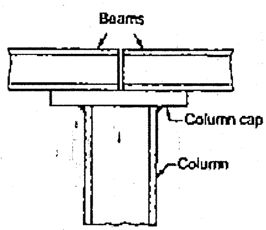
CHAPTER 07

Column Bases and Foundations

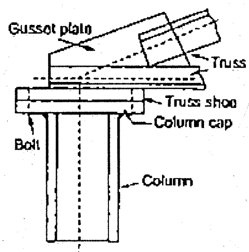
7.1 Introduction

- When column loads are large and bearing strength of concrete is low then failure may occur.
- Thus column loads are first distributed on the steel column bases which are placed on the concrete blocks. Base plate not only transfers the column load to the concrete block safely but also it keeps the proper alignment of column in plan, ensures verticality of the column and controls column deflections.
- Column bases are mainly subjected to bearing pressures from below and bending moments and shearing forces.
- When soil is having very low bearing capacity, large concrete blocks may become uneconomical and in that case grillage footing is resorted to.

Column caps are the plates provided between the member transferring the reaction to the column and the top of the column. For example: Columns supporting beams or roof trusses as shown in Fig. 7.1. The force on the column cap may either be compression or tension (uplift due to wind in trusses etc.). Column caps are designed in a same way as that of column bases.



(a) Beams supported over column



(b) Truss supported over column

Fig. 7.1 Column caps

7.2 Types of Column Bases

(a) Column base transmitting direct load only

(b) Column base transmitting bending moment in addition to direct load

- The column base of type (a) above does not require any design consideration but simply a steel plate is provided to distribute the column load over a sufficient area of concrete pedestal so that bearing strength of concrete pedestal is not exceeded.
- For this type of axially loaded column, the bearing pressure distribution between the base plate and concrete pedestal is assumed to be uniform as shown in Fig. 7.2.

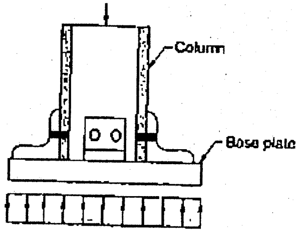


Fig. 7.2 Assumed uniform base pressure distribution under base plate

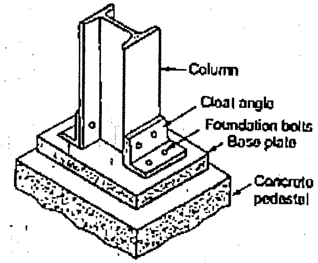


Fig. 7.3 Slab base

- Column bases which transmit moment in addition to axial load, angles enchored to the footing are used with angles being designed for uplift forces as shown in Fig. 7.3
- The two prevalent column bases are the slab base and the gusset base as shown in Fig. 7.3 and Fig. 7.4.
- Fig. 7.5 shows some of the welded column bases for small loads. Fig. 7.5 (a), column is shop welded to the base plate of the column. Fig. 7.5 (b and c) show column bases wherein angles are shop welded to the columns and column is field welded to the base plate after erection.

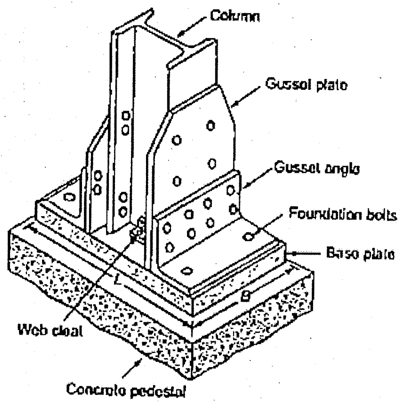


Fig. 7.4 Gusset base

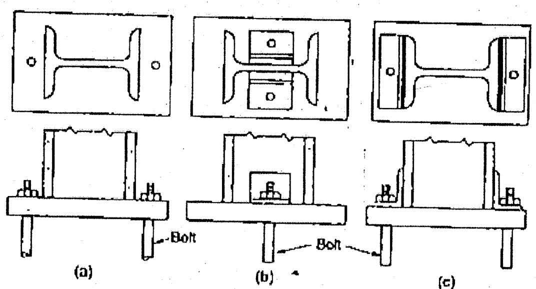


Fig. 7.5 Lightly loaded welded column bases

- Fig. 7.6 shows some of the typical welded column bases.

