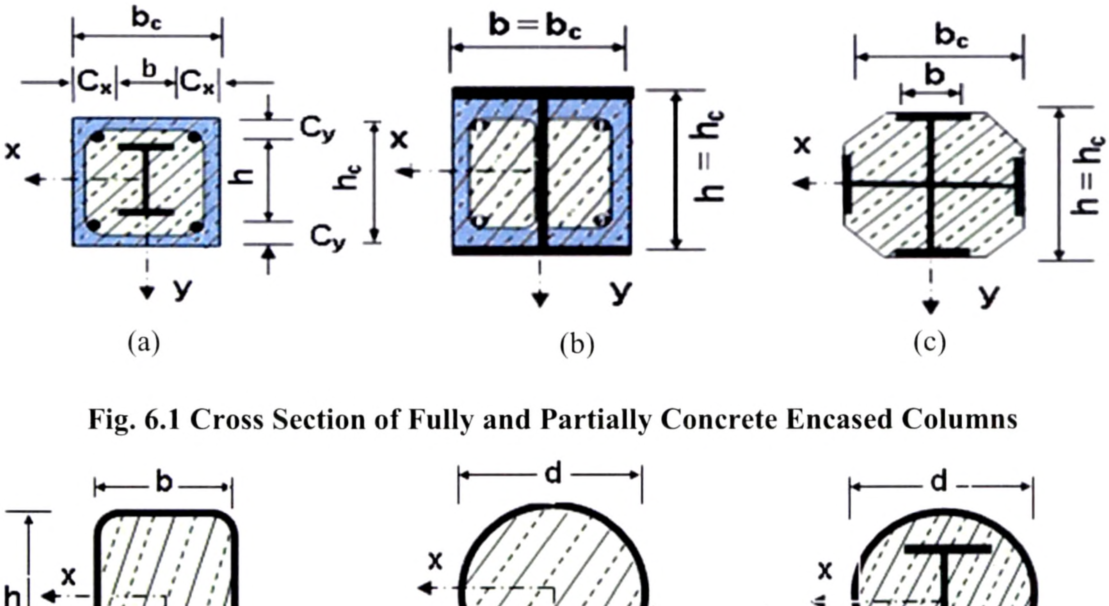
6. DEVELOPMENT OF PROGRAM FOR COMPOSITE COLUMNS

6.1 PREAMBLE

Composite columns may be classified into two principal types:

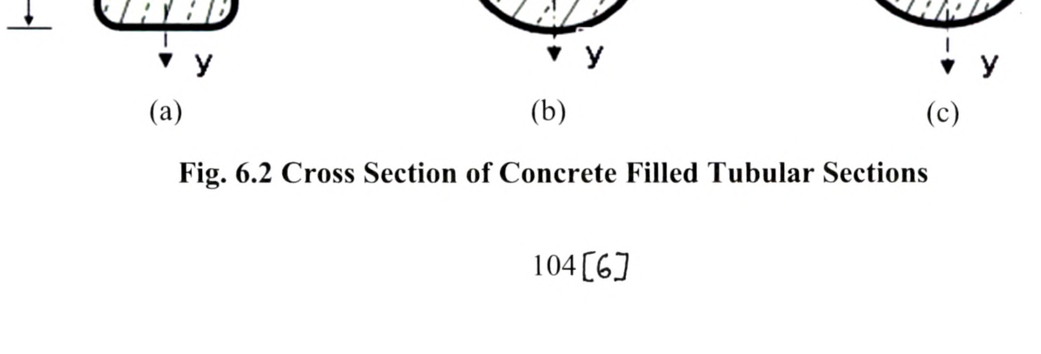
- Open sections partially or fully encased in concrete,
- Concrete-filled hollow steel sections.

Typical cross-sections of composite columns with fully and partially concrete encased steel sections are illustrated in **Fig. 6.1** whereas **Fig. 6.2** shows three typical cross-sections of concrete filled hollow steel sections.



(a) (b) (c)

Fig. 6.1 Cross Section of Fully and Partially Concrete Encased Columns



(a) (b) (c)

Fig. 6.2 Cross Section of Concrete Filled Tubular Sections

In composite construction, the bare steel sections support the initial construction loads, including the weight of structure during construction. Concrete is later cast around the steel section, or filled inside the tubular sections. In the case of concrete-filled hollow sections, the steel provides a permanent formwork to the concrete core. This allows, for example, the steel frame to be erected and the hollow column sections subsequently to be filled with pumped concrete. This leads to appreciable savings in the time and cost of erection. In addition, the confinement provided by the closed steel section allows higher strengths to be attained by the concrete. Creep and shrinkage of concrete, are also generally neglected in the design of concrete-filled tubes, which is not the case for concrete-encased sections. On the other hand, complete encasement of a steel section usually provides enough fire protection to satisfy the most stringent requirements without resorting to other protection systems. For partially encased sections, and for concrete-filled hollow sections, codes of practice on fire resistance require additional reinforcement.

Partially encased sections have the advantage of acting as permanent formwork; the concrete is placed in two stages with the section. In order to ensure adequate force transfer between the steel and concrete it is sometimes necessary to use stud connectors or reinforcement connected directly or indirectly to the metal profile. Another significant advantage of partially encased sections is the fact that, after concreting, some of the steel surfaces remain exposed and can be used for connection to other beams. Thus, concrete and steel are combined in such a fashion that the advantages of both the materials are utilised effectively in composite column. Further, the lighter weight and higher strength of steel permit the use of smaller and lighter foundations.

6.2 CALCULATION METHODS

At present there is no Indian Standard covering composite columns. The method of design largely follows EC4, which incorporate the latest research on composite construction. Any one of the following two methods can be used for the calculation.

The first is a General Method which takes explicit account of both second-order effects and imperfections. This method in particular can be applied to columns of asymmetric cross-section as well as to columns whose section varies with height. It requires the use of numerical computational tools, and can be considered only if suitable software is available.

The second is a Simplified Method [90] which makes use of the European buckling curves for steel columns, and implicitly take account of imperfections.

These two methods are based on the following assumptions:

- There is full interaction between the steel and concrete sections until failure occurs;
- Geometric imperfections and residual stresses are taken into account in the calculation, although this is usually done by using an equivalent initial member imperfection;
- Plane sections remain plane whilst the column deforms.

Here, Simplified Method is considered, because it is applicable to the majority of practical cases.

6.3 LOCAL BUCKLING OF STEEL ELEMENTS

The presence of concrete firmly held in place prevents local buckling of the walls of completely encased steel sections, provided that the concrete cover thickness is adequate. This thickness should not be less than the larger of the following two values:

- 40 mm;
- One sixth of the width b of the flange of the steel section.

This cover, which is intended to prevent premature separation of the concrete, must be laterally reinforced, to protect the encasement against damage from accidental impact and to provide adequate restraint against buckling of the flanges.

For partially encased sections and concrete-filled closed sections, the slenderness of the elements of the steel section must satisfy the following conditions:

- $d/t \leq \epsilon^2$ (concrete-filled circular hollow sections of diameter d and wall thickness t);
- $d/t \leq 52\epsilon$ (concrete-filled rectangular hollow sections of wall depth d and thickness t);
- $b/t_f \leq 44\epsilon$ (partially encased H-sections of flange width b and thickness t_f);

In which $\epsilon = \sqrt{235/f_y}$, where f_y is the characteristic yield strength of the steel section.

6.4 FORCE TRANSFER AT BEAM-COLUMN CONNECTIONS

The forces transmitted from a beam through the beam-column connection must be distributed between the steel and concrete parts of the composite column. The nature of this force transfer from the steel to the concrete depends on the structural details and follows a load path which must be clearly identified. The introduction length p, necessary for full development of the compressive force in the concrete part of the column, is usually less than twice the appropriate transverse dimension d as shown in **Fig. 6.3**, and should not in any event exceed 2.5d.

For the purposes of calculation, it is recommended that the shear resistance at the interface between steel and concrete is not assumed to be greater than the following (indicative) values:

- 0.3 N/mm^2 for sections completely encased in concrete;
- 0.4 N/mm^2 for concrete-filled hollow sections;
- 0.2 N/mm^2 for the flanges of partially encased sections;
- 0.0 N/mm^2 for the webs of partially encased sections.

