Steel Design

Notation:

\( a \) = name for width dimension
\( A \) = name for area
\( A_g \) = gross area, equal to the total area ignoring any holes
\( A_{\text{req’d-adj}} \) = area required at allowable stress when shear is adjusted to include self weight
\( A_w \) = area of the web of a wide flange section, as is \( A_{\text{web}} \)

\( AISC \) = 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
\( B \) = width of a column base plate
\( B_1 \) = 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, as is \( c_{\text{max}} \)
\( c_1 \) = 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
\( C_m \) = modification factor accounting for combined stress in steel design
\( C_v \) = web shear coefficient
\( d \) = name for depth
\( D \) = shorthand for dead load
\( DL \) = shorthand for dead load
\( E \) = shorthand for earthquake load
\( = \) modulus of elasticity
\( f_a \) = axial stress
\( f_b \) = bending stress
\( f_p \) = bearing stress
\( f_v \) = shear stress
\( f_{v-\text{max}} \) = maximum shear stress
\( f_y \) = yield stress
\( F \) = shorthand for fluid load
\( F_a \) = allowable axial (compressive) stress
\( F_b \) = allowable bending stress
\( F_{cr} \) = flexural buckling stress
\( F_e \) = elastic critical buckling stress
\( F_p \) = allowable bearing stress
\( F_u \) = ultimate stress prior to failure
\( F_y \) = yield strength
\( F_{yw} \) = yield strength of web material
\( h \) = 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
\( I_y \) = 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, as is \( L \)
\( = \) column base plate design variable
\( L \) = name for length or span length, as is \( l \)
\( = \) shorthand for live load
\( L_b \) = unbraced length of a steel beam in LRFD design
\( L_c \) = 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
\( LL \) = shorthand for live load
\( LRFD \) = load and resistance factor design
\( m \) = edge distance for a column base plate
\( M \) = internal bending moment
\( M_a \) = required bending moment (ASD)
\( M_{\text{max}} \) = maximum internal bending moment
\( M_{\text{max-adj}} \) = maximum bending moment adjusted to include self weight
$M_n$ = nominal flexure strength with the full section at the yield stress for LRFD beam design

$M_p$ = 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

$n$ = edge distance for a column base plate

$n'$ = column base plate design value

$n.a.$ = shorthand for neutral axis

$N$ = bearing length on a wide flange steel section

$= depth of a column base plate

$P$ = name for load or axial force vector

$P_a$ = required axial force (ASD)

$P_c$ = available axial strength

$P_{el}$ = Euler buckling strength

$P_r$ = required axial force

$P_n$ = nominal column load capacity in LRFD steel design

$P_p$ = nominal bearing capacity of concrete under base plate

$P_u$ = factored column load calculated from load factors in LRFD steel design

$r$ = radius of gyration

$R$ = generic load quantity (force, shear, moment, etc.) for LRFD design

$r$ = shorthand for rain or ice load

$R_a$ = 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

$S$ = shorthand for snow load

$= section modulus

$S_{req'd}$ = section modulus required at allowable stress

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

$T_f$ = thickness of flange of wide flange

$T_{min}$ = minimum thickness of column base plate

$T_w$ = thickness of web of wide flange

$T$ = torque (axial moment)

$= shorthand for thermal load

$V$ = internal shear force

$V_a$ = required shear (ASD)

$V_{max}$ = maximum internal shear force

$V_{max-adj}$ = maximum internal shear force adjusted to include self weight

$V_n$ = nominal shear strength capacity for LRFD beam design

$V_u$ = maximum shear from factored loads for LRFD beam design

$w_{equivalent}$ = the equivalent distributed load derived from the maximum bending moment

$w_{self wt}$ = name for distributed load from self weight of member

$W$ = shorthand for wind load

$X$ = column base plate design value

$Z$ = plastic section modulus of a steel beam

$Z_{req'd}$ = plastic section modulus required

$Z_x$ = plastic section modulus of a steel beam with respect to the x axis

$\Delta_{actual}$ = actual beam deflection

$\Delta_{allowable}$ = allowable beam deflection

$\Delta_{limit}$ = allowable beam deflection limit

$\Delta_{max}$ = maximum beam deflection

$\varepsilon_y$ = yield strain (no units)

$\phi$ = resistance factor

$\phi_b$ = resistance factor for bending for LRFD

$\phi_c$ = resistance factor for compression for LRFD

$\phi_v$ = resistance factor for shear for LRFD

$\lambda$ = column base plate design value

$\gamma$ = load factor in LRFD design

$\pi$ = pi (3.1415 radians or 180°)

