

Compression Members

Compression members are structural elements primarily subjected to axial compressive forces and hence, their design is guided by considerations of strength and buckling. examples: pedestal, column, wall and strut. While pedestal, column and wall carry the loads along its length in vertical direction, the strut in truss carries loads in any direction. These compression members may be made of bricks or reinforced concrete.

10.21.2 Definitions

(a) Effective length: The vertical distance between the points of inflection of the compression member in the buckled configuration in a plane is termed as effective length of that compression member in that plane. The effective length is different from the unsupported length L of the member, though it depends on the unsupported length and the type of end restraints. The relation between the effective and unsupported lengths of any compression member is $L_e = kL$

where K is the ratio of effective to the unsupported lengths. Clause 25.2 of IS 456 stipulates the effective lengths of compression members (vide Annex E of IS 456). This parameter is needed in classifying and designing the compression members.

(b) Pedestal: Pedestal is a vertical compression member whose effective length / 9 does not exceed three times of its least horizontal dimension b (cl. 26.5.3.1h, Note). The other horizontal dimension D shall not exceed four times of b (Fig.10.21.1a).

(c) Column: Column is a vertical compression member whose unsupported length L shall not exceed sixty times of b (least lateral dimension), if restrained at the two ends. Further, its unsupported length of a cantilever column shall not exceed $100b^2/D$, where D is the larger lateral dimension which is also restricted up to four times of b (vide cl. 25.3 of IS 456 and Fig.10.21.1b).

(d) Wall: Wall is a vertical compression member whose effective height H to thickness t (least lateral dimension) shall not exceed 30 (cl. 32.2.3 of IS 456). The larger horizontal dimension i.e., the length of the wall L is more than $4t$ (Fig.10.21.1c).

10.21.3 Classification of Columns Based on Types of Reinforcement

Based on the types of reinforcement, the reinforced concrete columns are classified into three groups:

(i) Tied columns: The main longitudinal reinforcement bars are enclosed within closely spaced lateral ties (Fig.10.21.2a).

(ii) Columns with helical reinforcement: The main longitudinal reinforcement bars are enclosed within closely spaced and continuously wound spiral reinforcement. Circular and octagonal columns are mostly of this type (Fig.10.21.2b).

(iii) Composite columns: The main longitudinal reinforcement of the composite columns consists of structural steel sections or pipes with or without longitudinal bars (Fig.10.21.2c and d).

Out of the three types of columns, the tied columns are mostly common with different shapes of the cross-sections viz. square, rectangular, T, L, cross etc. Helically bound columns are also used for circular or octagonal shapes of cross-sections. Architects prefer circular columns in some specific situations for the functional requirement.

10.21.4 Classification of Columns Based on Loadings

Columns are classified into the three following types based on the loadings:

(i) Columns subjected to axial loads only (concentric), as shown in Fig.10.21.3a.

(ii) Columns subjected to combined axial load and uniaxial bending, as shown in Fig.10.21.3b.

(iii) Columns subjected to combined axial load and bi-axial bending, as shown in Fig.10.21.3c.

Figure 10.21.4 shows the plan view of a reinforced concrete rigid frame having columns and inter-connecting beams in longitudinal and transverse directions. From the knowledge of structural analysis it is well known that the bending moments on the left and right of columns for every longitudinal beam will be comparable as the beam is continuous. Similarly, the bending moments at the two sides of columns for every continuous transverse beam are also comparable. All internal columns will be designed for axial force only. The side columns will have axial forces with uniaxial bending moment, while the four corner columns shall have axial forces with biaxial bending moments.

It is worth mentioning that pure axial forces in the inside columns is a rare case. Due to rigid frame action, lateral loadings and practical aspects of construction, there will be bending moments and horizontal shear in all the inside columns also. Similarly, side columns and corner columns will have the column shear along with the axial force and bending moments in one or both directions, respectively. The effects of shear are usually neglected as the magnitude is very small. Moreover, the presence of longitudinal and transverse reinforcement is sufficient to resist the effect of column shear of comparatively low magnitude.

The effect of some minimum bending moment, however, should be taken into account in the design even if the column is axially loaded. Accordingly, cls. 39.2 and 25.4 of IS 456 prescribes the minimum eccentricity for the design of all columns. In case the actual eccentricity is more than the minimum, that should be considered in the design.