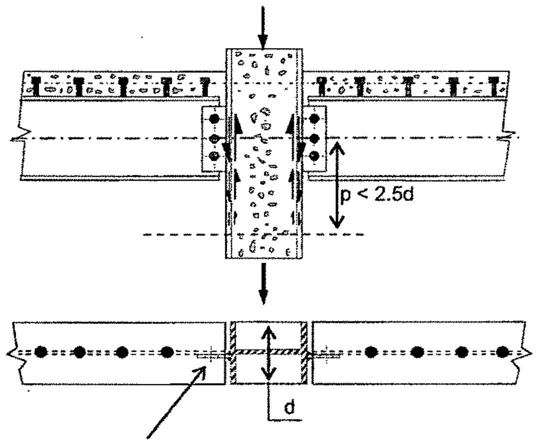


Fig. 6.3 Force Transfer in a Composite Beam-Column Connection

The detailed design of the beam-column connection has a considerable influence on the shear resistance, and the effects of hoop-stress, confinement and friction are intimately linked to the connection layout used. **Figure 6.3** shows a typical beam-column connection, and defines the introduction length p. In the particular case of a concrete-encased composite column for which the bond strength between steel and concrete is insufficient for the transfer to the concrete part to take place within the allowable length, it is possible to use shear connector studs welded to the web of the steel section. It is then possible to take account of the shear resistance P_{Rd} of the studs as an enhancement to the bond between the steel and concrete. This additional bond strength, acting only on the internal faces of the flanges, can be taken as $\mu P_{Rd} / 2$ on each flange. The coefficient μ can initially be taken as 0.5, although its real value depends on the degree of confinement of the concrete between the flanges of the section. This assumption is valid only if the distance between the flanges is less than the values in millimetres **Fig. 6.4**.

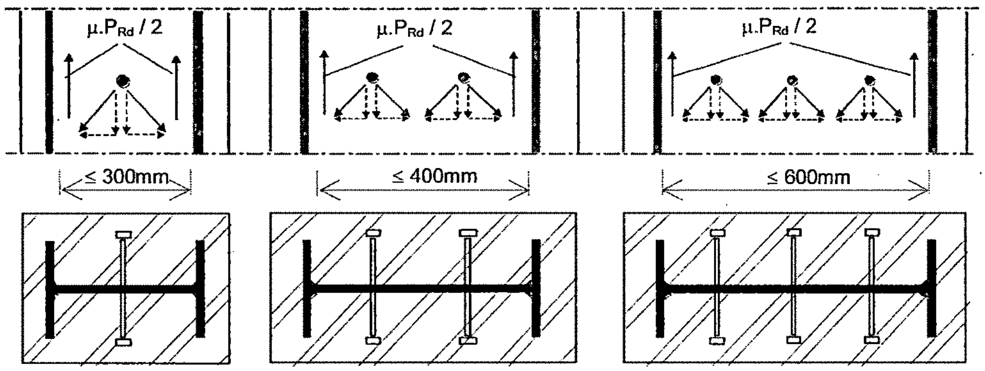


Fig. 6.4 Use of Studs to Enhance Force Transfer in Composite Columns

6.5 DESIGN METHOD

6.5.1 RESISTANCE OF CROSS-SECTION TO COMPRESSION

The plastic compression resistance of a composite cross-section represents the maximum load that can be applied to a short composite column. Concrete filled circular tubular sections exhibit enhanced resistance due to the tri-axial confinement effects. Fully or partially concrete encased steel sections and concrete filled rectangular tubular sections do not achieve such enhancement.

Encased steel sections and concrete filled rectangular/tubular sections

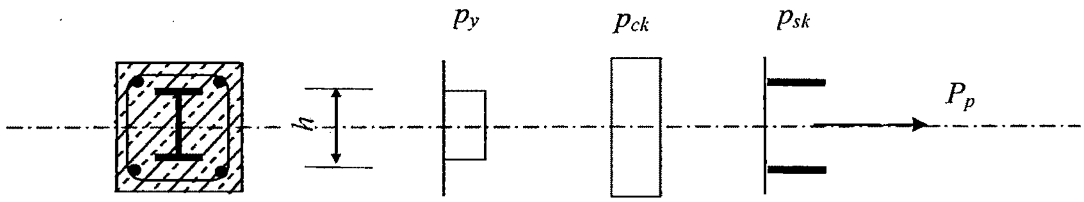


Fig. 6.5 Stress Distribution of Plastic Resistance to Compression

The plastic resistance of an encased steel section or concrete filled rectangular section (i.e. the so-called "squash load") is given by the sum of the resistances of the components as follows:

$$P_p = A_a p_y + A_c p_{ck} + A_s p_{sk} \qquad \text{... (6.1)}$$

where,

A_a, A_c *and* A_s = Areas of steel section, concrete and reinforcing steel respectively,

p_y, p_{ck} *and* p_{sk} = $\frac{f_y}{\gamma_a}, \frac{\alpha_{c(f_{ck})cy}}{\gamma_c}$ *and* $\frac{f_{sk}}{\gamma_s}$,

$f_y, (f_{ck})_{cy}$ *and* f_{sk} = Yield strength of the steel section, the characteristic compressive strength (cylinder) of the concrete, and the yield strength of the reinforcing steel respectively, and

α_c = Strength coefficient for concrete, which is 1.0 for concrete filled tubular sections, and 0.85 for fully or partially concrete encased steel sections.

At this stage it should be pointed out that the Indian Standards for composite construction does not make any specific reference to composite columns. The provisions contained in IS: 456 - 2000 [96] are often invoked for design of composite structures. Extension of IS: 456 - 2000 to composite columns will result in the following equation,

$$P_p = A_s p_y + A_c p_{ck} + A_s p_{sk} \quad \text{... (6.2)}$$

where, $p_y = 0.87 f_y$, $p_{ck} = 0.4(f_{ck})_{cu}$, $p_{sk} = 0.67 f_y$ and $(f_{ck})_{cu}$ is the characteristic compressive strength (cube) of the concrete.

Concrete filled circular tubular sections: Special Provisions

For composite columns using circular tubular sections, there is an increased resistance of concrete due to the confining effect of the circular tubular section.

The eccentricity, e, is defined as follows

$$e = \frac{M}{P} \leq \frac{d}{10} \quad \text{... (6.3)}$$

where, e = Eccentricity, M = Maximum applied design moment (second order effects are ignored) and P = Applied design load.

The plastic compression resistance of concrete filled circular tubular sections is calculated by using two coefficients η_1 and η_2 as given below.

$$P_p = A_a \eta_2 p_y + A_c p_{ck} \left[1 + \eta_1 \frac{t}{d} \frac{f_y}{f_{ck}}\right] + A_s p_{sk} \quad \text{... (6.4)}$$

where, t = Thickness of the circular tubular section,

$$\eta_1 = n_{10} \left[1 - \frac{10_e}{d}\right] \quad \text{... (6.5)}$$

$$\eta_2 = \eta_{20} + (1 - \eta_{20}) \frac{10e}{d} \quad \text{... (6.6)}$$

The basic values η_{10} and η_{20} depend on the non-dimensional slenderness $\bar{\lambda}$, which can be read off from **Table 6.1**. If the eccentricity e exceeds the value $d/10$, or if the non-dimensional slenderness exceeds the value 0.5 then $\eta_1 = 0$ and $\eta_2 = 1.0$.

Table 6.1 Basic Values of η_{10} and η_{20} as Provided in EC4 [$(f_{ck})_{cy}$ = 25 to 55 N/mm^2]

Co-efficient	$\bar{\lambda} = 0.0$	$\bar{\lambda} = 0.1$	$\bar{\lambda} = 0.2$	$\bar{\lambda} = 0.3$	$\bar{\lambda} = 0.4$	$\bar{\lambda} \geq 0.5$
η_{10}	4.90	3.22	1.88	0.88	0.22	0.00
η_{20}	0.75	0.80	0.85	0.90	0.95	1.00

$\bar{\lambda}$ can be found out from the following equation,

$$\bar{\lambda} = \sqrt{\frac{P_{pu}}{P_{cr}}} \qquad \text{... (6.7)}$$

where, P_{pu} = Plastic resistance of the cross-section to compression according to Eqn. 6.2 or Eqn. 6.4 with $\gamma_a = \gamma_c = \gamma_s = 1.0$, and P_{cr} = Elastic buckling load of the column as defined in Eqn. 6.8.

6.5.2 EFFECTIVE ELASTIC FLEXURAL STIFFNESS

Composite columns may fail in buckling and one important parameter for the buckling design of composite columns is its elastic critical buckling load, P_{cr}, which is defined as follows:

$$P_{cr} = \frac{\pi^2 (EI)_e}{l^2} \qquad \text{... (6.8)}$$

Where $(EI)_e$ is the effective elastic flexural stiffness of the composite column, and l is the effective length of the column.

6.5.2.1 Short Term Loading

The effective elastic flexural stiffness, $(EI)_e$, is obtained by adding up the flexural stiffness of the individual components of the cross-section as follows:

$$(EI)_e = E_a I_a + 0.8 E_{cd} I_c + E_s I_s \qquad \text{... (6.9)}$$

Where I_a, I_c and I_s are the second moments of area of the steel section, concrete (assumed uncracked) and the reinforcement about the axis of bending considered respectively, E_a and E_s are the moduli of elasticity of the steel section and the reinforcement, and $0.8E_{cd}I_c$ is the effective stiffness of the concrete; the factor 0.8 is an empirical multiplier determined by a calibration exercise to give good agreement with test result. Note that I_c is the moment of inertia about the centroid of the uncracked column section.

$$E_{cd} = E_{cm}/\gamma_c \qquad \text{... (6.10)}$$

E_{cm} is the secant modulus of the concrete and γ_c is reduced to 1.35 for the determination of the effective stiffness of concrete according to Eurocode 2.