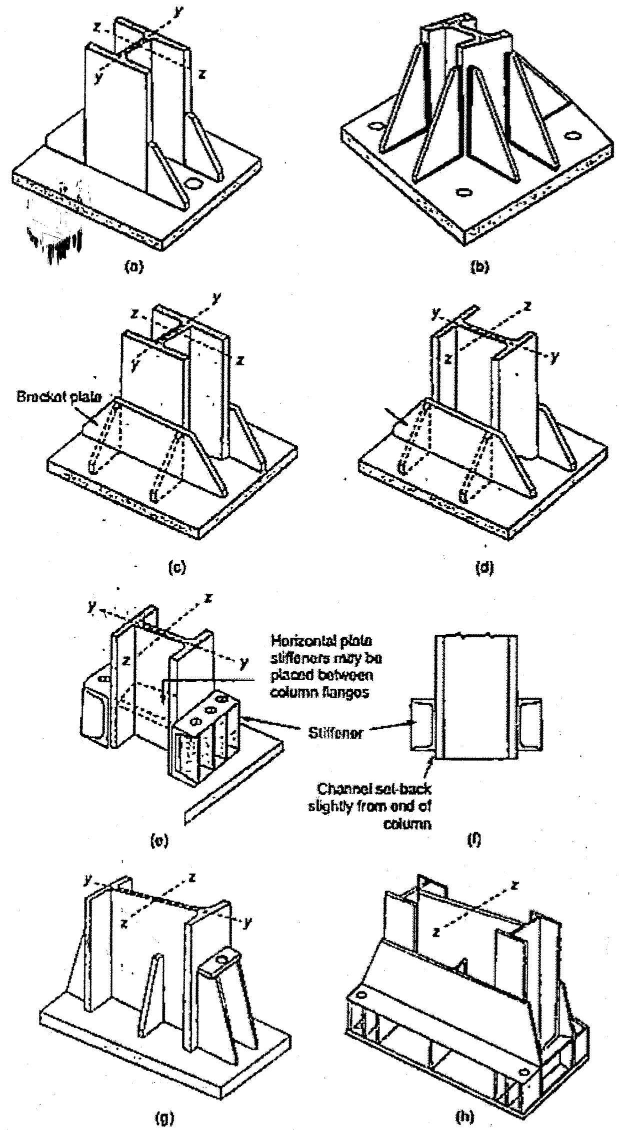


Fig. 7.6 Some typical welded column bases

7.3 Slab Base

- When column is subjected only to axial loads then column base plate can be designed by assuming a uniform base pressure distribution from below.
- For small loads, a steel plate shop welded to the column can be used to transmit the column load to the concrete pedestal as shown in Fig. 7.7.
- For bolted construction, cleat angles are bolted to the base plate in shop and column is placed between them while erection.
- When column is subjected to bending moments along with axial load, angle sections are attached to the flanges of the column. These angles are anchored with the foundation bolts to the concrete pedestal. (Fig. 7.8)
- Even if a column is subjected to direct axial load only, there also nominal angle sections are provided in order to keep the column at place and to resist tension during erection and connection. These angles may safely be omitted in case base plate is shop welded to the column.
- Depending on the values of axial load and bending moment coming over the column, it may be possible that whole of the column base is under compression or a part of the base is under compression and part under tension. Hold down bolts are provided to resist tension forces which also help in fixing the column to the base.
- The thickness of the base plate is designed from consideration of bending of the portion of base plate that extends beyond the column profile. In case where such projections are large, or the loads are very heavy, or the moments are large, there the thickness of the base plate may be reduced by the use of vertical stiffeners or plates (Fig. 7.6).

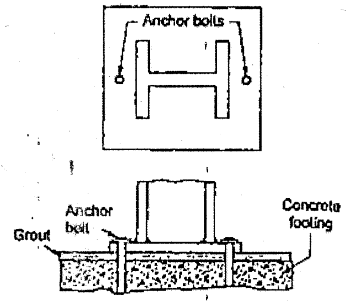


Fig. 7.7 Column base with column subjected to axial load only

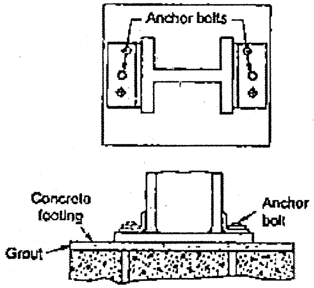


Fig. 7.8 Column base with column subjected to axial load and bending moment

7.3.1 Design of Slab Base

- The projection of slab base beyond the column profile acts as cantilevers and thus maximum bending moment occurs at the edge of the column.
- The slab tends to bend simultaneously about the two principal axes of the slab and therefore the stresses induced in one direction are influenced by the stresses due to bending in other axis. A Poisson's ratio of 0.3 is generally used to account for this effect.

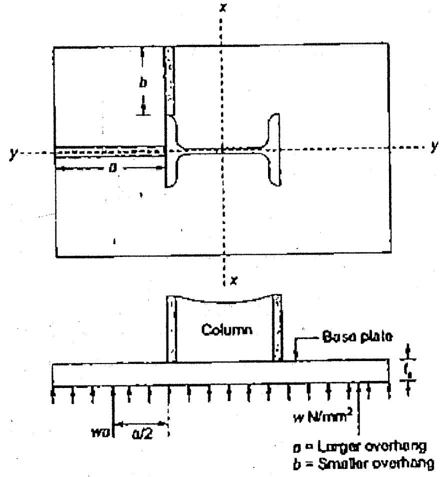


Fig. 7.9 Design of slab base

Consider 1 mm thick strip of slab projection along *y-y* axis.

