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:

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

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

$$\phi_c P_n = \phi_c F_{cr} A_g = 0.85 F_{cr} A_g$$

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

$$G = \frac{\sum \frac{I_c}{L_c}}{\sum \frac{I_g}{L_g}}.$$

   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** $\phi_V V_n$ 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 Δ_{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 $\phi_V V_n$ is greater than the maximum shear computed from the loads V_u. The shear capacity is given by

$$\phi_v V_n = 0.54 F_y d t_w$$

Moment Capacity for Local Buckling

The moment capacity M_n for local buckling analysis is calculated based on the slenderness ratios λ_f and λ_w. The AISC Specifications gives the following values for λ_p and λ_r:

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

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

  $$\lambda_f = \frac{b_f}{2t_f} \leq \lambda_{pf} \qquad \lambda_w = \frac{h_c}{t_w} \leq \lambda_{pw}$$

  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 Appencides 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:

$$L_p = \frac{300 r_y}{\sqrt{F_y}} \qquad L_r = \frac{r_y X_1}{F_y - F_r}\sqrt{1 + \sqrt{1 + X_2(F_y - F_r)^2}}$$

$$X_1 = \frac{\pi}{S_x}\sqrt{\frac{EGJA}{2}} \quad \text{and} \quad X_2 = \frac{4G_w}{I_y}\left(\frac{S_x}{GJ}\right)^2$$

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:

$$\phi_b M_n = C_b \left[\phi_b M_p - \mathrm{BF}(L_b - L_p)\right] \leq \phi_b M_p$$

- $L_r < L_b$: 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:

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