of this lap joint, assuming A36 steel with E60XX electrodes.

\[ \phi_S = 6.96 \text{ k/in} \]

\[ 6.96 \text{ k/in} = 153.1 \text{ k} \]

\[ \phi_P \frac{n}{A} = \phi_{Fy} A \]

\[ \phi = 0.9 \]

\[ 0.9 \times 36 \text{ k/in}^2 \times 3/8'' \times 6'' = 72.9 \text{ k} \]

\[ \phi_P = 72.9 \text{ k} \]

Example 16

10.7 Determine the capacity of the connection in Figure 10.44 assuming A36 steel with E70XX electrodes.

Solution:

Capacity of weld:

For a 5/16" fillet weld, \( \phi_S = 6.96 \text{ k/in} \)

Weld length = 8 in + 6 in + 8 in = 22 in.

Weld capacity = 22" \( \times \) 6.96 k/in = 153.1 k

Capacity of plate:

\[ \phi_P = \phi_{Fy} A \]

\[ \phi = 0.9 \]

Plate capacity = 0.9 \( \times \) 36 k/in\(^2\) \( \times \) 3/8" \( \times \) 6" = 72.9 k

\[ \therefore \text{Plate capacity governs, } \phi_P = 72.9 \text{ k} \]

The weld size used is obviously too strong. What size, then, can the weld be reduced to so that the weld strength is more compatible to the plate capacity? To make the weld capacity = plate capacity:

\[ 22'' \times (\text{weld capacity per in.}) = 72.9 \text{ k} \]

Weld capacity per inch = \( \frac{72.9 \text{ k}}{22 \text{ in.}} \) = 3.31 k/in.

From Available Strength table, use 3/16" weld \( (\phi_S = 4.18 \text{ k/in.}) \)

Minimum size fillet = 3/16" based on a 3/8" thick plate.
Example 17

10.5 Using the AISC framed beam connection bolt shear in Table 7-1, determine the shear adequacy of the connection shown in Figure 10.28. What thickness and angle length are required? Also determine the bearing capacity of the wide flange sections.

Factored end beam reaction = 90 k.

Example 18

Verify the tensile strength of an L4 x 4 x ½, ASTM 36, with one line of (4) ½ in.-diameter bolts and standard holes. The member carries a dead load of 20 kips and a live load of 60 kips in tension. Assume that connection limit states do not govern, and $U = 0.869$. 
Example 19

The steel used in the connection and beams is A992 with \( F_y = 50 \) ksi, and \( F_u = 65 \) ksi. Using A490-N bolt material, determine the maximum capacity of the connection based on shear in the bolts, bearing in all materials and pick the number of bolts and angle length (not staggered). Use A36 steel for the angles.

\[ W21x93: \ d = 21.62 \text{ in}, \ t_w = 0.58 \text{ in}, \ t_f = 0.93 \text{ in} \]
\[ W10x54: \ t_f = 0.615 \text{ in} \]

SOLUTION:

The maximum length the angles can be depends on how it fits between the top and bottom flange with some clearance allowed for the fillet to the flange, and getting an air wrench in to tighten the bolts. This example uses 1” of clearance:

\[
\text{Available length} = \text{beam depth} - \text{both flange thicknesses} - 1” \text{ clearance at top} & \ 1” \text{ at bottom} \\
= 21.62 \text{ in} - 2(0.93 \text{ in}) - 2(1 \text{ in}) = 17.76 \text{ in.}
\]

With the spaced at 3 in. and 1 ¼ in. end lengths (each end), the maximum number of bolts can be determined:

\[
\text{Available length} \geq 1.25 \text{ in.} + 1.25 \text{ in.} + 3 \text{ in.} \times (\text{number of bolts} - 1)
\]

\[
\text{number of bolts} \leq \frac{(17.76 \text{ in} - 2.5 \text{ in.} - (-3 \text{ in.}))/3 \text{ in.} = 6.1, \ so \ 6 \ bolts.}
\]

It is helpful to have the All-bolted Double-Angle Connection Tables 10-1. They are available for ¾”, 7/8”, and 1” bolt diameters and list angle thicknesses of ¼”, 5/16”, 3/8”, and ½”. Increasing the angle thickness is likely to increase the angle strength, although the limit states include shear yielding of the angles, shear rupture of the angles, and block shear rupture of the angles.