Let w = Intensity of bearing pressure from concrete below the base plate

$$\text{Maximum bending moment} = \frac{w \times 1 \times a \times a}{2} = \frac{wa^2}{2} \quad ...(7.1)$$

Similarly, consider 1 mm thick strip of slab projection along *x-x* axis.

$$\text{Maximum bending moment} = \frac{w \times 1 \times b \times b}{2} = \frac{wb^2}{2} \quad ...(7.2)$$

$$\text{Thus net moment} \quad M_{net} = \frac{wa^2}{2} - 0.3\frac{wb^2}{2} = \frac{w}{2}\left(a^2 - 0.3b^2\right) \quad ...(7.3)$$

Moment capacity of the plate is given by,

$$M_p = 1.2 f_y Z_e \quad ...(7.4)$$

where Z_e = Elastic section modulus of the base plate

Thus moment capacity of 1 mm strip is,

$$M_{ps} = 1.2 f_y \times 1 \times \frac{t_s^2}{6} = 1.2 f_y \frac{t_s^2}{6} \quad ...(7.5)$$

where t_s = Thickness of the base plate

Equating M_{net} with M_{ps}, i.e., Eq. (7.3) and Eq. (7.5)

$$1.2 f_y \frac{t_s^2}{6} = \frac{w}{2}\left(a^2 - 0.3b^2\right)$$

Applying partial safety factor for the material,

$$1.2\frac{f_y}{\gamma_{m0}}\frac{t_s^2}{6} = \frac{w}{2}(a^2-0.3b^2)$$

$$t_s^2 = \frac{6w}{2\times 1.2}(a^2-0.3b^2)\frac{\gamma_{m0}}{f_y} = 2.5w(a^2-0.3b^2)\frac{\gamma_{m0}}{f_y}$$

$$\Rightarrow \qquad t_s = \sqrt{2.5w(a^2-0.3b^2)\frac{\gamma_{m0}}{f_y}} \qquad \text{...(7.6)}$$

The thickness of base plate as determined from Eq. (7.6) above must be greater than the thickness of the column flange (t_f).

7.3.2 Procedure for the Design of Slab Base

Step-1 Assume a suitable grade of concrete (if not already mentioned) and compute bearing capacity of concrete as $0.45f_{ck}$.

Step-2 The required area of the slab base is computed as,

$$A = \frac{P_u}{0.45f_{ck}} \qquad \text{...(7.7)}$$

where A = Required area of the base plate in mm²

P_u = Factored load from column

Step-3 In general, a square slab base is provided the side of which is given by,

$$L = B = \sqrt{A}$$

Another practice of column base design is that projections beyond the column profile i.e. a and b may be kept equal and the sides of the base plate is worked out as,

$$(D+2b).(b_f+2a) = A$$

where, L = Length of the base plate in mm

B = Width of the base plate in mm

a = Larger projection of base plate beyond column in mm

b = Smaller projection of base plate beyond column in mm

D = Overall depth of the column section in mm

b_f = Width of flange of column section in mm

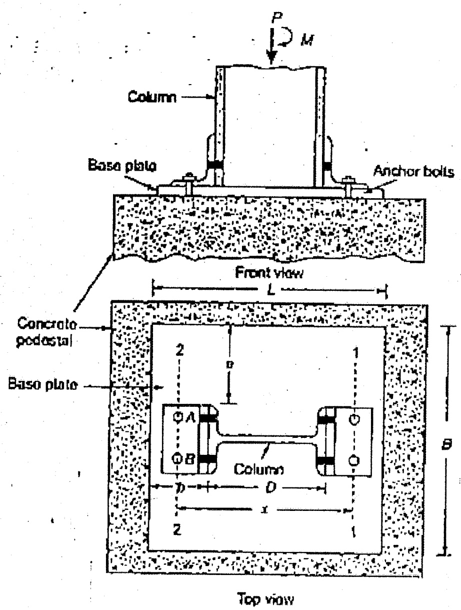


Fig. 7.10 Design of column base plate

Step-4 The intensity of the base pressure w from concrete pedestal is calculated as,

$$w = \frac{P_u}{A_1} \qquad \text{...(7.8)}$$

where A_1 = Area of base plate provided in mm²

w = Intensity of base pressure from concrete pedestal in N/mm²

Step-5 The minimum required thickness of the base plate (t_s) is computed from Eq. (7.8) and it must not be less than thickness of the column flange (t_f).

Step-6 Holding down bolts 2 or 4 in numbers of 20 mm diameter are generally provided.

Step-7 Welded joint between the column and base plate is designed. If however, column and base have machined ends and perfect bearing is assumed then axial load is assumed to be transferred directly and welding is designed for moments only.

7.4 Gusset Base

- Gusset base is provided where column load is too heavy or when the axial column load is accompanied by bending moments.
- It usually consists of two gusset plates, a base plate and two gusset angles when bolted connections are made. The gusset plates and the angles are placed on the column flanges as shown in Fig. 7.11.

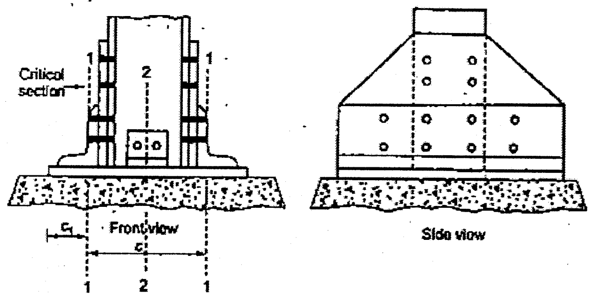


Fig. 7.11 Gusset Base

- Gusset plates increase the bearing area there by reducing the base plate thickness.
- Also, the gusset material supports the base plate against bending and thus results in smaller thickness of base plate.
- This type of base is considered as rigid.
- Where welded connections are used, there gusset angles are not required.
- The upward acting pressure from below the base causes bending of gusset plates thereby bringing the top edge of gusset in compression which may therefore buckle. This may be prevented by limiting the width to thickness ratio for the gusset plate. Referring to Fig. 7.12,

