SUMMARY FOR COMPRESSION MEMBERS

Columns with Pinned Supports

**Step 1:** Determine the factored design loads (AISC/LRFD Specification A4).

**Step 2:** From the column tables, determine the effective length $KL$ using

$$KL = \max \left\{ K_y L_y \text{(weak-axis)}, \frac{K_x L_x}{r_x/r_y} \text{(strong-axis)} \right\}$$

and pick a section.

**Step 3:** Check using Table 3-36 or 3-50.

1. Calculate $KL/r$ and enter into Table 3-36 or 3-50.
2. Find the design stress $\phi_c F_{cr}$.
3. Find the design strength $\phi_c F_{cr} A_g$.

or using formulas:

$$\lambda_c = \max \left\{ \frac{K_x L_x}{r_x \pi} \sqrt{\frac{F_y}{E}}, \frac{K_y L_y}{r_y \pi} \sqrt{\frac{F_y}{E}} \right\}$$

$$F_{cr} = \begin{cases} 0.658 \lambda_c^2 F_y & \text{for } \lambda_c < 1.5 \\ 0.877 F_E = \frac{0.877}{\lambda_c^2} F_y & \text{for } \lambda_c \geq 1.5 \end{cases}$$

$$\phi_c P_n = \phi_c F_{cr} A_g = 0.85 F_{cr} A_g$$
Design Procedure for Columns in Frames

**Step 1:** Determine the factored design loads.

**Step 2:** Guess initial column size: Since $K_x$ is unknown, use $KL = K_y L_y$.

**Step 3:** Calculate design strength.

1. Find the properties of all girders and columns.
2. Calculate $G_A$ and $G_B$ using the equation
   \[ G = \frac{\sum I_c}{\sum I_g}, \]
   \[ G = \frac{\sum \frac{I_c}{L_c}}{\sum \frac{I_g}{L_g}}. \]
   If the column is **not rigidly** supported by a footing or foundation (i.e., pinned footing), then take $G_B = 10$. If the column is **rigidly** supported by a footing or foundation, then take $G_B = 1$.
3. Use stiffness reduction factor if applicable.
4. Determine $K_x$ from the alignment chart. There are two cases: braced frame (sideway is inhibited), and unbraced frames (sideway is uninhibited).
5. Determine the effective length $KL$:
   \[ KL = \max \left\{ K_y L_y (\text{weak-axis}), \frac{K_x L_x}{r_x/r_y} (\text{strong-axis}) \right\}. \]
6. Enter into the column table to get the approximate design strength.

**Step 4:** Redesign. If the capacity is significantly different from the design load, it is necessary to pick a new column. Use the following approximate formula:

\[ \text{Weight (new column)} = \frac{\text{Weight (old column)} \times \text{Load}}{\text{Capacity (old column)}} \]

Repeat Steps 3 and 4 until satisfactory conditions are met.

**Step 5:** Check the result using Table 3-36, 3-50, or the formula.
SUMMARY FOR BEAMS

Design Procedure for Beams

There is no standard set of design steps but the following will give some indication of how most designs proceed:

Step 1: Design Load
Find the maximum moment $M_u$ and the maximum shear $V_u$. The Beam Diagrams and Formulas are helpful for the case of unusual loads. For laterally unsupported beams, also find the moment gradient factor $C_b$.

Step 2: Select a member.
Find the lightest beam which has a moment capacity $\phi_b M_n$, greater than the design load $M_u$. Use the Load Factor Design Selection Table for laterally supported beams and the Beam Design Moments Charts for laterally unsupported beams.

Step 3: Check member.

- **Deflection:** Check if deflections for the unfactored live load and for the service load are less that $L/360$ and $L/240$, respectively. The Beam Diagrams and Formulas are useful in this step. If deflections are too large, use the Moment of Inertia Selection Tables to find a beam with a larger moment of inertia.

- **Shear:** Check if the shear capacity $\phi_s V_n$ is greater than the maximum shear $V_u$. If the shear capacity is too small, find a heavier and deeper beam using the Load Factor Design Selection Table.

- **Moments:** Calculate the moment capacity $\phi_b M_n$ using the design formulas for local buckling and, if necessary, lateral-torsional buckling. The factor of safety for beams is $\phi_b = 0.90$. The result should be very close to the value tabulated in Load Factor Design Selection Table or Beam Design Moments Charts.
Moment Gradient Factor $C_b$

The increased strength from the moment gradient is quantified by the moment gradient factor $C_b$. The formula for this factor is

$$C_b = 1.75 + 1.05 \left( \frac{M_1}{M_2} \right) + 0.3 \left( \frac{M_1}{M_2} \right)^2 \leq 2.3$$

where $M_1$ and $M_2$ are the end moments, chosen such that $|M_1| \leq |M_2|$; the sign of $M_1/M_2$ is negative for single curvature bending and positive for reverse curvature bending; $C_b = 1$ if the moment with a “significant portion of the unbraced segment” is greater than or equal to $|M_2|$; and $C_b = 1$ for cantilevers.