$\rho$ = radial distance

$\Omega$ = safety factor for ASD
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 used 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 60 has the same properties as A992)

**ASD**

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

where
- \( R_a \) = 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**

\[ R_u \leq \phi R_n \]

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

**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 combinations of factored nominal loads:

- \( 1.4D \)
- \( 1.2D + 1.6L + 0.5(L_n or S or R) \)
- \( 1.2D + 1.6(L_n or S or R) + (L or 0.5W) \)
- \( 1.2D + 1.0W + L + 0.5(L_n or S or R) \)
- \( 1.2D + 1.0E + L + 0.2S \)
- \( 0.9D + 1.0W \)
- \( 0.9D + 1.0E \)

**Criteria for Design of Beams**

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

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

\( (M_u \leq M_n / \Omega \text{ or } M_u \leq \phi_n M_n) \)

Knowing \( M \) and \( F_y \), the minimum plastic section modulus fitting the limit is:

\[ Z_{req'd} \geq \frac{M_a}{F_y \Omega} \]

\[ (S_{req'd} \geq \frac{M}{F_b}) \]

Besides strength, we also need to be concerned about serviceability. This involves things like limiting deflections & cracking, controlling noise and vibrations, preventing excessive settlements of foundations and durability. When we know about a beam section and its material, we can determine beam deformations.
Determining Maximum Bending Moment

Drawing V and M diagrams will show us the maximum values for design. Computer applications are very helpful.

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

Elastic curve equations can be found in handbooks, textbooks, design manuals, etc...Computer programs can be used as well.

Elastic curve equations can be superpositioned 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. \[ \Delta_{\text{actual}} \leq \Delta_{\text{allowable}} = \frac{L}{\text{value}} \]

<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 I 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:

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

where \( t_w \) = thickness of the web  
\( N \) = bearing length  
\( k \) = dimension to fillet found in beam section tables

\[ \phi = 1.00 \text{ (LRFD)} \quad \Omega = 1.50 \text{ (ASD)} \]

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, \( \phi_bM_n \), for resistance to pure bending (no axial load) in most flexural members where the following conditions exist, a single calculation will suffice:

\[ \Sigma \gamma_i R_i = M_n \leq \phi_b M_n = 0.9F_{y}Z \]
where 

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

**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.

\[
A_{tension} = A_{compression}
\]
**Instability from Plastic Hinges:**

![Diagram of plastic hinges]

**Shape Factor:**

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

\[
\frac{M_p}{M_y} = k \quad \text{for a rectangle}
\]

\[
\kappa = \frac{3}{2} \quad \text{for a rectangle}
\]

\[
\kappa \approx 1.1 \quad \text{for an I beam}
\]

**Plastic Section Modulus**

\[
Z = \frac{M_p}{f_y} \quad \text{and} \quad k = \frac{Z}{S}
\]

**Design for Shear**

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

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)**

\[
\frac{h}{t_w} \leq 2.24 \frac{E}{F_y} \sqrt{\frac{V_n}{0.6F_{yw}A_w}} \quad \phi_V = 1.00 \quad (LRFD) \quad \Omega = 1.50 \quad (ASD)
\]

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)**

\[
\frac{h}{t_w} > 2.24 \frac{E}{F_y} \sqrt{\frac{V_n}{0.6F_{yw}A_wC_v}} \quad \phi_V = 0.9 \quad (LRFD) \quad \Omega = 1.67 \quad (ASD)
\]

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

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

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

1. yielding, limited at length \( L_p = 1.76 r_y \sqrt{\frac{E}{F_y}} \), where \( r_y \) is the radius of gyration in \( y \)
2. lateral-torsional buckling limited at length \( L_r \)
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, \( L_{lb} \), 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 \( C_b \). \( C_b = 1 \) when the bending moment at any point within an unbraced length is larger than that at both ends of the length. \( C_b \) 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.

**NOTE:** the self weight is not included in determination of \( \phi_b M_n \)

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

\[ M_a = C_b \left[ M_p - (M_p - 0.7 F_y S_s \left( \frac{L_p - L_p}{L_r - L_p} \right)) \right] \leq M_p \]

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:

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

$M_{\text{max}}$ = 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 $M_a \leq \frac{M_u}{\Omega}$ or $M_a \leq \phi M_u$. 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 limit $L_p$ is indicated by a solid dot (●), while $L_r$ 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_{max} and M_{max} for unfactored loads (ASD, V_a & M_a) or from factored loads (LRFD, V_u & M_u).