10.21.5 Classification of Columns Based on Slenderness Ratios

Columns are classified into the following two types based on the slenderness ratios:

- (i) Short columns (ii) Slender or long columns

Figure 10.21.5 presents the three modes of failure of columns with different slenderness ratios when loaded axially. In the mode 1, column does not undergo any lateral deformation and collapses due to material failure. This is known as compression failure. Due to the combined effects of axial load and moment a short column may have material failure of mode 2. On the other hand, a slender column subjected to axial load only undergoes deflection due to beam-column effect and may have material failure under the combined action of direct load and bending moment. Such failure is called combined compression and bending failure of mode 2. Mode 3 failure is by elastic instability of very long column even under small load much before the material reaches the yield stresses. This type of failure is known as elastic buckling.

The slenderness ratio of steel column is the ratio of its effective length L_e to its least radius of gyration r . In case of reinforced concrete column, however, IS 456 stipulates the slenderness ratio as the ratio of its effective length L_e to its least lateral dimension. As mentioned earlier in sec. 10.21.2(a), the effective length is different from the unsupported length, the rectangular reinforced concrete column of cross-sectional dimensions b and D shall have two effective lengths in the two directions of b and D . Accordingly, the column may have the possibility of buckling depending on the two values of slenderness ratios as given below:

Slenderness ratio about the major axis = L_{ex}/D

Slenderness ratio about the minor axis = L_{ey}/b

A compression member may be considered as short when both the slenderness ratios L_{ex}/D and L_{ey}/b are less than 12 where L_{ex} : effective length in respect of the major axis, D = depth in respect of the major axis, L_{ey} effective length in respect of the minor axis, and b = width of the member. It shall otherwise be considered as a slender compression member.

Further, it is essential to avoid the mode 3 type of failure of columns so that all columns should have material failure (modes 1 and 2) only. Accordingly, cl. 25.3.1 of IS 456 stipulates the maximum unsupported length between two restraints of a column to sixty times its least lateral dimension. For cantilever columns, when one end of the column is unrestrained, the unsupported length is restricted to $100b^2/D$.

10.21.6 Braced and unbraced columns

It is desirable that the columns do not have to resist any horizontal loads due to wind or earthquake. This can be achieved by bracing the columns as in the case of columns of a water tank or tall buildings (Figs.10.21.6a and b). Lateral tie members for the columns of water tank or shear walls for the columns of tall buildings resist the horizontal forces and these columns are called braced columns. Unbraced columns are supposed to resist the horizontal loads also.

The bracings can be in one or more directions depending on the directions of the lateral loads. It is worth mentioning that the effect of bracing has been taken into account by the IS code in determining the effective lengths of columns (vide Annex E of IS 456).

10.21.7 Longitudinal Reinforcement: The longitudinal reinforcing bars carry the compressive loads along with the concrete.

(a) The minimum amount of steel should be at least 0.8 per cent of the gross cross sectional area of the column required if for any reason the provided area is more than the required area.

(b) The maximum amount of steel should be 4 per cent of the gross cross-sectional area of the column so that it does not exceed 6 per cent when bars from column below have to be lapped with those in the column under consideration.

(c) Four and six are the minimum number of longitudinal bars in rectangular and circular columns, respectively.

(d) The diameter of the longitudinal bars should be at least 12 mm.

(e) Columns having helical reinforcement shall have at least six longitudinal bars within and in contact with the helical reinforcement. The bars shall be placed equidistant around its inner circumference.

(f) The bars shall be spaced not exceeding 300 mm along the periphery of the column.

(g) The amount of reinforcement for pedestal shall be at least 0.15 per cent of the cross-sectional area provided.

10.21.8 Transverse Reinforcement: Transverse reinforcing bars are provided in forms of circular rings, polygonal links (lateral ties) with internal angles not exceeding 135° or helical reinforcement. The transverse reinforcing bars are provided to ensure that every longitudinal bar nearest to the compression face has effective lateral support against buckling.

(a) Transverse reinforcement shall only go round corner and alternate bars if the longitudinal bars are not spaced more than 75 mm on either side (Fig.10.21.7).

(b) Longitudinal bars spaced at a maximum distance of 48 times the diameter of the tie shall be tied by single tie and additional open ties for in between longitudinal bars (Fig.10.21.8).

(c) For longitudinal bars placed in more than one row (Fig.10.21.9): (i) transverse reinforcement is provided for the outer most row in accordance with (a) above, and (ii) no bar of the inner row is closer to the nearest compression face than three times the diameter of the largest bar in the inner row.

(d) For longitudinal bars arranged in a group such that they are not in contact and each group is adequately tied as per (a), (b) or (c) above, as appropriate, the transverse reinforcement for the compression member as a whole may be provided assuming that each group is a single longitudinal bar for determining the pitch and diameter of the transverse reinforcement as given in sec.10.21.9. The diameter of such transverse reinforcement should not, however, exceed 20 mm (Fig.10.21.10).