6.5.2.2 Long Term Loading

For slender columns under long-term loading, the creep and shrinkage of concrete will cause a reduction in the effective elastic flexural stiffness of the column, thereby reducing the buckling resistance. However, this effect is significant only for slender columns. As a simple rule, the effect of long term loading should be considered if,

- The buckling length to depth ratio of a composite column exceeds 15.
- The eccentricity of loading is less than twice the cross-section dimension.
- The non-dimensional slenderness $\bar{\lambda}$ is more than the limiting values given in **Table 6.2.**

Table 6.2 Limiting Values of $\bar{\lambda}$ for Long Term Loading

Type of Column	Braced Non-sway systems	Unbraced and/or sway systems
Concrete encased cross-sections	0.8	0.5
Concrete filled cross sections	0.8/1- δ	0.5/1- δ

Here, δ is the steel contribution ratio and is given by [18],

$$\delta = A_a \times p_y / P_p \qquad \text{... (6.11)}$$

However, when $\bar{\lambda}$ limits given by **Table 6.2**, the effect of creep and shrinkage of concrete should be allowed for by employing the modulus of elasticity of the concrete $E_{c\infty}$ instead of E_{cd} in Eqn. 6.12, which is formulated as

$$E_{c\infty} = E_{cd}\left[1 - \frac{0.5P_d}{P}\right] \qquad \text{... (6.12)}$$

where, P and P_d are the applied design load and a part of it permanently acting on the column.

The effect of long-term loading may be ignored for concrete filled tubular sections with $\bar{\lambda} \leq 2.0$ provided that δ is greater than 0.6 for braced (or non-sway) columns, and *0.75* for unbraced (and/or sway) columns.

6.5.3 RESISTANCE OF MEMBERS TO AXIAL COMPRESSION

For each of the principal axes of the column, the designer should check,

$$P \leq \chi P_p \qquad \text{... (6.13)}$$

where, P_p = Plastic resistance to compression of the cross-section, from Eqn. 6.2 or Eqn. 6.4, and χ = Reduction factor due to column buckling and is a function of the non-dimensional slenderness of the composite column.

The European buckling curves given in **Fig. 6.6** are proposed to be used for composite columns. They are selected according to the types of the steel sections and the axis of bending as follows.

Curve a = For concrete filled tubular sections

Curve b = For fully or partially concrete encased I-sections buckling about the strong axis of the steel sections (x-x axis).

Curve c = For fully and partially concrete encased I-sections buckling about the weak axis of the steel sections (y-y axis).

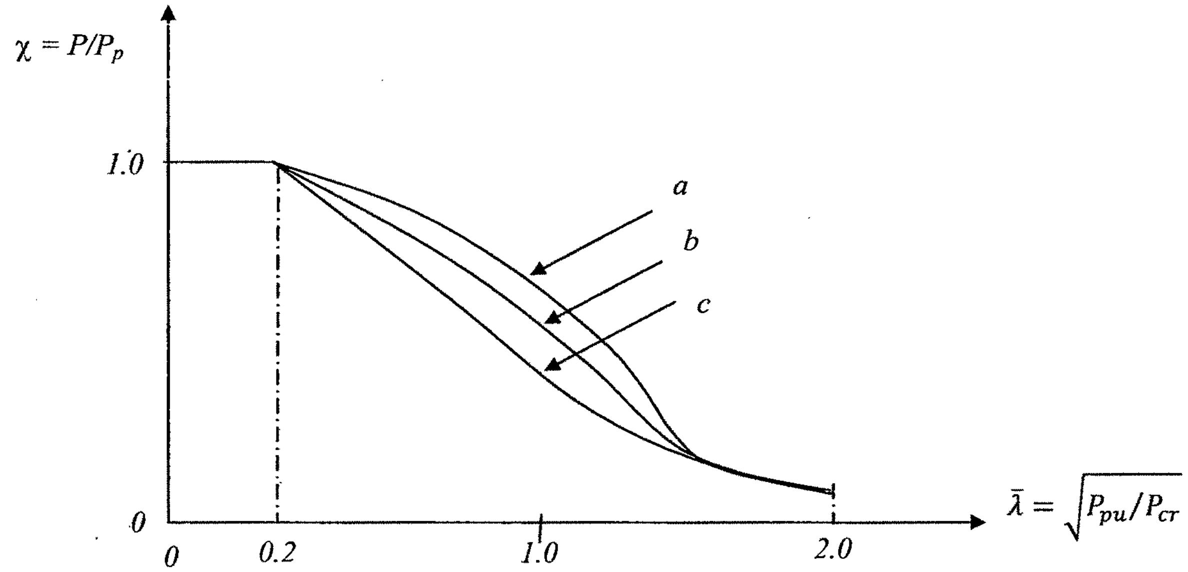


Fig. 6.6 European Buckling Curves

These curves can also be described mathematically as follows:

$$\chi = \frac{1}{\emptyset + \sqrt{\emptyset^2 - \bar{\lambda}^2}} \quad but \ \chi \leq 1.0 \qquad \text{... (6.14)}$$

$$\emptyset = 0.5\left[1 + \alpha(\bar{\lambda} - 0.2) + \bar{\lambda}^2\right] \qquad \text{... (6.15)}$$

where, the factor α allows for different levels of imperfections and residual stresses in the columns corresponding to curves a, b, and c. **Table 6.3** [8] gives the value of α for each buckling curve. Note that the second order moment due to imperfection has been incorporated in the method by using multiple buckling curves; no additional considerations are necessary.

Using the values of determined $\bar{\lambda}$ from Eq. 6.7 and the reduction factor χ calculated from Eq. 6.13, the design buckling resistance of the composite column to compression, P_b or χP_p may thus be evaluated.

Table 6.3 Imperfection Factor α for the Buckling Curves

European buckling curve	*a*	*b*	*c*
Imperfection factor α	0.21	0.34	0.49

6.5.4 Combined Compression and Uni-Axial Bending

To design a composite column under combined compression and bending, it is first isolated from the framework, and the end moments which result from the analysis of the system as a whole are taken to act on the column under consideration. Internal moments and forces within the column length are determined from the structural consideration of end moments, axial and transverse loads. For each axis of symmetry, the buckling resistance to compression is first checked with the relevant non-dimensional slenderness of the composite column. Thereafter the moment of resistance of the composite cross-section is checked in the presence of applied moment about each axis, e.g. x-x and y-y axis, with the relevant non-dimensional slenderness values of the composite column. For slender columns, both the effects of long term loading and the second order effects are included. The design method used here is an extension of the Simplified Design Method for the design of steel-concrete composite column under axial load.

6.5.4.1 Interaction Curve for Compression and Uni-Axial Bending

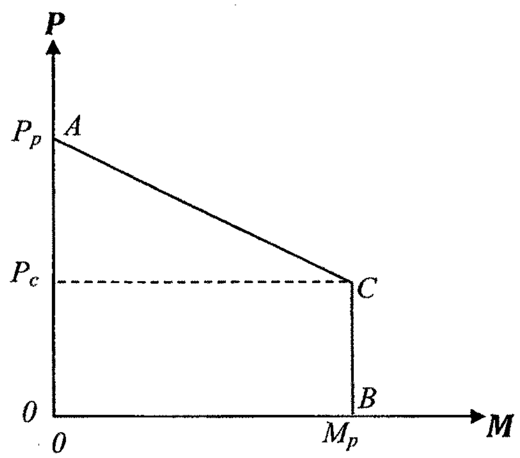


Fig. 6.5 Interaction Curve For Compression and Uni-Axial Bending

The resistance of the composite column to combined compression and bending is determined using an interaction curve shown in **Fig. 6.5** drawn using simplified design method suggested in the UK National Application Document (NAD) for EC 4. This neglects the increase in moment capacity beyond M_P (Under relatively low axial compressive loads).