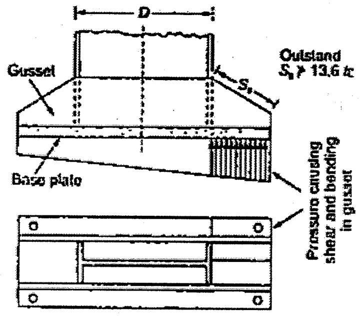


Fig. 7.12 Welded gusset base plate

$D \le 29.3\varepsilon.t_g$ for portion of gusset plate welded to the column flange

$S_o \le 13.6\varepsilon.t_g$ for the outstand of gusset plate from the edge of the column flange

Where t_g = Thickness of gusset plate

- Gusset plates are designed to resist shear and bending. The moment in the gusset must not exceed the bending strength of the gusset plate.
- Bending strength of gusset plate is given by,

$$M_{dg} = \frac{f_{yg}}{\gamma_{m0}} Z_e \quad ...(7.9)$$

where, f_{yg} = Design yield strength of gusset plate

Z_e = Elastic section modulus of the gusset plate

γ_{m0} = Partial safety factor = 1.1

- For columns provided with gusseted base, the gusset plates, gusset angles, stiffeners, fasteners, etc. in combination with the bearing area of the column should be sufficient to take the imposed and dead loads, bending moments and reactions to the base plate without exceeding the specified strength.
- In order to ensure a perfect contact, all the bearing surfaces must be machined.
- Where the column ends and gusset plates are not faced for complete bearing there the welding and fasteners connecting them to the base plate must be sufficient to transmit all the forces to which the base is subjected.

7.4.1 Procedure for the Design of Gusset Base

Step-1 Assume a suitable grade of concrete (if not already mentioned) and compute bearing capacity of concrete as $0.45f_{ck}$.

Step-2 The required area of the slab base is computed as,

$$A = \frac{P_u}{0.45f_{ck}}$$

where A = Required area of the base plate in mm², P_u = Factored load

Step-3

- The size of the gusset material is assumed.
- The thickness of gusset plate must not be less than 16 mm for bolted gusset plate.
- The gusset angle is so chosen so as to accommodate two rows of bolts in the vertical leg and one row of bolts in the horizontal leg. Thus an unequal angle section is provided. The thickness of gusset angle must be almost same as that of gusset plate.
- The length of the gusset is usually kept equal to the length of the base plate, parallel to the flange of the column. For welded gusset bases, gusset angles are not required.

Length of base plate parallel to the web.

L = depth of section + 2(thickness of gusset plate + leg length of angle + overhang) (For bolted gusset plate)

L = depth of section + 2 × thickness of gusset plate + overhang (For welded gusset plate)

The dimension of base plate parallel to flange is,

$$B = A/L$$

Step-4 The bearing pressure intensity w from concrete below the base is computed as,

$$w = \frac{P}{A_1} \qquad \text{where } A_1 = \text{Area of base plate provided}$$

Step-5

- The thickness of the base plate is computed by equating the moment at the critical section to the moment of resistance of the gusset at that section.
- This critical section is assumed to lie at the root of the fillet of the angle section for bolted connections as the load from the flanges of the column is transferred to the base plate through gusset material.
- Thicker the gusset angle, lesser will be the required base plate thickness.
- For welded connections, the critical section is assumed to lie at the face of the column flange.

Referring to Fig. 7.11,

Bending moment at critical section,

$$M = \frac{wc_1^2}{2}$$

Design bending strength at the critical section is given by,

$$M_d = 1.2\frac{f_y}{\gamma_{m0}}Z_e = 1.2\frac{f_y}{\gamma_{m0}}\left(\frac{1}{6} \times 1 \times t^2\right) = \frac{0.2}{1.1}f_y t^2$$

Thus,

$$M = M_d$$

$$\frac{wc_1^2}{2} = \frac{0.2}{1.1}f_y t^2$$

$$\Rightarrow \quad t = c_1\sqrt{2.75\frac{w}{f_y}} \quad ...(7.10)$$

where t = Aggregate thickness of the base plate and gusset angle for bolted gusset base and thickness of the base plate for welded gusset base, at the critical section

w = Intensity of pressure from concrete under the slab base in N/mm²

f_y = Yield stress of steel in N/mm²

Step-6 Holding down bolts 2 or 4 in numbers of 20 mm diameter are generally provided.

Step-7

- Welded joint between column and base plate is designed. If however, column and base have machined ends and perfect bearing is assumed then axial load is assumed to be transferred directly and welding is designed for moments only.
- For bolted gusset plate, the strength of the bolt in single shear is determined. The number of bolts required to connect the column flange with the gusset plate are worked out. Same numbers of bolts are provided to connect the gusset plate with the gusset angle since the force to be transferred from gusset plate to gusset angle will be the same.

Step-8

- The size of the gusset plate is determined.
- The length of the gusset plate is kept equal to the side of the base plate parallel to which it is provided.