For these diameters, the available shear (double) from Table 7-1 for 6 bolts is (6)45.1 k/bolt = 270.6 kips, (6)61.3 k/bolt = 367.8 kips, and (6)80.1 k/bolt = 480.6 kips.

Tables 10-1 (not all provided here) list a bolt and angle available strength of 271 kips for the ¾” bolts, 296 kips for the 7/8” bolts, and 281 kips for the 1” bolts. It appears that increasing the bolt diameter to 1” will not gain additional load. Use 7/8” bolts.

\[
\phi R_n = 367.8 \text{ kips} \quad \text{for double shear of 7/8” bolts} \\
\phi R_n = 296 \text{ kips} \quad \text{for limit state in angles}
\]

We also need to evaluate bearing of bolts on the beam web, and column flange where there are bolt holes. Table 7-4 provides available bearing strength for the material type, bolt diameter, hole type, and spacing per inch of material thicknesses.

a) Bearing for beam web: There are 6 bolt holes through the beam web. This is typically the critical bearing limit value because there are two angle legs that resist bolt bearing and twice as many bolt holes to the column. The material is A992 (\( F_u = 65 \) ksi), 0.58” thick, with 7/8” bolt diameters at 3 in. spacing.

\[
\phi R_n = 6 \text{ bolts} \times 102 \text{ k/bolt/inch} \times (0.58 \text{ in}) = 355.0 \text{ kips}
\]

b) Bearing for column flange: There are 12 bolt holes through the column. The material is A992 (\( F_u = 65 \) ksi), 0.615” thick, with 1” bolt diameters.

\[
\phi R_n = 12 \text{ bolts} \times 102 \text{ k/bolt/inch} \times (0.615 \text{ in}) = 752.8 \text{ kips}
\]

Although, the bearing in the beam web is the smallest at 355 kips, with the shear on the bolts even smaller at 324.6 kips, the maximum capacity for the simple-shear connector is 296 kips limited by the critical capacity of the angles.
Example 20

10.2 The butt splice shown in Figure 10.22 uses two 8 × ½" plates to “sandwich” in the 8 × ½" plates being joined. Four ½" A325-5C bolts are used on both sides of the splice. Assuming A36 steel and standard round holes, determine the allowable capacity of the connection.

**SOLUTION:**

Shear, bearing and net tension will be checked to determine the critical conditions that govern the capacity of the connection.

**Shear:** Using the AISC available shear in Table 7-3 (Group A):

\[
\phi R_n = 26.4 \text{k/bolt} \times 4 \text{ bolts} = 105.6 \text{k}
\]

**Bearing:** Using the AISC available bearing in Table 7-4:

There are 4 bolts bearing on the center (1/2") plate, while there are 4 bolts bearing on a total width of two sandwich plates (3/4" total). The thinner bearing width will govern.

Assume 3 in. spacing (center to center) of bolts. For A36 steel, \( F_u = 58 \text{ ksi} \).

\[
\phi R_n = 91.4 \text{k/bolt/in.} \times 0.5 \text{ in.} \times 4 \text{ bolts} = 182.8 \text{k} \quad \text{(Table 7-4)}
\]

With the edge distance of 2 in., the bearing capacity might be smaller from Table 7-5 which says the distance should be 2 ¼ in for full bearing (and we have 2 in.).

\[
\phi R_n = 79.9 \text{k/bolt/in.} \times 0.5 \text{ in.} \times 4 \text{ bolts} = 159.8 \text{k}
\]

**Tension:** The center plate is critical, again, because its thickness is less than the combined thicknesses of the two outer plates. We must consider tension yielding and tension rupture:

\[
\phi R_n = \phi F_y A_g \quad \text{and} \quad \phi R_n = \phi F_u A_e \quad \text{where} \quad A_e = A_{net} U
\]

\[
A_g = 8 \text{ in.} \times \frac{1}{2} \text{ in.} = 4 \text{ in}^2
\]