Figure 6.6 shows the stress distributions in the cross-section of a concrete filled rectangular tubular section at each point A, B and C of the interaction curve given in **Fig. 6.5.** It is important to note that

- **Point** *A* marks the plastic resistance of the cross-section to compression (at this point the bending moment is zero).

$$P_A = P_p = \frac{A_a f_y}{\gamma_a} + \frac{\alpha_c A_c (f_{ck})_{cy}}{\gamma_c} + \frac{A_s f_{sk}}{\gamma_s} \qquad \text{... (6.16)}$$

$$M_A = 0 \qquad \text{... (6.17)}$$

- **Point B** corresponds to the plastic moment resistance of the cross-section (the axial compression is zero).

$$P_B = 0 \qquad \text{... (6.18)}$$

$$M_B = M_p = p_y(Z_{pa} - Z_{pan}) + p_{sk}(Z_{ps} - Z_{psn}) + p_{ck}(Z_{pc} - Z_{pcn}) \qquad \text{... (6.19)}$$

where, Z_{ps}, Z_{pa}, and Z_{pc} = Plastic section moduli of the reinforcement, steel section and concrete about their own centroids respectively, and Z_{psn}, Z_{pan} and Z_{pcn} = Plastic section moduli of the reinforcement, steel section and concrete about neutral axis respectively.

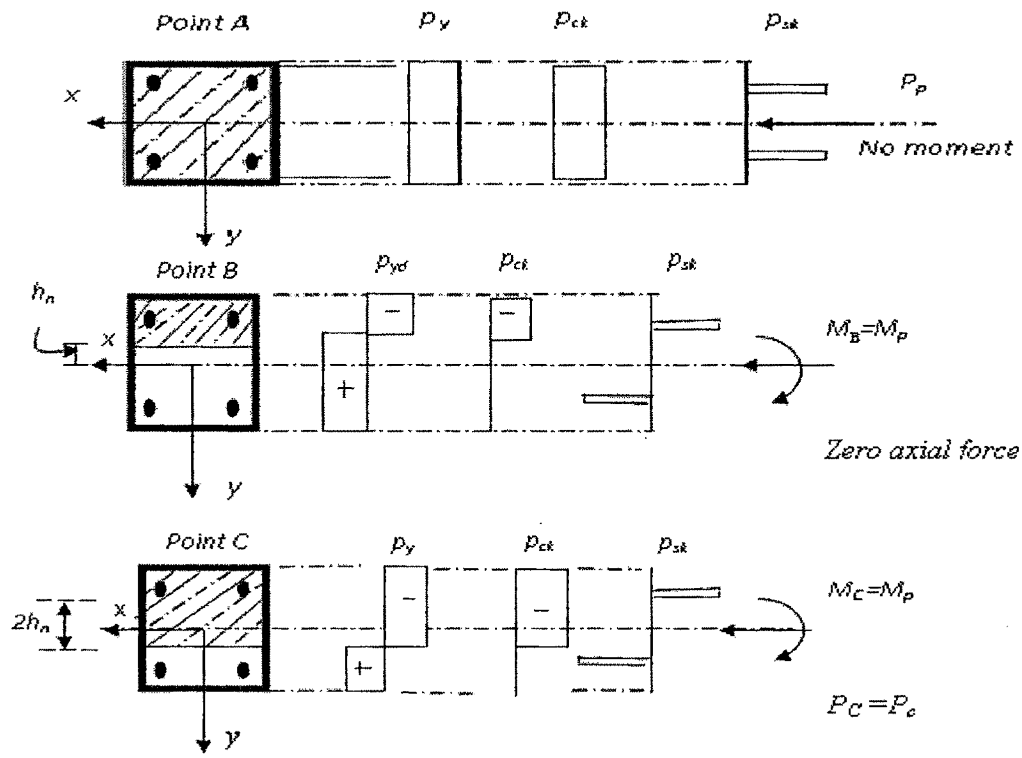


Fig. 6.6 Stress Distributions for the Points of the Interaction Curve

- **Point** *C*, the compressive and the moment resistances of the column are given by;

$$Pc = P_c = A_c P_{ck} \qquad ...(6.20)$$

$$M_c = M_p \qquad ...(6.21)$$

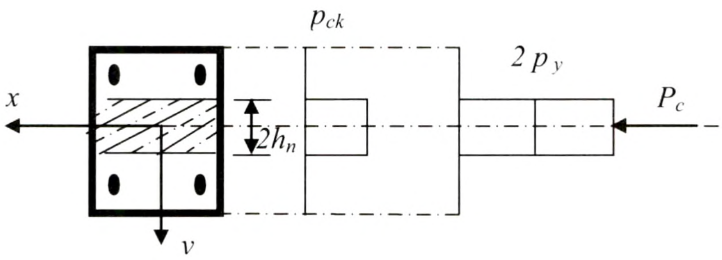


Fig. 6.7 Variation in the Neutral Axis Positions

It is important to note that the positions of the neutral axis for points B and C, h_n, can be determined from the difference in stresses at points B and C. The resulting axial forces, which are dependent on the position of the neutral axis of the cross-section, h_n, can easily be determined as shown in **Fig. 6.7**. The sum of these forces is equal to P_c. This calculation enables the equation defining h_n to be determined, which is different for various types of sections.

I. For Concrete Encased Steel Sections:

MAJOR AXIS BENDING (**Fig. 6.8**)

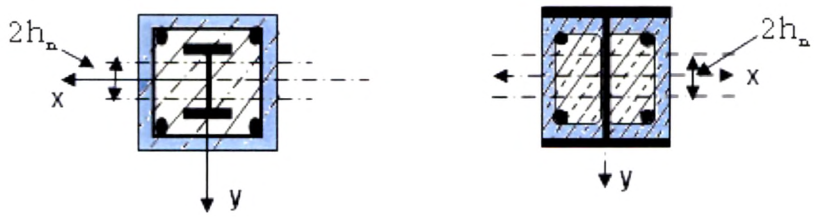


Fig. 6.8 h_n For Concrete Encase Steel Sections (Major Axis Bending)

i. Neutral axis in the web: $h_n \leq [\, h/2 - t_f \,]$

$$h_n = \frac{A_c p_{ck} - A_s'(2p_{sk} - p_{ck})}{2b_c p_{ck} + 2t_w(2p_y - p_{ck})} \qquad ...(6.22)$$

ii. Neutral axis in the flange: $[\, h/2 - t_f \leq h_n \leq h/2 \,]$

$$h_n = \frac{A_c p_{ck} - A_s'(2p_{sk} - p_{ck}) + (b - t_w)(h - 2t_f)(2p_y - p_{ck})}{2b_c p_{ck} + 2b(2p_y - p_{ck})} \qquad ...(6.23)$$

iii. Neutral axis outside the steel section: $h/2 \leq h_n \leq h_c/2$

$$h_n = \frac{A_c p_{ck} - A'_s(2p_{sk} - p_{ck}) + A_a(2p_y - p_{ck})}{2b_c p_{ck}} \quad \ldots (6.24)$$

<u>MINOR AXIS BENDING</u> **(Fig. 6.9)**

i. Neutral axis in the web: $h_n \leq t_w/2$

$$h_n = \frac{A_c p_{ck} - A'_s(2p_{sk} - p_{ck})}{2h_c p_{ck} + 2h(2p_y - p_{ck})} \quad \ldots (6.25)$$

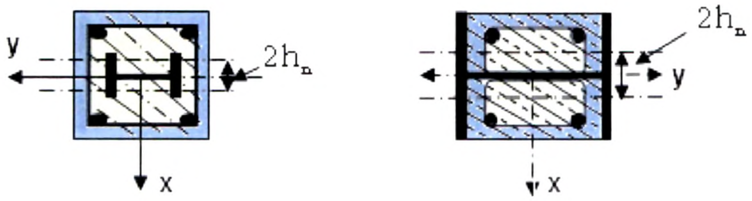


Fig. 6.9 h_n For Concrete Encased Steel Sections (Minor Axis Bending)

ii. Neutral axis in the flange: $t_w/2 < h_n < b/2$

$$h_n = \frac{A_c p_{ck} - A'_s(2p_{sk} - p_{ck}) + t_w(2t_f - h)(2p_y - p_{ck})}{2h_c p_{ck} + 4t_f(2p_y - p_{ck})} \quad \ldots (6.26)$$

iii. Neutral axis outside the steel section: $b/2 \leq h_n \leq b_c/2$

$$h_n = \frac{A_c p_{ck} - A'_s(2p_{sk} - p_{ck}) + A_a(2p_y - p_{ck})}{2h_c p_{ck}} \quad \ldots (6.27)$$

Where, A'_s is the sum of the reinforcement area within the region of $2h_n$.

II. For Concrete Filled Tubular Sections:

<u>MAJOR AXIS BENDING</u> **(Fig. 6.10)**

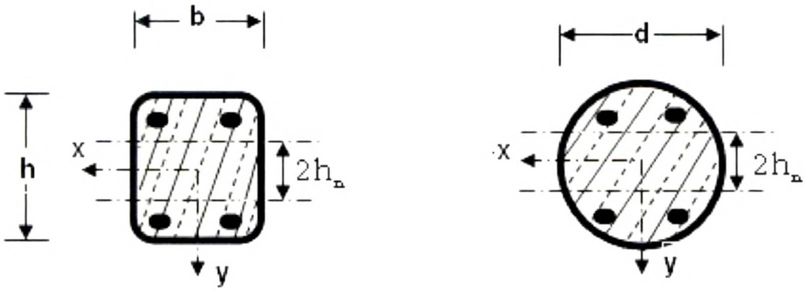


Fig. 6.10 h_n For Concrete Filled Tubular Sections (Major Axis Bending)

$$h_n = \frac{A_c p_{ck} - A'_s(2p_{sk} - p_{ck})}{2b_c p_{ck} + 4t(2p_y - p_{ck})} \quad \text{... (6.28)}$$

- For circular tubular section, $b_c = d$.
- For minor axis bending the same equations can be used by interchanging h and b as well as the subscripts x and y.

