14 DESIGN OF BEAM-COLUMNS - II

1.0 INTRODUCTION

Beam-columns are members subjected to combined bending and axial compression. Their behaviour under uniaxial bending, biaxial bending and torsional flexural buckling were discussed in Part I on this topic in the previous chapter. It was shown that a range of behaviour varying from flexural yielding to torsional flexural or flexural buckling is possible.

In this chapter evaluation of strength of beam-columns is discussed. The steps in the analysis of strength of beam-column are presented along with an example.

2.0 STRENGTH OF BEAM-COLUMNS

The discussions in the part I of this topic, clearly indicated that the behaviour of beam-columns is fairly complex, particularly at the ultimate stage and hence exact evaluation of the strength would require fairly complex analysis. However, for design purposes, simplified equations are available, using which it is possible to obtain the strength of members, conservatively. These are discussed below.

2.1 Modes of Failure

The following are the possible modes of failure of beam-columns

2.1.1 *Local section failure*

This is usually encountered in the case of short, stocky beam columns ($\ell/r << 50$) with relatively smaller axial compression ratio ($F_c/P_y < 0.33$) and beam-columns bent in reverse curvature.

- The strength of the end section reached under combined axial force and bending moment, governs the failure.

- The strength of the section may be governed by plastic buckling of plate elements in the case of plastic, compact and semi-compact sections or the elastic buckling of plate elements in the case of slender sections (see the chapter on plate buckling).

2.1.2 Overall instability failure under flexural yielding

This type of failure is encountered in the case of all members subjected to larger compression ($F_c/P_y > 0.5$) and single curvature bending about the minor axis as well as not very slender members subjected to axial compression and single curvature bending about the major axis.

- The member fails by reaching the strength of the member at a section over the length of the member, under the combined axial compression and magnified bending moment.

- In the case of weak axis bending of slender members ($\ell/r > 80$), the failure may be by weak axis buckling, or failure of the maximum moment section under the combined effect of axial force and magnified moment.

- The section failure may be due to elastic or plastic plate buckling depending on the slenderness ratio (b/t) of the plate (See the chapter on plate buckling).

2.1.3 Overall instability by torsional flexural buckling

This is common in slender members ($\ell/r>80$) subjected to large compression ($F_c/P_y>0.5$) and uniaxial bending about the major axis or biaxial bending.

- At the ultimate stage the member undergoes biaxial bending and torsional instability mode of failure.

2.2 Design Equations

The design code specifies, as given below, the linear interaction equations to check the section strength to prevent local section failure as well as member failure by flexural yielding and torsional flexural buckling. These are conservative simplifications of the non-linear failure envelopes discussed in the previous chapter.

2.2.1 *Local section failure*

The interaction equation is given by:

$$\frac{F_c}{A_g\, f_{yd}} + \frac{M_x}{Z_x\, f_{yd}} + \frac{M_y}{Z_y\, f_{yd}} \leq 1.0 \qquad (5)$$

where F_c, M_x and M_y are the actual compressive force and bending moments about the major (x) axis and minor (y) axis of the cross section, respectively. A_g is the gross area of cross section in the case of plastic, compact and semi-compact sections and effective area, A_{eff}, of cross section in the case of slender sections (see the chapters on cold - formed steel members). Z_x , Z_y are the plastic section moduli, Z_p, of the cross section about the major and minor axis, respectively in plastic and compact sections and the elastic section moduli, S_x, S_y in non-compact sections and effective section moduli, S_{eff}, in slender sections, respectively. The f_{yd} is the design yield strength given by f_y/γ_{mo} ($\gamma_{mo} = 1.15$). Normally, actual bending moments at one of the two ends of the compressive member would govern. The moments obtained from the linear-elastic analysis would suffice for normal buildings with only a few storeys and lower axial compression. In very tall buildings with a large axial compression and large lateral sway, the end moments after accounting for the P-Δ effects have to be considered.

2.2.2 *Overall Member failure*

The interaction equation to check the member capacity as governed by overall member buckling is given by

$$\frac{F_c}{F_{cl}} + k_x \frac{M_x}{M_{ux}} + k_y \frac{M_y}{M_{uy}} \leq 1.0 \qquad (6)$$

where, F_c, M_x, and M_y are the actual axial compression, and actual bending moments about the major (x) and minor (y) axes, respectively. F_{cl}, M_{ux}, M_{uy} are the design compressive strength, and the bending strength about the x and y axis, respectively, when only the corresponding axial force/bending moment is acting. These are calculated considering buckling in the case of compression and bending about major axis. The method of calculating these strengths was discussed in the respective chapters on compression members and beams. These design strengths have to be calculated considering the type of section (plastic, compact, semi-compact and slender) as well as lateral buckling, in the case of strong axis bending. k_x, k_y are the moment amplification factors which account for the effect of moment gradient over the member length, instead of uniform moment over the entire length, and magnification of moments due to the axial force acting on the deformed column. These are given by:

$$k = 1 - \frac{\mu F_c}{P_c} \leq 1.5 \qquad (7)$$

$$\text{where, } \mu = \bar{\lambda}\,(2\beta_M - 4) + \left(\frac{Z - S}{S}\right) \leq 0.9 \quad \text{or Class 1 and Class 2 sections}$$

$$\beta_M = 1.8 - 0.7\,(M_{\min} / M_{\max})$$

$$= 1.3 \qquad (\text{for in plane lateral UDL over the member})$$

$$= 1.4 \qquad (\text{for in plane lateral concentrated load over the member})$$

$$P_c = \text{compressive strength about the respective axis}$$

$$Z, S = \text{plastic and elastic section moduli, respectively}$$

More accurate evaluation of beam-column strength is possible by resorting to non-linear P-Δ analysis. In this case, the actual axial compression and bending moments as obtained from such an analysis may be used in the interaction equation and the sway effects may be disregarded in evaluation of P_u, P_{ex} and P_{ey}. These methods of analysis and design are beyond the scope of this chapter and are not discussed herein.

4.0 STEPS IN ANALYSING A BEAM-COLUMN

(i) Calculate the cross section properties.

Area, principal axes moments of inertia, section moduli, radii of gyration, effective lengths and slenderness ratios.

(ii) Evaluate the type of section based on the (*b/t*) ratio of the plate elements, as plastic, compact, semi-compact, or slender.

(iii) Check for resistance of the cross-section under the combined effects as governed by yielding (Eq. 5).

(iv) Check for resistance of member under the combined effects as governed by buckling (Eq. 6).

4.0 SUMMARY

This chapter presented equations for the design of beam-columns and an example design. The behaviour and design of beam-columns are contained in the two parts on this topic. The following are the important points discussed in these chapters.

- The beam-column may fail by reaching either the ultimate strength of the section (in the case of smaller axial load and shorter members) or by the buckling strength as governed by weak axis buckling or lateral torsional buckling.
- At lower loads, the failure is likely to be after the formation of the plastic hinges, especially in the case of shorter members.
- In slender beam-columns with larger axial compression, either weak axis or lateral torsional buckling would control failure.
- The interaction formulae given for the design are conservative and simple, considering the complicated nature of beam-column failure.
- In the design of beam-columns in frames, the magnification of moment due to P-δ and P-Δ effects are to be considered.

5.0 REFERENCES

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2. Dowling P.J, Knowles and Owens, G.W., “Structural Steel Design”, Butterworth, London, 1998.

3. Eurocode 3: 1992, “Design of Steel structures”, British Standards Institution