The holes are considered 1/8 in. larger than the bolt hole diameter = \((7/8 + 1/8) = 1.0 \text{ in.} \)

\[
A_v = (8 \text{ in.} - 2 \text{ holes} \times 1.0 \text{ in.}) \times \frac{1}{2} \text{ in.} = 3.0 \text{ in}^2
\]

The whole cross section sees tension, so the shear lag factor \( U = 1 \)

\[
\phi F_y A_g = 0.9 \times 36 \text{ ksi} \times 4 \text{ in}^2 = 129.6 \text{k}
\]

\[
\phi F_u A_e = 0.75 \times 58 \text{ ksi} \times (1) \times 3.0 \text{ in}^2 = 130.5 \text{k}
\]

The maximum connection capacity (smallest value) so far is governed by bolt shear: \( \phi R_n = 105.6 \text{k} \)

**Block Shear Rupture:** It is possible for the center plate to rip away from the sandwich plates leaving the block (shown hatched) behind:

\[
\phi R_n = \phi(0.6 F_u A_{nv} + U_{bs} F_u A_{nt}) \leq \phi(0.6 F_y A_{gv} + U_{bs} F_u A_{nt})
\]

where \( A_{nv} \) is the area resisting shear, \( A_{nt} \) is the area resisting tension, \( A_{gv} \) is the gross area resisting shear, and \( U_{bs} = 1 \) when the tensile stress is uniform.

\[
A_{nv} = 2 \times (4 + 2 \text{ in.}) \times \frac{1}{2} \text{ in.} = 6 \text{ in}^2
\]

\[
A_{nv} = A_{gv} - 1 \frac{1}{2} \text{ holes areas} \times 2 \text{ sides} = 6 \text{ in}^2 - 1.5 \times 1 \text{ in.} \times \frac{1}{2} \text{ in.} \times 2 = 4.5 \text{ in}^2
\]

\[
A_{nt} = 3.5 \text{ in.} \times (3 \frac{1}{2} \text{ hole areas}) = 3.5 \text{ in.} \times \frac{1}{2} \text{ in.} - 1 \text{ x 1 in.} \times \frac{1}{2} \text{ in.} = 1.25 \text{ in}^2
\]

\[
\phi(0.6 F_y A_{gv} + U_{bs} F_u A_{nt}) = 0.75 \times (0.6 \times 36 \text{ ksi} \times 6 \text{ in}^2 + 1 \times 58 \text{ ksi} \times 1.25 \text{ in}^2) = 171.8 \text{k}
\]

\[
\phi(0.6 F_y A_{gv} + U_{bs} F_u A_{nt}) = 0.75 \times (0.6 \times 36 \text{ ksi} \times 6 \text{ in}^2 + 1 \times 58 \text{ ksi} \times 1.25 \text{ in}^2) = 151.6 \text{k}
\]

The maximum connection capacity (smallest value) is governed by block shear rupture: \( \phi R_n = 105.6 \text{k} \)
### Listing of W Shapes in Descending order of $Z_x$ for Beam Design

<table>
<thead>
<tr>
<th>$Z_x$ – US (in.³)</th>
<th>$I_x$ – US (in.⁴)</th>
<th>Section</th>
<th>$I_x$ – SI (10⁶mm.⁴)</th>
<th>$Z_x$ – SI (10⁶mm.³)</th>
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<th>$I_x$ – SI (10⁶mm.⁴)</th>
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Listing of W Shapes in Descending order of Zx for Beam Design (Continued)
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Available Critical Stress, $\phi F_{cr}$, for Compression Members, ksi ($F_y = 50$ ksi and $\phi = 0.90$)

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## Bolt Strength Tables

### Table 7-1
Available Shear
Strength of Bolts, kips

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Available Tensile
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\( \Omega = 2.00 \)  
\( \delta = 0.75 \)
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#### Slip-Critical Connections
Available Shear Strength, kips
(Class A Faying Surface, $\mu = 0.30$)

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<th>Bolt Type</th>
<th>Loading</th>
<th>Minimum Group B Bolt Pretension, kips</th>
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</tr>
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</table>