6.5.5 Resistance of Members under Combined Compression and Uni-axial Bending

The design checks are carried out in the following stages:

(1) The resistance of the composite column under axial load is determined in the absence of bending, which is given by χP_p.

(2) The moment of resistance of the composite column is then checked with the relevant non-dimensional slenderness, in the plane of the applied moment. As mentioned before, the initial imperfections of columns have been incorporated and no additional consideration of geometrical imperfections is necessary.

The design is adequate when the following condition is satisfied

$$M \leq 0.9\mu M_p \quad \text{... (6.29)}$$

where, M = Design bending moment, μ = Moment resistance ratio obtained from the interaction curve, and M_p = Plastic moment resistance of the composite cross section.

In accordance with the UK NAD, the moment resistance ratio μ for a composite column under combined compression and uni-axial bending is evaluated as follows:

$$\mu = \frac{(\chi - \chi_d)}{(1 - \chi_c)\chi} \quad \text{... (6.30)}$$

$$= 1 - \frac{(\chi - \chi_d)}{(1 - \chi_c)\chi}\ , when \quad \chi_d < \chi_c \quad \text{... (6.31)}$$

where,

χ_c = Axial resistance ratio due to the concrete, (P_c/P_p),

χ_d = Design axial resistance ratio, (P/P_p) and

χ = Reduction factor due to column buckling.

6.5.6 Combined Compression And Bi-Axial Bending

For the design of a composite column under combined compression and bi-axial bending, the axial resistance of the column in the presence of bending moment for each axis has to be evaluated separately. Thereafter the moment resistance of the composite column is checked in the presence of applied moment about each axis, with the relevant non-dimensional slenderness of the composite column. Imperfections have to be considered only for that axis along which the failure is more likely. If it is not evident which plane is more critical, checks should be made for both the axes.

The moment resistance ratios μ_x and μ_y for both the axes are evaluated as given below

$$\mu_x = \frac{(\chi_x - \chi_d)}{(1-\chi_c)\chi_x} \qquad when\ \chi_d \geq \chi \qquad ...(6.32)$$

$$= \frac{(\chi_x - \chi_d)}{(1-\chi_c)\chi_x} \qquad when\ \chi_d < \chi_c \qquad ...(6.33)$$

$$\mu_y = \frac{(\chi_y - \chi_d)}{(1-\chi_c)\chi_y} \qquad when\ \chi_d \geq \chi_c \qquad ...(6.34)$$

$$= \frac{(1-\chi_y)\chi_d}{(1-\chi_c)\chi_y} \qquad when\ \chi_d < \chi_c \qquad ...(6.35)$$

where, χ_x and χ_y = reduction factors for buckling in the x and y directions respectively.

In addition to the two conditions given by Eqs. 6.36 and 6.37, the interaction of the moments must also be checked using moment interaction curve as shown in **Fig. 6.11**. The linear interaction curve is cut off at $0.9\mu_x$ and $0.9\mu_y$. The design moments, M_x and M_y related to the respective plastic moment resistances must lie within the moment interaction curve.

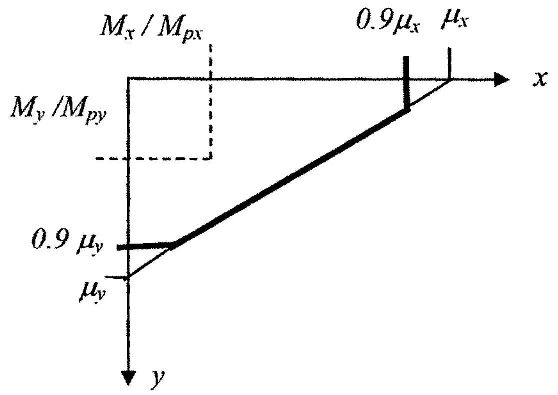


Fig. 6.11 Moment Interaction Curve for Bi-Axial Bending

Hence the three conditions to be satisfied are

$$\frac{M_x}{\mu_x M_{px}} \leq 0.9 \qquad ...(6.36)$$

$$\frac{M_y}{\mu_y M_{py}} \leq 0.9 \qquad ...(6.37)$$

$$\frac{M_x}{\mu_x M_{px}} + \frac{M_y}{\mu_y M_{py}} \leq 1.0 \qquad ...(6.38)$$

When the effect of geometric imperfections is not considered, the moment resistance ratio is evaluated as given below:

$$\mu = \frac{(1-\chi_d)}{(1-\chi_c} \qquad when\ \chi_d > \chi_c \qquad ...(6.39)$$

$$= 1.0 \qquad when\ \chi_d \leq \chi_c \qquad ...(6.40)$$

6.6 ILLUSTRATIVE EXAMPLE

Check adequacy of the concrete encased composite section shown in **Fig. 6.12** for bi-axial bending.

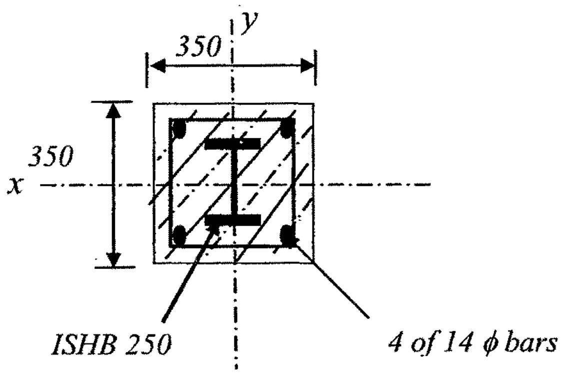


Fig. 6.12 Cross Section of Composite Column

Details of the Section

Column dimension	= 350 × 350 × 3000
Concrete grade	= M30
Steel section	= ISHB 250
Steel reinforcement	= Fe 415, 4 Nos. of 14 mm dia. bar
Design axial load	= 1500 kN
Design B.M. about x-x axis	= 180 kNm
Design B.M. about y-y axis	= 120 kNm

List Material Properties

Structural steel

Steel section ISHB 250

Nominal yield strength f_y = 250 N/mm^2

Modulus of elasticity E_a= 200 kN/mm^2

Concrete

Concrete grade M30

Characteristic strength $(f_{ck})_{cu}$ = 30 N/mm^2

Secant modulus of elasticity for short term loading, E_{cm} = 31220 N/mm^2

Reinforcing steel

Steel grade Fe 415

Characteristic strength f_{sk}= 415 N/mm^2

Modulus of elasticity E_s = 200 kN/mm^2

Partial safety factors

$\gamma_a = 1.15 \quad \gamma_c = 1.5 \quad \gamma_z = 1.15$

List Section Properties of the Given Section

Steel section

A_a = 6971 mm^2 $\quad$ I_{ax} = 79.8 x 10^6 mm^4

t_f = 9.7 mm $\quad$ I_{ay} = 20.1 x 10^6 mm^4

h = 250 mm $\quad$ Z_{px} = 699.8 x 10^3 mm^3

t_w = 8.8 mm $\quad$ Z_{py} =307.6 x 10^3 mm^3

Reinforcing steel

4 bars of 14 mm dia, A_s = 616 mm^2

Concrete

$$A_c = A_{gross} - A_a - A_s$$

$$= 350 \times 350 - 6971 - 616 = 114913 \text{ mm}^2$$

Design Checks

Plastic resistance of the section

$$P_p = A_a f_y/\gamma_a + \alpha_c A_c (f_{ck})_{cy}/\gamma_c + A_s f_{sk}/\gamma_s$$

$$P_p = A_a\, f_y/\gamma_a + \alpha_c A_c\, 0.80 \times (f_{ck})_{cu}/\gamma_c + A_s\, f_{sk}/\gamma_s$$

$= [6971 \times 250/1.15 + 0.85 \times 114913 \times 25/1.5 + 616 \times 415/1.15]/1000$