- The height of gusset plate is governed from the number of rows of bolts (for bolted connection) or by length of weld to be accommodated (for welded connection).
- The thickness of gusset plate may be assumed and buckling of top edge is checked by,

$$D \le 29.3\,\varepsilon\, t_g$$
$$S_o \le 13.6\,\varepsilon\, t_g$$

The thickness of gusset plate may also be determined as,

$$M_{dg} = \frac{f_{yg}}{\gamma_{m0}} Z_e$$

where f_{yg} = Design yield strength of gusset plate, t_g = Thickness of gusset plate
Z_e = Elastic section modulus of the gusset plate, γ_{m0} = Partial safety factor

7.5 Moment Resisting Base Plate and its Design

- When base plates are eccentrically loaded or when the base plates are subjected to axial load along with moment then pressure distribution from the concrete below does not remain uniform and it in fact varies from one end to other.
- Pure axial load causes uniform compression between the base plate and the supporting concrete (Fig. 7.13 (a)).
- When moment is also acting on the base plate then it increases the compression on one side and decreases on other side of the base plate.

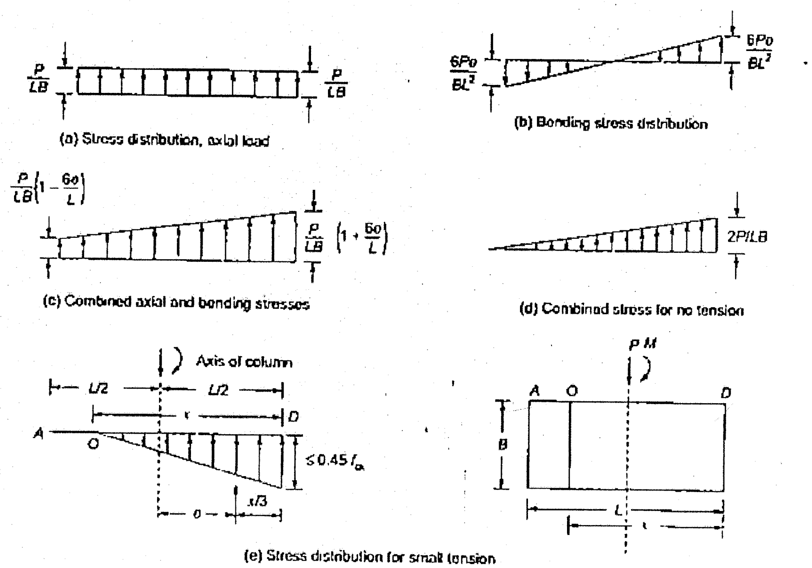


Fig. 7.13 Stress distribution in bases

Let, P = Axial load acting on the column
e = Eccentricity of the column load

Thus moment, $M = Pe$

This moment M and axial load P leads to axial compressive stress and bending stress. Due to axial load P, the stress distribution is uniform (Fig. 7.13(a)) and is given as,

$$\text{Axial compressive stress} = \frac{P}{A_1} = \frac{P}{LB}$$

where A_1 = Area of base plate provided = LB

Due to moment M, bending stress developed is given as,

$$\text{Bending stress} = \pm\frac{M}{I}y$$

$$= \pm\frac{M}{BL^3/12}\cdot\frac{L}{2} = \pm\frac{6Pe}{BL^2} \quad ...(7.11)$$

The combined stress due to axial load and bending moment is shown in Fig. 7.13(c) which are given by,

$$\text{Combined stress} = f$$

$$= \frac{P}{LB} \pm \frac{6Pe}{BL^2} = \frac{P}{LB}\left(1 \pm \frac{6e}{L}\right) \quad ...(7.12)$$

Now concrete cannot resist any tension and thus it is desirable to have compressive stress throughout and no where there is any tension. Thus depending on the eccentricity e and length of the base plate L, we may have three cases as:

Case 1: When there is no tension i.e.

$$1 - \frac{6e}{L} = 0$$

$$\Rightarrow \quad e = \frac{L}{6} \quad ...(7.13)$$

Case 2: When tension developed is quite substantial i.e.

$$e > \frac{L}{3} \quad ...(7.14)$$

Case 3: When tension developed is small i.e.

$$\frac{L}{6} < e < \frac{L}{3} \quad ...(7.15)$$

7.5.1 Base Plate Design Consideration

- Base plate design can be done either with elastic method or the plastic method.
- In the elastic method of base plate design, it is assumed that pressure from the concrete below varies linearly from maximum to minimum from one side of the base plate to the other side.
- However in the plastic method of base plate design, the pressure from the concrete below is assumed to be constant from the neutral axis to the edge of the base plate being equal to $0.45 f_{ck}$

7.5.2 Procedure for the Design of Moment Resisting Base Plate

Case I: $e < \dfrac{L}{6}$

In this case the entire base plate is under compression. Also the maximum pressure from concrete must not exceed $0.45f_{ck}$. As shown in Fig. 7.13(c), the pressure distribution varies linearly.

Step 1: By substituting the value of eccentricity e, the stress diagram is drawn just like as shown in Fig. 7.13(c). In the limiting case, when $e = L/6$, the combined stresses will be $2P/LB$ and zero.

Step 2: The width of the base plate is arrived at as,

$$\frac{2P}{LB} \leq 0.45f_{ck}$$

$$B \geq \frac{2P}{0.5Lf_{ck}} \qquad ...(7.16)$$

Thus length and width of base plate is now known.

Step 3: The thickness of the base plate is determined by equating the moment capacity of the base plate with critical moment at the critical section. The critical section is the outer edge of the column and section flange. The moment capacity of the base plate is given by,

$$M_d = 1.2\frac{f_y}{\gamma_{m0}}Z_e \qquad ...(7.17)$$

Case II: $\frac{L}{6} < e < \frac{L}{3}$

In this case, most of the part of base plate is under compression and a very little part in in tension. It is assumed that tension in the far side of the bolt is negligible. Here also, the bearing stress on concrete must not exceed $0.45f_{ck}$.