Deflections Limits

The maximum deflections of the beam must be checked for live and service loads. The Beam Diagrams and Formulas is a handy reference for the maximum deflections, which are denoted by $\Delta_{\text{max}}$. After the maximum deflections are computed for the live load ($\Delta_{\text{live, max}}$) and for the service load ($\Delta_{\text{service, max}}$), they must be compared with the design limits, which are given below:

- **Live load limit**: Live load only, without 1.6 factor
  $$\Delta_{\text{live, max}} \leq \frac{L}{360}$$

- **Service load limit**: Dead load + service loads, without 1.2 or 1.6 factors
  $$\Delta_{\text{service, max}} \leq \frac{L}{240}$$

Web Shear Capacity

Check if the shear capacity $\phi_y V_n$ is greater than the maximum shear computed from the loads $V_u$. The shear capacity is given by

$$\phi_y V_n = 0.54 F_y d t_w$$
Moment Capacity for Local Buckling

The moment capacity $M_n$ for local buckling analysis is calculated based on the slenderness ratios $\lambda_f$ and $\lambda_w$. The AISC Specifications gives the following values for $\lambda_p$ and $\lambda_r$:

$$
\lambda_{pf} = \frac{65}{\sqrt{F_y}} \quad \lambda_{pw} = \frac{640}{\sqrt{F_y}} \\
\lambda_{rf} = \frac{141}{\sqrt{F_y - 10}} \quad \lambda_{rw} = \frac{970}{\sqrt{F_y}}
$$

One of the following three sets of formulas for the moment capacities are used:

- **Compact Sections:** The slenderness ratios for both the flange and web satisfy

\[
\lambda_f = \frac{b_f}{2t_f} \leq \lambda_{pf} \quad \lambda_w = \frac{h_c}{t_w} \leq \lambda_{pw}
\]

The nominal strength is given by the full plastic moment:

$$
M_n = M_p = ZF_y
$$

- **Partially Compact Sections:** The slenderness ratio for the flange or web (or both) satisfies

$$
\lambda_p < \lambda \leq \lambda_r
$$

The nominal moment is given by an interpolated value between the plastic and residual moments:

$$
M_n = M_p - (M_p - M_r) \left( \frac{\lambda - \lambda_p}{\lambda_r - \lambda_p} \right)
$$

The residual moment is less than the yield moment to account for residual stresses, and is defined by:

$$
M_r = (F_y - F_r)S
$$

If both the flange and the web satisfy the condition $\lambda_p < \lambda \leq \lambda_r$, then the smaller of the two interpolated values is used.

- **Non-Compact Sections:** The slenderness ratio for the flange or web satisfies

$$
\lambda > \lambda_r
$$

This case is not studied in 1.51. The details are found in AISC Appendices F and G.
Moment Capacity for Lateral-Torsional Buckling

The moment capacity $M_n$ for lateral torsional buckling analysis is calculated based on the unbraced length $L_b$.

Unbraced Length Limits:

$$L_p = \frac{300r_y}{\sqrt{F_y}} \quad L_r = \frac{r_yX_1}{F_y - F_r} \sqrt{1 + \sqrt{1 + X_2(F_y - F_r)^2}}$$

$$X_1 = \frac{\pi}{S_x} \sqrt{\frac{E G J A}{2}} \quad \text{and} \quad X_2 = \frac{4G_w}{I_y} \left( \frac{S_x}{G J} \right)^2$$

Moment Capacities for LTB:

- $L_b \leq L_p$: No LTB. The nominal strength is given by the full plastic moment:
  $$M_n = M_p = Z_x F_y$$

- $L_p < L_b \leq L_r$: Inelastic LTB. The nominal moment is given by an interpolated value between the plastic and residual moments:
  $$\phi_b M_n = C_b [\phi_b M_p - BF(L_b - L_p)] \leq \phi_b M_p$$

- $L_r < L_b$: Elastic LTB.
  $$M_n = M_{cr} = \frac{C_b S_x X_1 \sqrt{2}}{L_b / r_y} \sqrt{1 + \frac{X_1^2 X_2}{2(L_b / r_y)^2}} \leq M_p$$

Comparison of Moment Capacities:

The nominal strength is the smaller of the LB and LTB results:

$$M_n = \min\{ M_n \text{ from LB analysis}, \ M_n \text{ from LTB analysis} \}$$