$= 3366$ kN

Effective elastic flexural stiffness of the section for short term loading

About the major axis

$$(EI)_{ex} = E_a I_{ax} + 0.8\, E_{cd} I_{cx} + E_s I_{sx}$$

$I_{ax} = 79.8 \times 106$ mm4

$I_{sx} = Ah^2$

$= 616 \times [350/2 - 25 - 7]^2$

$= 12.6 \times 10^6\ mm^4$

$I_{cx} = (350)4/12 - [79.8 + 12.6] \times 106$

$= 1158 \times 10^6\ mm^4$

$(EI)_{ex} = 2.0 \times 10^5 \times 79.8 \times 10^6 + 0.8 \times 23125 \times 1158 \times 10^6 + 2.0 \times 10^5 \times 12.6 \times 10^6$

$= 39.4 \times 10^{12}\ N\ mm^2$

About minor axis

$(EI)_{ey} = 2.0 \times 10^5 \times 20.1 \times 10^6 + 0.8 \times 23125 \times 1217.8 \times 10^6 + 2.0 \times 10^5 \times 12.6 \times 10^6$

$= 28.5 \times 10^{12}\ N\ mm^2$

Non Dimensonal Slenderness

$$(P_{cr})_x = \frac{\pi^2 (EI)_{ex}}{l^2} = \frac{\pi^2 \times 39.4 \times 10^{12}}{(3000)^2} = 43207 kN$$

$$\bar{\lambda} = \left(P_{pu}/P_{cr}\right)^{1/2}$$

Value of P_{pu}:

$$P_{pu} = A_a f_y + \alpha_c A_c (f_{ck})_{cu} + A_s f_{sk}$$

$$P_{pu} = A_a f_y + \alpha_c A_c 0.8 \times (f_{ck})_{cu} + A_s f_{sk}$$

$= (6971 \times 250 + 0.85 \times 114913 \times 25 + 415 \times 616)/1000 = 4440\ kN$

$$(P_{cr})_y = \frac{\pi^2 \times 39.4 \times 10^{12}}{(3000)^2} = 31254 kN$$

$\overline{\lambda_x} = (44.4 / 432.07)$ ½ $= 0.320$

$\overline{\lambda_y} = (44.4 / 312.54)$ ½ $= 0.377$

Check for the effect of long term loading

The effect of long term loading can be neglected if the following conditions are satisfied:

- Eccentricity, e given by

  $e = M\ /\ P \geq 2$ times the cross section dimension in the plane of bending considered)

  $e_x\ =\ 180\ /1500 = 0.12 < 2(0.350)$

  $e_y\ =\ 120/1500\ =\ 0.08 < 2\ (0.350)$

- $\overline{\lambda}\ < 0.8$

Since condition is satisfied, the influence of creep and shrinkage on the ultimate load need not be considered.

Resistance of the composite column under axial compression

Design against axial compression is satisfied if following condition is satisfied:

$P < \chi\, P_p$

Here, P =*1500* kN, P_p =3366 kN and χ = reduction factor for column buckling

χ values:

About major axis

$\alpha_x = 0.34$

$\phi_x = 0.5\ [1\ +\ \alpha_x(\overline{\lambda_x} - 0.2)\ +\ \overline{\lambda_x^2}]$

$= 0.5\ [1\ +\ 0.34(0.320 - 0.2)\ +\ (0.320)^2]\ =\ 0.572$

$\chi_x = 1/\left\{\phi + (\phi^2 - \overline{\lambda_x^2})^{1/2}\right\}$

$=\ 1\ /\ \{0.572\ +\ [(0.572)^2 - (0.320)^2]^{1/2}\} = 0.956$

$\chi_x\, P_{px} > P$

$0.956 \times\ 3366\ \ =\ 3218\ kN\ \ >\ P\ (= 1500\ kN)$

About minor axis

$\alpha_y\ =\ 0.49$

$\phi_y\ =\ 0.5\ [1\ +\ 0.49(0.377 - 0.2)\ +\ (0.377)^2 = 0.61$

$\chi_y\ =\ 1\ /\ \{0.61\ +\ \left[(0.61)^2 - (0.377)^2\right]^{1/2}\}\ = 0.918$

$\chi_y\, P_{py} > P$

$0.918 \times\ 3366\ \ =\ 3090\ kN\ \ >\ P\ (= 1500\ kN)$

$\therefore$ The design is OK for axial compression.

Check for second order effects

Isolated non – sway columns need not be checked for second order effects if:

$P/(P_{cr})_x \leq 0.1$ for major axis bending.

$1500/43207 = 0.035 \leq 0.1$

$P/(P_{cr})_y \leq 0.1$ for minor axis bending.

$1500/31254 = 0.048 \leq 0.1$

∴ Check for second order effects is not necessary

Resistance of the composite column under axial compression and bi-axial bending

Compressive resistance of concrete, $P_c = A_c p_{ck} = 1628$ kN

About Major axis

Plastic section modulus of the reinforcement

$Z_{ps} = 4(\pi/4 \times 14^2) \times (350/2\text{-}25\text{-}14/2) = 88 \times 10^3 \text{ mm}^3$

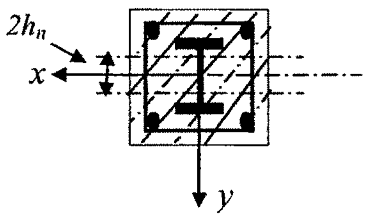
Plastic section modulus of the steel section

$Z_{pa} = 699.8 \times 10^3 \text{ mm}^3$

Plastic section modulus of the concrete

$Z_{pc} = b_c h_c^2/4 - Z_{ps} - Z_{pa} = (350)^3/4 - 88 \times 10^3 - 699.765 \times 10^3 = 9931 \times 10^3 \text{ mm}^3$

Check that the position of neutral axis is in the web



$$h_n = \frac{A_c p_{ck} - A_s'(2p_{sk} - p_{ck})}{2b_c p_{ck} + 2t_w(2p_y - p_{ck})}$$

$$= \frac{114913 \times \frac{0.85 \times 25}{1.5}}{2 \times 350 \times \frac{0.85 \times 25}{1.5} + 2 \times 8.8 \times (2 \times \frac{250}{1.15} - \frac{0.85 \times 25}{1.5})}$$

$$= 93.99\,mm < \left(\frac{h}{2} - t_f\right) = \left(\frac{250}{2} - 9.7\right) = 115.3\,mm.$$

The neutral axis is in the web.

$A'_s = 0$ as there is no reinforcement within the region of the steel web

<u>Section modulus about neutral axis</u>

Z_{psn} = 0 (As there is no reinforcement within the region of 2h_n from the middle line of the cross section)

$Z_{pan} = t_w h_n^2 = 8.8 \times (93.99)^2 = 77740.3$ mm^3

$Z_{pcn} = b_c h_n^2 - Z_{psn} - Z_{pan} = 350\,(93.99)^2 - 77740.3 = 3014.2 \times 10^3$ mm^3

Plastic moment resistance of section

$M_p = p_y\,(Z_{pa} - Z_{pan}) + 0.5\,p_{ck}\,(Z_{pc} - Z_{pcn}) + p_{sk}\,(Z_{ps} - Z_{psn})$

$= 217.4\,(699800 - 77740.3) + 0.5 \times 0.85 \times 25/1.5\,(9931000 - 3014200) + 361\,(88 \times 1000)$

= 216 kNm

<u>About minor axis</u>

Plastic section modulus of the reinforcement

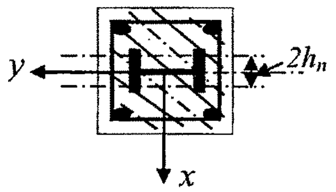
$Z_{ps} = 4(\pi / 4 \times 14^2) \times (350/2 - 25 - 14/2) = 88 \times 10^3$ mm^3

Plastic section modulus of the steel section

$Z_{pa} = 307.6 \times 10^3$ mm^3

Plastic section modulus of the concrete

$Z_{pc} = b_c h_c^2 / 4 - Z_{ps} - Z_{pa} = (350)^3/4 - 88 \times 10^3 - 307.6 \times 10^3 = 10323 \times 10^3$ mm^3



$$h_n = \frac{A_c p_{ck} - A_s^t(2p_{sk} - p_{ck}) + t_w(2t_f - h)(2p_y - p_{ck})}{2h_c p_{ck} + 4t_f(2p_y - p_{ck})}$$

$$= \frac{114913 \times 14.2 + 8.8(2 \times 9.7 - 250)(2 \times 218 - 14.2)}{2 \times 350 \times 14.2 + 4 \times 9.7(2 \times 218 - 14.2)}$$

$$= 29.5\,mm\left({}^{t_w}/_2 < h_n < {}^{b}/_2\right) = {}^{8.8}/_2 < h_n < {}^{250}/_2$$

A'_s= 0 as there is no reinforcement within the region of the steel web

Section modulus about neutral axis

Z_{psn} =0 (As there is no reinforcement within the region of $2h_n$ from the middle line of the cross section).