Step 1: Referring to Fig. 7.13(e), let length of plate under compression is x.

Thus, $$e+\frac{x}{3} = \frac{L}{2}$$

$\Rightarrow$ $$x = 3\left(\frac{L}{2}-e\right) \qquad ...(7.18)$$

Step 2: The width of the base plate is determined as follows.

Compressive force in concrete (C) = Area of stress diagram x width

$$C = \frac{0.45f_{ck}x}{2}B$$

This compressive force (C) should be equal to the load acting on the base plate (P).

Thus, $$C = \frac{0.45f_{ck}x}{2}B = P$$

$\Rightarrow$ $$B = \frac{2P}{0.45f_{ck}x} \qquad ...(7.19)$$

Substituting the value of x as determined in Step-1.

$$B = \frac{2P}{0.45f_{ck}\left[3\left(\frac{L}{2}-e\right)\right]} \qquad ...(7.20)$$

Step 3: The thickness of the base plate is determined by equating the moment capacity of the base plate with the moment at critical section which is outer edge of the column flange. As stated earlier, the moment capacity of the base plate is given by,

$$M_d = 1.2\frac{f_y}{\gamma_{m0}}Z_e$$

Case III: $e > \frac{L}{3}$

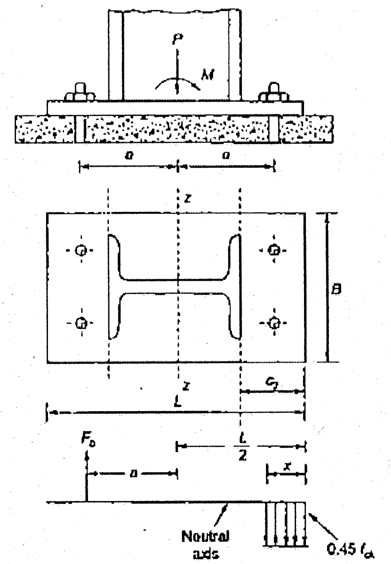


Fig. 7.14 Base plate with a part of it in compression with tension in anchor bolts

Here in this case, a part of the base plate will be in compression and substantial part will be in tension. This tension is taken up by anchor bolts.

Step 1: The size of the base plate is either determined as given earlier or is assumed.

Step 2: If P is the axial compressive force, F_b is the tensile force in the bolt then from static equilibrium of forces,

$$P = 0.45f_{ck}Bx - F_b$$

$$M = 0.45f_{ck}Bx\left(\frac{L}{2}-\frac{x}{2}\right)+F_b a \qquad ...(7.21)$$

From above two equations, the value of x is obtained as,

$$x = \left(\frac{L}{2}+a\right) - \sqrt{\left(\frac{L}{2}+a\right)^2 - \frac{2(M+Pa)}{0.45f_{ck}B}} \qquad ...(7.22)$$

If c_2 is the outstand of base plate from column flange then the maximum moment at critical section is given by,

$$M = 0.45f_{ck}Bx\left(c_2-\frac{x}{2}\right) \qquad ...(7.23)$$

Step-3: By equating the moment capacity of the base plate with the moment at critical section, the thickness of the base plate is arrived at. The moment capacity of the base plate is given as,

$$M_d = 1.2\frac{f_y}{\gamma_{m0}}Z_e \qquad ...(7.24)$$

Step-4: Now, by substituting the value of x the design tensile force in the bolt can be determined as,

$$P = 0.45f_{ck}Bx - F_b$$

$$F_b = 0.45f_{ck}Bx - P$$

Step-5: Welded connection is designed to join the column section with the base plate for maximum tension in the column flange due to the applied moment.

7.6 Foundation Bolts

- The foundation bolts are also referred to as anchor bolts and are provided to check the uplift of base plate.
- The bolts are anchored into the foundation by a hook or a washer plate or by some other arrangements.
- These bolts may either be cast-in-place (i.e. placed before the concrete is cast) or post-installed.

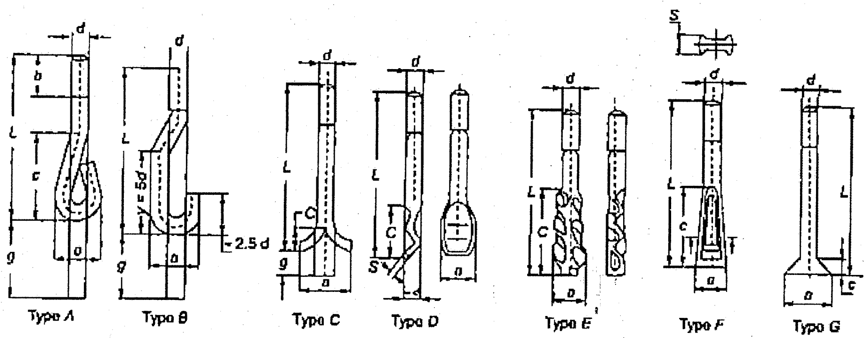


Fig. 7.15 Some typical anchor bolts

- The transfer of tensile load occurs due to bond between anchor bolt and concrete (in case of cast-in-place anchor bolts) or epoxy/grout filling (in case of post-installed anchor bolts).
- For very heavy uplift forces, a plate may be bolted at the lower end of the bolt.
- Cast-in-place anchor bolts generally fail due to yielding and fracture of anchor shank and breakout of concrete.
- A minimum of four bolts, one each at the corners of the base plate are provided which are in fact sufficient also.
- Larger number of bolts may also be provided if required but it should not exceed eight (08) from considerations of construction aspects.
- A minimum of two anchor bolts are provided even if column is subjected to only axial loads.
- In presence of bending moments, foundation bolts must be of adequate size so as to hold down the column to the concrete pedestal.
- The capacity of these bolts to resist tension depends on their lengths available to deform elastically. Their capacity can be increased by pre-tensioning them.