10.21.9 Pitch and Diameter of Lateral Ties

(a) Pitch: The maximum pitch of transverse reinforcement shall be the least of

the following:

(i) the least lateral dimension of the compression members;
(ii) sixteen times the smallest diameter of the longitudinal reinforcement bar to be tied; and (iii) 300 mm.

(b) Diameter: The diameter of the polygonal links or lateral ties shall be not less than one fourth of the diameter of the largest longitudinal bar, and in no case less than 6 mm.

10.21.10 Helical Reinforcement

(a) Pitch: Helical reinforcement shall be of regular formation with the turns of the helix spaced evenly and its ends shall be anchored properly by providing one and a half extra turns of the spiral bar. The pitch of helical reinforcement shall be determined as given in sec.10.21.9 for all cases except where an increased load on the column is allowed for on the strength of the helical reinforcement. In such cases only, the maximum pitch shall be the lesser of 75 mm and one-sixth of the core diameter of the column, and the minimum pitch shall be the lesser of 25 mm and three times the diameter of the steel bar forming the helix.

(b) Diameter: The diameter of the helical reinforcement shall be as mentioned in sec.10.21.9b.

10.21.11 Assumptions in the Design of Compression Members by Limit State of Collapse

Tied and helically reinforced short and slender columns subjected to axial loadings with or without the combined effects of uniaxial or biaxial bending will be taken up. The assumptions (i) to (v) given in sec.3.4.2 of Lesson 4 for the design of flexural members are also applicable here. Further more, the following are the additional assumptions for the design of compression members (cl. 39.1 of IS 456).

(i) The maximum compressive strain in concrete in axial compression is taken as 0.002.

(ii) The maximum compressive strain at the highly compressed extreme fibre in concrete subjected to axial compression and bending and when there is no tension on the section shall be 0.0035 minus 0.75 times the strain at the least compressed extreme fibre.

The assumptions (i) to (v) of section 3.4.2 of Lesson 4 and (i) and (ii) mentioned above are discussed below with reference to Fig.10.21.11a to c presenting the cross-section and strain diagrams for different location of the neutral axis.

The discussion made in sec. 3.4.2 of Lesson 4 regarding the assumptions (i), (iii), (iv) and (v) are applicable here also. Assumption (ii) of sec.3.4.2 is also

applicable here when kD , the depth of neutral axis from the highly compressed right edge is within the section i.e., $K < 1$. The corresponding strain profile IN in

Fig.10.21.11b is for particular value of P and M such that the maximum compressive strain is 0.0035 at the highly compressed right edge and tensile strain develops at the opposite edge. This strain profile is very much similar to that of a beam in flexure of Lesson 4.

The additional assumption (i) of this section refers to column subjected axial load P only resulting compressive strain of maximum (constant) value of 0.002 and for which the strain profile is EF in Fig.10.21.11b. The neutral axis is at infinity (outside the section).

Extending the assumption of the strain profile IN (Fig.10.21.11b), we can draw another strain profile IH (Fig.10.21.11c) having maximum compressive strain of 0.0035 at the right edge and zero strain at the left edge. This strain profile 1H along with EF are drawn in Fig.10.21.11c to intersect at V. From the two similar triangles EVI and GHI, we have

$EV/GH = 0.0015/0.0035 = 3/7$, which gives

$EV = 3D/7$
(10.2)

The point V, where the two profiles intersect is assumed to act as a fulcrum for the strain profiles when the neutral axis lies outside the section. Another strain profile JK drawn on this figure passing through the fulcrum V and whose neutral axis is outside the section. The maximum compressive strain GJ of this profile is related to the minimum compressive strain HK as explained below.

$GJ = GI - IJ = GI - 0.75 HK$, as we can write IJ in term of HK from two similar triangles JVI and HVK:

$IJ/HK = VENF = 0.75$.

The value of the maximum compressive strain GJ for the profile JK is,

therefore, 0.0035 minus 0.75 times the strain HK on the least compressed edge. This is the assumption (ii) of this section (cl. 39.1b of IS 456).

10.21.12 Minimum Eccentricity: In practical construction, columns are rarely truly concentric. Even a theoretical column loaded axially will have accidental eccentricity due to inaccuracy in construction or variation of materials etc. Accordingly, all axially loaded columns should be designed considering the minimum eccentricity as stipulated in cl. 25.4 of IS 456 and given below (Fig.10.21.3c)

$e_{min} = \max \left(\frac{L}{500} + \frac{D}{30} \right) \text{ or } 20 \text{ mm}$

$e_{min} = \max \left(\frac{L}{500} + \frac{b}{30} \right) \text{ or } 20 \text{ mm}$

where L, D and b are the unsupported length, larger lateral dimension and least lateral dimension, respectively.