$Z_{pan} = 2t_f h_n^2+(h-2t_f)/4\times t_w^2$

$= 2(9.7)(29.5)^2 +[\{ 250-2(9.7)\} /4]\times 8.8^2$

$=21.3\times 10^3$ mm^3

$Z_{pcn} = h_c h_n^2 - Z_{psn} - Z_{pan}$

$= 350 (29.5)^2 - 21.3\times 10^3$

$= 283.3 \times 10^3$ mm^3

$M_{py} = p_y (Z_{pa} - Z_{pan}) + 0.5 p_{ck} (Z_{pc} - Z_{pcn}) + p_{sk} (Z_{ps}- Z_{psn})$

$= 217.4 (307.589 - 21.3)\times 10^3 + 0.5 \times 14.2 \times (10323 - 283.3)\times 10^3 + 361 (88 \times 1000)$

=165 kNm

Check of column resistance against combined compression and bi-axial bending

The design against combined compression and bi-axial bending is adequate if following conditions are satisfied:

i. $M \leq 0.9 \mu M_P$

About major axis

M_x = 180 kNm

M_{px} = 216 kNm

μ_x = moment resistance ratio

$= 1- \{(1 - \chi_x) \chi_d\}/\{(1 - \chi_c) \chi_x\}$

$= 1- \{(1 - 0.956) 0.446\}/\{(1 - 0.484) 0.956\}$

= 0.960

$\therefore M_x < 0.9 \mu_x M_{px}$

$< 0.9 (0.960) \times (216) = 187$ kNm

About minor axis

M_y = 120 kNm

M_{py} =165 kNm

$\mu_y = 1- \{(1 - \chi_y) \chi_d\}/\{(1 - \chi_c) \chi_y\}$

$= 1- \{(1 - 0.918) 0.446\}/\{(1 - 0.448) 0.918\}$

= 0.928

$\therefore$ $M_y < 0.9\ \mu_y\ M_{py}$

$< 0.9\ (0.928) * (165)$

<138 kNm (M_y=120 kN)

ii. $$\frac{M_x}{\mu_x M_{px}} + \frac{M_y}{\mu_y M_{py}} \leq 1.0$$

$$\frac{180}{0.96 \times 216} + \frac{120}{0.928 \times 165} > 1.0$$

Since design check is not satisfied, the composite column is not acceptable.

6.7 Programs for Composite Columns

Using the features of the VB.Net, a program is developed for the design of composite columns. Selection menu for axially loaded, uniaxially loaded and biaxially loaded column is depicted in **Fig. 6.13** whereas **Fig. 6.14** shows the form in which data for an example of a design of biaxially loaded composite column is shown. Steel table is also interfaced with the software as shown in **Fig. 6.15.** After the selection of a particular section, various checks are carried out by the software according to the code. Software also checks whether the secondary effect and long term loading effects are required to be considered or not as shown in **Figs. 6.16** and **6.17.** After that, depth of neutral axis is calculated. This is required for the calculation of the plastic section modulus and finally the plastic moment resistance of the section for adequacy check as shown in **Figs. 6.18** and **6.19** respectively.