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

\[ f_b = \frac{M_{max}}{S} \leq F_b, \quad Z_{req'd} \geq \frac{M_{max}}{F_b} = \frac{M_{max}}{F_{\gamma}} \frac{1}{\Omega} \quad \text{and} \quad Z_{req'd} \geq \frac{M_{u}}{\phi_y F_{\gamma}} \]

\[ M_a \leq M_n / \Omega \quad \text{or} \quad M_u \leq \phi_b M_n \]

If the limit state is something other than yielding, determine the nominal moment, M_{sn}, 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_{req'd}. The design charts show the lightest section within a grouping of similar Z's.

5. Evaluate horizontal shear using V_{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 \( V_u \leq \phi_v V_n \).

For I beams: \[ f_{v-max} = \frac{3V}{2A} \approx \frac{V}{A_{web}} = \frac{V}{t_w d} \quad V_n = 0.6F_{yw}A_w \quad \text{or} \quad V_n = 0.6F_{yw}A_w C_v \]

Others: \[ f_{v-max} = \frac{VQ}{I_b} \]

6. 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 \( P_u \leq \phi_p P_n \).

7. Evaluate shear due to torsion \( f_v = \frac{T \rho}{J} \frac{T}{c_t ab^2} \leq F_v \) (circular section or rectangular)

8. Evaluate the deflection to determine if \( \Delta_{max,LL} \leq \Delta_{LL-allowed} \quad \text{and/or} \quad \Delta_{max,Total} \leq \Delta_{Total-allowed} \)

**** note: when \( \Delta_{calculated} > \Delta_{limit} \), \( I_{required} \) can be found with: \( I_{req'd} \geq \frac{\Delta_{no\big} \Delta_{limit}}{\Delta_{trial}} \)

**** Determine the "updated" \( V_{max} \) and \( M_{max} \) including the beam self weight, and verify that the updated \( Z_{req'd} \) has been met.********
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 either provide the safe distributed load based on bending and deflection limits, they give the allowable span for specific live and dead loads including live load 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 is less, the design live load to check against allowable must be increased, ex.

\[
w_{\text{adjusted}} = \left( \frac{L}{360} \right) \left( \frac{L}{400} \right) \text{table limit} \quad \text{wanted}
\]

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.

Design for Compression

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

\[
P_u \leq \frac{P_n}{\Omega} \quad \text{or} \quad P_u \leq \phi P_n
\]

where

\[
P_u = \sum \gamma_i P_i
\]

\(\gamma\) 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 \( \frac{KL}{r} \leq 4.71 \left( \frac{E}{F_y} \right) \) or \( (F_e \geq 0.44F_y) \):

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

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

\[
F_{cr} = 0.877F_e
\]

where \( F_e \) is the elastic critical buckling stress:

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

**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 \( r_x/r_y \).
Procedure for Analysis

1. Calculate $KL/r$ for each axis (if necessary). The largest will govern the buckling load.
2. Find $F_{cr}$ as a function of $KL/r$ from the appropriate equation (above) or table.
3. Compute $P_n = F_{cr} \cdot A_g$
   or alternatively compute $f_c = P_d/A$ or $P_u/A$
4. Is the design satisfactory?
   Is $P_a \leq P_n/\Omega$ or $P_u \leq \phi P_n$? ⇒ yes, it is; no, it is no good
   or Is $f_c \leq F_{cr}/\Omega$ or $\phi F_{cr}$? ⇒ 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 $F_{cr}$ as a function of $KL/r$ from appropriate equation (above) or table.
4. Compute $P_n = F_{cr} \cdot A_g$
   or alternatively compute $f_c = P_d/A$ or $P_u/A$
5. Is the design satisfactory?
   Is $P \leq P_n/\Omega$ or $P_u \leq \phi P_n$? yes, it is; no, pick a bigger section and go back to step 2.
   Is $f_c \leq F_{cr}/\Omega$ or $\phi F_{cr}$? ⇒ yes, it is; no, pick a bigger section and go back to step 2.
6. Check design efficiency by calculating percentage of stress used:
   \[ \frac{P_a}{P_n} \cdot 100\% \text{ or } \frac{P_a}{\phi P_n} \cdot 100\% \]
   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

1. See if we can find values for $r$ or $A$ or $Z$. 
2. Pick a trial section based on if we think $r$ or $A$ is going to govern the section size.
3. Analyze the stresses and compare to allowable using the allowable stress method or interaction formula for eccentric columns.

4. 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)?

5. 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 \( \phi_c P_n \) for LRFD). The increased bending moment due to the \( P\Delta \) effect must be determined and used as the moment to resist.