Note: Slip-critical bolt values assume no more than one fillet has been provided or bolts have been bored to distribute loads in the fillets.
See AISC Specification Sections .18 and .20 for provisions when fillets are present.
For Class B faying surfaces, multiply the tabulated available strength by 1.67.
### Table 7-4
Available Bearing Strength at Bolt Holes Based on Bolt Spacing

#### kips/in. thickness

<table>
<thead>
<tr>
<th>Hole Type</th>
<th>Bolt Spacing, s, in.</th>
<th>( F_{s, k} ) ksi</th>
<th>Nominal Bolt Diameter, ( d_i ), in.</th>
<th>( \frac{1}{4} )</th>
<th>( \frac{1}{4} )</th>
<th>( \frac{1}{4} )</th>
<th>( \frac{1}{4} )</th>
<th>( \frac{1}{4} )</th>
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<td>81.7</td>
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<td>82.6</td>
<td>50.8</td>
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<td>65.3</td>
<td>52.2</td>
<td>73.3</td>
<td>63.0</td>
<td>91.4</td>
<td>67.4</td>
<td>101.0</td>
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<td>73.1</td>
<td>58.5</td>
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<td>85.3</td>
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</table>

#### Spacing for full bearing strength, \( f_{min} \), in.

<table>
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<th>Hole Type</th>
<th>STD, SLLT</th>
<th>LSLP</th>
<th>OVS</th>
<th>SSLP</th>
<th>LSLP</th>
<th>Minimum Spacing, ( s = 2 d_i ), in.</th>
</tr>
</thead>
<tbody>
<tr>
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<td>1 1/16</td>
<td>2 1/16</td>
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</tbody>
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#### Minimum Spacing, \( s = 2 d_i \), in.

<table>
<thead>
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<th>STD, SLLT</th>
<th>LSLP</th>
<th>OVS</th>
<th>SSLP</th>
<th>LSLP</th>
<th>Minimum Spacing, ( s = 2 d_i ), in.</th>
</tr>
</thead>
<tbody>
<tr>
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<td>1 1/16</td>
<td>2 1/16</td>
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<td>1 1/16</td>
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</table>

#### Notes:
- \( \omega = 2.00 \) \( \phi = 0.75 \)
- Decimal value has been rounded to the nearest sixteenth of an inch.

---

### Table 7-4 (continued)
Available Bearing Strength at Bolt Holes Based on Bolt Spacing

#### kips/in. thickness

<table>
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<tr>
<th>Hole Type</th>
<th>Bolt Spacing, s, in.</th>
<th>( F_{s, k} ) ksi</th>
<th>Nominal Bolt Diameter, ( d_i ), in.</th>
<th>( \frac{1}{4} )</th>
<th>( \frac{1}{4} )</th>
<th>( \frac{1}{4} )</th>
<th>( \frac{1}{4} )</th>
<th>( \frac{1}{4} )</th>
<th>( \frac{1}{4} )</th>
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#### Spacing for full bearing strength, \( f_{min} \), in.

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<th>SSLP</th>
<th>LSLP</th>
<th>Minimum Spacing, ( s = 2 d_i ), in.</th>
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#### Minimum Spacing, \( s = 2 d_i \), in.

<table>
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<th>OVS</th>
<th>SSLP</th>
<th>LSLP</th>
<th>Minimum Spacing, ( s = 2 d_i ), in.</th>
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#### Notes:
- \( \omega = 2.00 \) \( \phi = 0.75 \)
- Decimal value has been rounded to the nearest sixteenth of an inch.
### Table 7-5

**Available Bearing Strength at Bolt Holes Based on Edge Distance**

**kips/in. thickness**

<table>
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<tr>
<th>Hole Type</th>
<th>Edge Distance L_e, in.</th>
<th>F_e, ksi</th>
<th>Nominal Bolt Diameter, d, in.</th>
<th>( f_e / kips/ft )</th>
<th>( f_o / kips/ft )</th>
<th>( f_e / L_{fr} )</th>
<th>( f_o / L_{fr} )</th>
<th>( f_e / L_{fr} )</th>
<th>( f_o / L_{fr} )</th>
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<td>30.5</td>
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</tr>
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| **Table 7-5 (continued)**  | Available Bearing Strength at Bolt Holes Based on Edge Distance **kips/in. thickness**
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<td>Edge Distance L_e, in.</td>
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<td>( f_e / kips/ft )</td>
<td>( f_o / kips/ft )</td>
<td>( f_e / L_{fr} )</td>
<td>( f_o / L_{fr} )</td>
<td>( f_e / L_{fr} )</td>
<td>( f_o / L_{fr} )</td>
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<td>19.5</td>
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<td>14.6</td>
<td>21.9</td>
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<td>20.3</td>
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<tr>
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<td>58</td>
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</table>