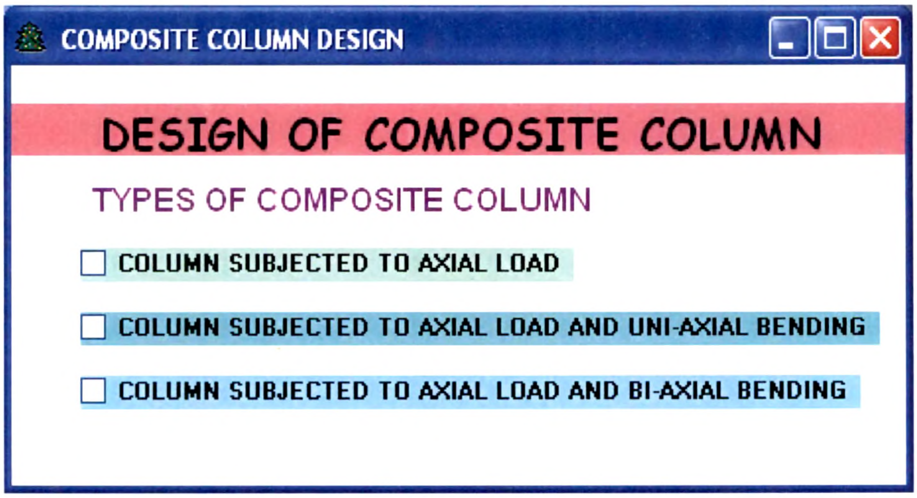


Fig. 6.13 Form for Selection of Type of Loading

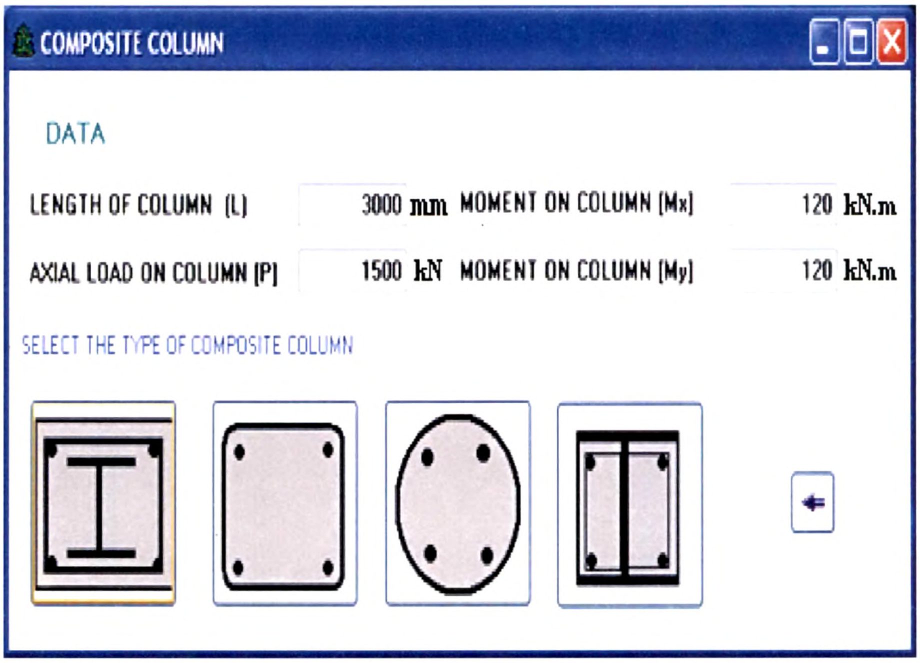


Fig 6.14 Form for Entering the Data for the Design of Section

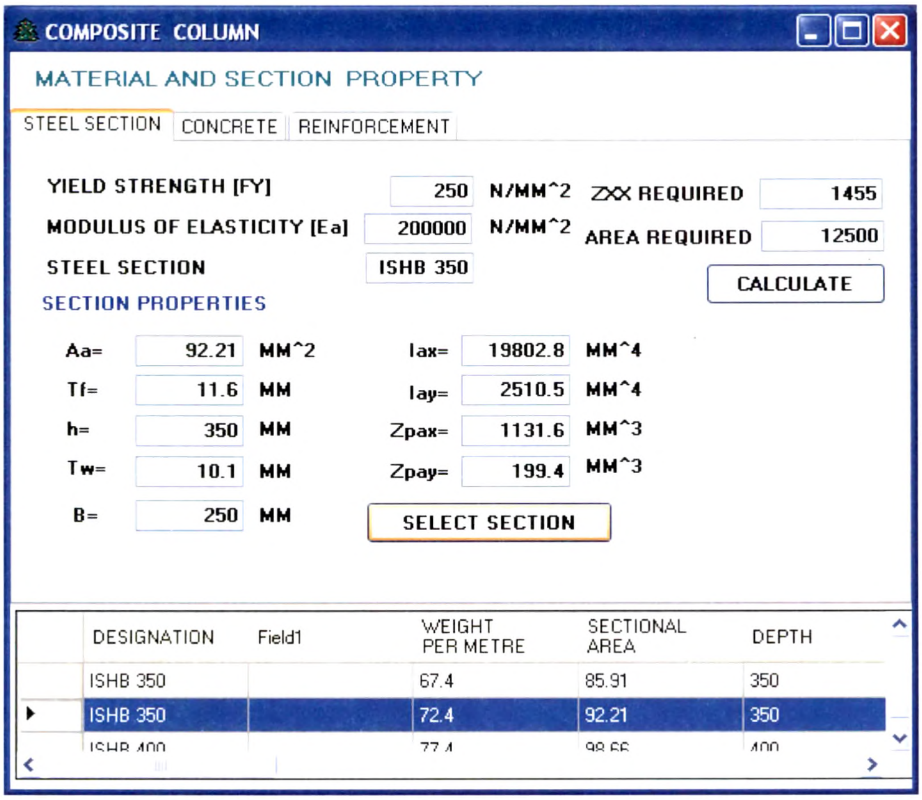


Fig. 6.15 Form for Selection of Section

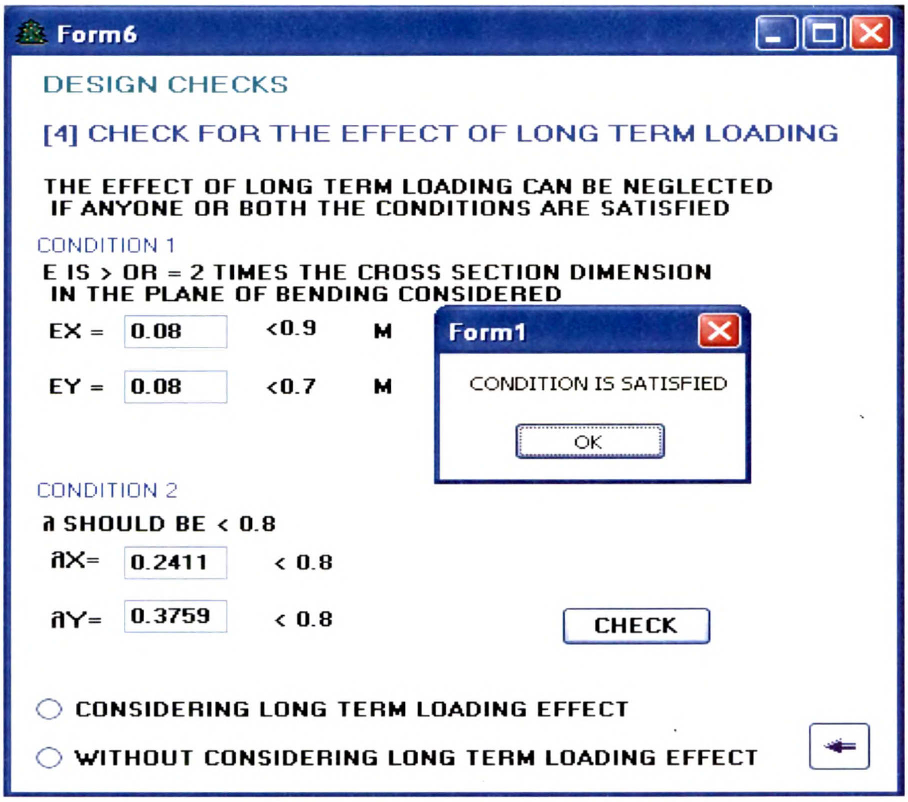


Fig. 6.16 Form for Checking Long Term Loading Effect

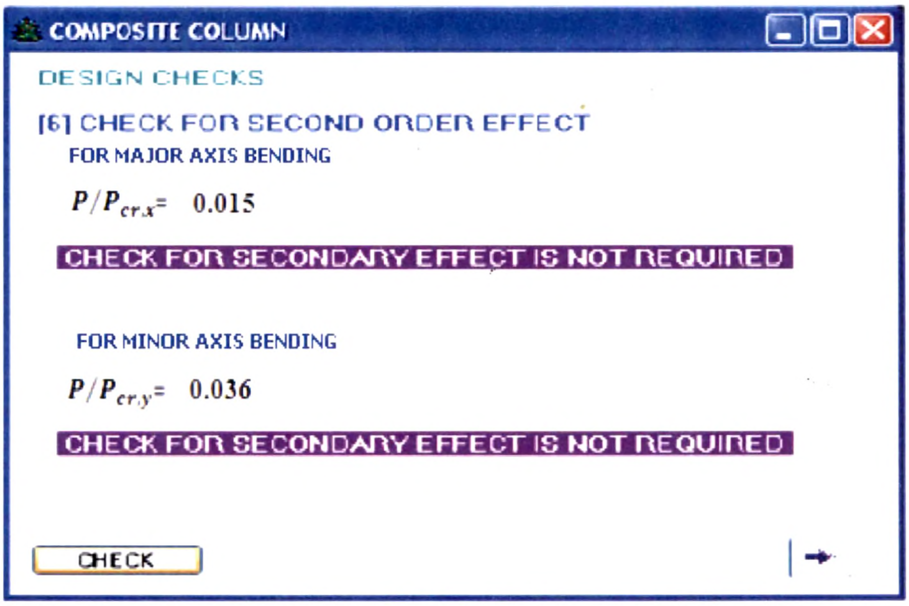


Fig. 6.17 Form for Checking Secondary Effect

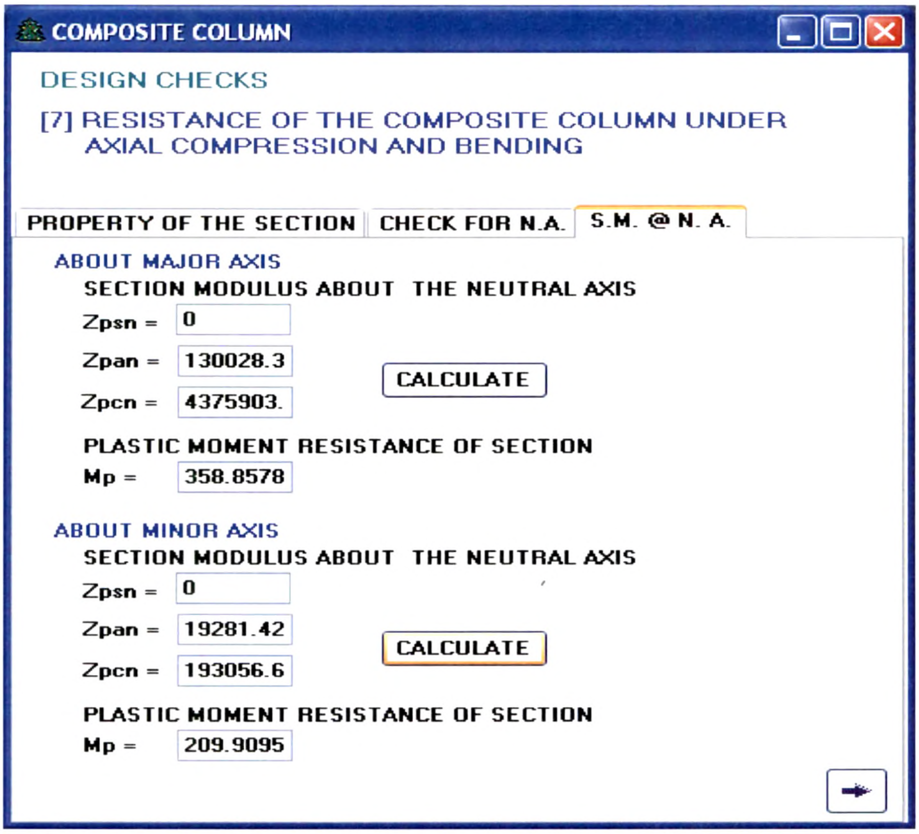


Fig. 6.18 Form for Calculation of Plastic Section Modulus

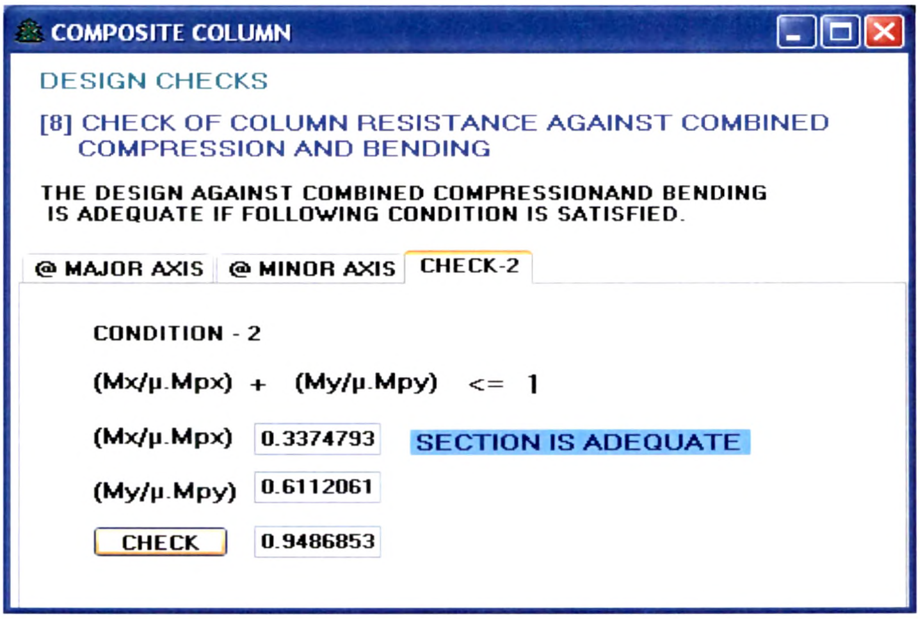


Fig. 6.19 Form for Checking the Adequacy of the Section