For \( \frac{P_r}{P_c} \geq 0.2 \):

\[
\frac{P}{P_n} + \frac{8}{9} \left( \frac{M_x}{M_{nx}/\Omega} + \frac{M_y}{M_{ny}/\Omega} \right) \leq 1.0
\]

\[
\frac{P_u}{\phi_c P_n} + \frac{8}{9} \left( \frac{M_{ux}}{\phi_b M_{nx}/\Omega} + \frac{M_{uy}}{\phi_b M_{ny}/\Omega} \right) \leq 1.0
\]

(ASD) (LRFD)

For \( \frac{P_r}{P_c} < 0.2 \):

\[
\frac{P}{2P_n} + \left( \frac{M_x}{M_{nx}/\Omega} + \frac{M_y}{M_{ny}/\Omega} \right) \leq 1.0
\]

\[
\frac{P_u}{2\phi_c P_n} + \left( \frac{M_{ux}}{\phi_b M_{nx}/\Omega} + \frac{M_{uy}}{\phi_b M_{ny}/\Omega} \right) \leq 1.0
\]

(ASD) (LRFD)

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

\[
B_1 = \frac{C_m}{1 - \alpha \left( \frac{P_u}{P_{el}} \right)} \geq 1.0
\]

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 (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)

\( P_{el} = \frac{\pi^2 EA}{(Kl/r)^2} \)
Criteria for Design of Connections and Tension Members

Refer to the specific note set.

Criteria for Design of Column Base Plates

Column base plates are designed for bearing on the concrete (concrete capacity) and flexure because the column “punches” down the plate and it could bend upward near the edges of the column (shown as $0.8b_f$ and $0.95d$). The plate dimensions are $B$ and $N$ and are preferably in full inches with thicknesses in multiples of 1/8 inches.

**LRFD minimum thickness:**

$$t_{\text{min}} = l \sqrt{\frac{2P_u}{0.9F_y BN}}$$

where $l$ is the larger of $m$, $n$ and $\lambda n'$

$$m = \frac{N - 0.95d}{2} \quad n = \frac{B - 0.8b_f}{2}$$

$$n' = \frac{\sqrt{db_f}}{4} \quad \lambda = \frac{2\sqrt{X}}{(1 + \sqrt{1 - X})} \leq 1$$

where $X$ depends on the concrete bearing capacity of $\phi_c P_p$, with

$$\phi_c = 0.65 \quad P_p = 0.85 f'_c A$$

$$X = \frac{4db_f}{(d + b_f)^2} \cdot \frac{P_u}{\phi_c P_p} = \frac{4db_f}{(d + b_f)^2} \cdot \frac{P_u}{\phi_c (0.85 f'_c) BN}$$
### Listing of W shapes in Descending Order of $Z_x$ for Beam Design

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### Listing of W Shapes in Descending order of $Z_x$ for Beam Design (Continued)

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Beam Design Flow Chart

1. **Collect data:** L, ω, γ, ∆_{lim}Δ_max ≤ ∆_{lim}?
   - This may be both the limit for live load deflection and total load deflection.

2. **ASD Allowable Stress Design?**
   - Collect data: F_y, F_u, and safety factors Ω
   - Find V_{max} & M_{max} from constructing diagrams or using beam chart formulas
   - Find Z_{req'd} and pick a section from a table with Z_x greater or equal to Z_{req'd}
   - Determine ω_{self wt} (last number in name) or calculate ω_{self wt}, using A found. Find M_{max-adj} & V_{max-adj}.
   - Is Z_{picked} ≥ Z_{req'd-adj}?
     - Yes: Calculate Δ_{max} (no load factors!) using superpositioning and beam chart equations with the I_{x} for the section
       - Is Δ_{max} ≤ ∆_{lim}?
         - Yes: (DONE)
         - No: pick a section with a larger I_{x}

3. **LRFD Design?**
   - Collect data: load factors, F_y, F_u, and equations for shear capacity with φ_v
   - Find V_u & M_u from constructing diagrams or using beam chart formulas with the factored loads
   - Pick a steel section from a chart having φ_b M_{n} ≥ M_{u} for the known unbraced length OR find Z_{req'd} and pick a section from a table with Z_x greater or equal to Z_{req'd}
   - Determine ω_{self wt} (last number in name) or calculate ω_{self wt}, using A found. Factor with γ_D. Find M_{u-max-adj} & V_{u-max-adj}.
   - Is M_u ≤ φ_b M_{n}?
     - Yes: Is V_u ≤ φ_v (0.6F_{yw}A_{w})?
       - Yes: pick a section with a larger web area
       - No: Calculate ∆_{max}
     - No: pick a new section with a larger web area