Referring to Fig. 7.10, let the column is subjected to an axial load P and a moment M. This column will try to overturn either about 1-1 or 2-2 passing through the bolts depending on whether the moment is clockwise or anti-clockwise. The stabilizing moment (=Px/2) is provided by the axial load P.

When a clockwise moment M acts on the column (Fig. 7.10), bolts A and B will experience uplift and C and D to thrust as shown in Fig. 7.16.

This uplift and thrust will provide a resisting couple ($=2F_b x$).

Thus total stabilizing moment $= \dfrac{Px}{2} + 2F_b x$ ...(7.25)

At equilibrium, $M = \dfrac{Px}{2} + 2F_b x$

$\Rightarrow$ $F_b = \dfrac{M}{2x} - \dfrac{P}{4}$ ...(7.26)

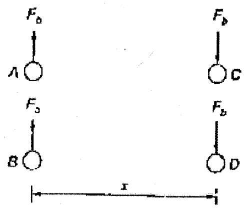


Fig. 7.16 Forces on bolts under moment on a column

where F_b = Uplift force on bolt (in N)

x = Distance between the bolt lines

The tensile capacity based on concrete pull out failure is given by,

$$N_n = k\sqrt{f_{ck}}\, l^{3/2} \qquad ...(7.27)$$

where k = 13.5 for post – installed anchor bolts

= 15.5 for cast-in-situ anchor bolts

f_{ck} = Characteristic strength of concrete

l = Length of embedment

In case of grouted anchors, failure may occur by bond failure at the grout – concrete interface. The bond strength is given by,

$$N_b = \tau_b \pi d_o l \qquad ...(7.28)$$

where τ_b = Bond strength of grout – concrete

d_o = Diameter of bolt hole

l = Length of embedment

7.6.1 Procedure for the Design of Foundation Bolts

Step-1. The uplift force F_b on the bolts is determined from Eq. (7.26).

Step-2. The diameter of the bolts is assumed beforehand and stress area is referred from the Table 7.1.

Table 7.1 Tensile stress area of bolt

Bolt size, d (mm)	12	16	20	22	24	27	30	36
Tensile stress area (mm²)	84.3	157	245	303	353	459	561	817

The tensile strength of the bolt is,

$$T_{db} = \frac{T_{nb}}{\gamma_{mb}} = 0.9\, f_{ub} A_{nb}$$

$$< \frac{f_{yb} A_{sb} \gamma_{mb}}{\gamma_{m0}} \qquad ...(7.29)$$

where A_{nb} = Area of bolt at the root of the thread i.e. stress area, in mm²

f_{ub} = Ultimate stress of the bolt

γ_{mb} = Partial factor of safety = 1.25

Step-3. The number of bolts required to resist the uplift force are given by,

$$n = \frac{F_b}{T_{db}}$$

Step-4. The tensile force to be transferred by the bolt is resisted by the bond between the anchor bolt and the concrete.

Length of bolt, $$l = \frac{d}{4\tau_{bd}} \cdot \frac{f_y}{\gamma_{mo}} \qquad ...(7.30)$$

where τ_{bd} = Bond stress, in N/mm²

d = Diameter of the bolt, in mm

γ_{mo} = Partial factor of safety = 1.10

The length of the bolt can be calculated from eq. (iv) i.e. based on concrete pull – out failure and larger of the two lengths i.e. from eq. (iv) and eq. (vi) should be adopted in design.

Table 7.2 Bond stress for plain bars in concrete (N/mm^2)

Concrete grade	M25	M25	M30	M35	M40
Bond stress (N/mm^2)	1.2	1.4	1.5	1.7	1.9

NOTE

1. The bolt diameter may be assumed between 20 to 36 mm.
2. For HYSD and other rough bars, bond stress as given in Table 7.2 may be increased by 60%.

Illustrative Examples

Example 7.1 A column section is ISHB 350 @ 72.4 kg/m and is subjected to an axial compressive load of 1050 kN under service conditions. Design the slab base for the column is load gets transmitted to the base plate by welded connections and column end and base plate are not machined for bearing. The base sets on a concrete pedestal of M20 concrete. Determine whether undation bolts are required?