**Notes:**
- **ASD** = American Standard
- **LRFD** = Load and Resistance Factor Design

**Symbols:**
- \( L_e \) = edge distance
- \( d \) = nominal bolt diameter
- \( f_e \) = bearing strength
- \( f_o \) = ultimate strength
- \( L_{fr} \) = effective length of hole

---

**Notes:**
- 1 Decimal values have been rounded to the nearest sixteenth of an inch.
- Indicates spacing less than minimum spacing required per AISC Specification Section J3.3.
- Edge distance for full bearing strength is from the center of the hole or slot to the center of adjacent hole or slot in the line of force. Hole deformation is considered. When hole deformation is not considered, see AISC Specification Section J3.4.
- Indicates spacing less than minimum spacing required per AISC Specification Section J3.3.
- Note: Spacing indicated is from the center of the hole or slot to the center of adjacent hole or slot in the line of force.
Beam Design Flow Chart

Collect data: L, \( \omega \), \( \gamma \), \( \Delta_{\text{limit}} \); find beam charts for load cases and \( \Delta_{\text{actual}} \) equations

ASD (Unified)
Allowable Stress or LRFD Design?

LRFD

Collect data: \( F_y \), \( F_u \), and safety factors \( \Omega \)

Find \( V_{\text{max}} \) & \( M_{\text{max}} \) from constructing diagrams or using beam chart formulas

Find \( Z_{\text{req}} \) and pick a section from a table with \( Z_x \) greater or equal to \( Z_{\text{req}} \)

Determine \( \omega_{\text{self wt}} \) (last number in name) or calculate \( \omega_{\text{self wt}} \) using \( A \) found. Find \( M_{\text{max-adj}} \) & \( V_{\text{max-adj}} \)

Is \( V_{\text{max-adj}} \leq (0.6F_yA_w)/\Omega \)?

No

Calculate \( Z_{\text{req}} \) using \( M_{\text{max-adj}} \)
Is \( Z_{\text{picked}} \geq Z_{\text{req}} \)?

Yes

Yes

Calculate \( \Delta_{\text{max}} \) (no load factors!) using superpositioning and beam chart equations with the \( I_x \) for the section

Is \( \Delta_{\text{max}} \leq \Delta_{\text{limits}} \)? This may be both the limit for live load deflection and total load deflection.

No

pick a section with a larger \( I_x \)

Yes

(DONE)

No

Is \( M_u \leq \phi bM_n \)

No

Is \( V_u \leq \phi v(0.6F_yA_w) \)

No

pick a section with a larger \( I_x \)

Yes

Is \( V_{\text{max-adj}} \leq (0.6F_yA_w)/\Omega \)?

No

pick a new section with a larger web area

Yes

Find a steel section from a chart having \( \phi bM_n \geq M_u \) for the known unbraced length
OR
find \( Z_{\text{req}} \) and pick a section from a table with \( Z_x \) greater or equal to \( Z_{\text{req}} \)

Determine \( \omega_{\text{self wt}} \) (last number in name) or calculate \( \omega_{\text{self wt}} \) using \( A \) found. Factor with \( \gamma_D \). Find \( M_{u-\text{max-adj}} \) & \( V_{u-\text{max-adj}} \)

Is \( Z_{\text{picked}} \geq Z_{\text{req}} \)?

Yes

No

pick a section with a larger web area

Is \( M_u \leq \phi bM_n \)

No

Is \( V_u \leq \phi v(0.6F_yA_w) \)

No

pick a new section with a larger web area

Yes

(DONE)