Solution:

For Fe410 steel, $f_y = 250$ N/mm2, $F_u = 410$ N/mm^2

Bearing strength of M20 concrete, $f_u = 0.45\,f_{ck} = 0.45 \times 20 = 9$ N/mm^2

For shop weld, partial factor of safety for material, $\gamma_{mw} = 1.25$

and partial factor of safety against yielding, $\gamma_{m0} = 1.1$

For ISHB 350 @72.4 kg/m,

Depth of section, $D = 350$ mm

Flange width, $b_f = 250$ mm

Flange thickness, $t_f = 11.6$ mm

Web thickness, $t_w = 10.1$ mm

Area of slab base required, $A_{reqd} = \dfrac{1.5\times1050\times10^3}{9} = 1{,}75{,}000$ mm^2 = [illegible]75 m^2

Provide a rectangular base plate of side $L \times B$, with equal overhangs beyond the column flanges i.e. $a = b$

Area of base plate $= (L + 2a)(B + 2a)$

$\Rightarrow \quad 0.175 \times 10^6 = (350 + 2a)(250 + 2a)$

$\Rightarrow \quad 175000 = 87500 + 1200a + 4a^2$

$\Rightarrow \quad a^2 + 300a - 21875 = 0$

$$\therefore \quad a = \frac{-300 \pm \sqrt{300^2 + 4(21875)}}{2}$$

$$= \frac{-300 + 421.308}{2} \text{ (Neglecting -ve sign)} = 60.654 \text{ mm} \simeq 65 \text{ mm (say)}$$

Thus provide overhangs of 65 mm

$\therefore \quad L = 350 + 2 \times 65 = 480$ mm

$B = 250 + 2 \times 65 = 380$ mm

Bearing pressure due to concrete,

$$W = \frac{P}{\text{Area provided}} = \frac{P}{A_1} = \frac{1.5\times1050\times10^3}{480\times380} = 8.635 \text{ N/mm}^2 < 9 \text{ N/mm}^2$$

Thickness of slab base required,

$$t_s = \sqrt{2.5w(a^2 - 0.3b^2)\frac{\gamma_{mo}}{f_y}}$$

$$= \sqrt{2.5\times8.635\times(65^2 - 0.3\times65^2)\frac{1.1}{250}} = 16.761 \text{ mm} = 18 \text{ mm (say)}$$

Flexural strength of weld,

$$dw - \frac{1.2zf_y}{\gamma_{mo}} = 1.2\times91875\times\frac{250}{1.1} \text{Nmm}$$

$= 25.057$ kNm

> 22.5 kNm

Thus design flexural strength of weld > factored bending moment

Double U butt used safe in bending ...(i)

factored shear force $= 1.5 \times 170 = 255$ kN

Design shear strength of used, $V_{dw} = l_w t_a \cdot \dfrac{f_{yw}}{\sqrt{3}.\gamma_{mw}}$

$\Rightarrow \quad l_w = 122.687$ mm < 175 mm

Length of weld required < Length of weld provided

Double U butt weld is safe in shear. ...(ii)

From, (i) and (ii), double U butt weld provided is adequate and is safe.

Example 7.2 A column section ISHB 450 @ 859 N/m is subjected to an axial compressive load of 485 kN and a moment of 103 kNm. Using M30 concrete for the pedestal, design the base plate and anchor bolts. Design weld also. Use Fe410 steel

Solution:

For Fe410 grade of steel,

$f_u = 410$ N/mm^2

$f_y = 250$ N/mm^2

Bearing strength of concrete $= 0.45\,f_{ck} = 0.45 \times 30 = 13.5$ N/mm^2

Partial factor of safety for material,

$\gamma_{m0} = 1.1$

Partial factor of safety for shop welding,

$$\gamma_{mw} = 1.25$$

Partial factor of safety for site welding,

$$\gamma_{mw} = 1.5$$

Section properties of ISHB 450@ 859 Nm are as under:

Depth of section, h = 450 mm

Flange width, b_f = 250 mm

Flange thickness, t_f = 13.7 mm

web thickness, t_w = 9.8 mm

Axial compressive load (P) = 485 kN

Bending moment (M) = 103 kNm

$$\therefore \quad \text{Eccentricity } (e) = \frac{M}{P} = \frac{103 \times 10^6}{485 \times 10^3} = 212.37 \text{ mm}$$

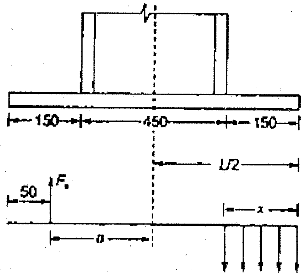
Let projection of base plate beyond column flanges

= 150 mm

$\therefore$ Minimum length of base plate (L) = 450 + 2 × 150 = 750 mm

$$\therefore \quad \frac{6e}{L} = \frac{6 \times 212.37}{750} = 1.699 > 1$$

$$\therefore \quad \left(1 - \frac{6e}{L}\right) < 0$$



$\Rightarrow$ There will be tension in the anchor bolts.

Take $L = B = 750$ mm

$$a = \frac{L - 2 \times 50}{2} = \frac{750 - 2 \times 50}{2} = 325 \text{ mm}$$

$$x = \left(\frac{L}{2} + a\right) - \sqrt{\left(\frac{L}{2} + a\right)^2 - \frac{2(M + P_a)}{0.45 f_{ck} B}}$$

$$= \left(\frac{750}{2} + 325\right) - \sqrt{\left(\frac{750}{2} + 325\right)^2 - \frac{2(103 \times 10^6 + 485 \times 10^3 \times 325)}{0.45 \times 30 \times 750}}$$

$$= 37.793 \text{ mm}$$

$\therefore$Tensile force on anchor bolt

$$f_b = 0.45 f_{ck} Bx - P$$

$$= (0.45 (30) (750) 37.793 - 485 \times 1000) \text{N}$$

$$= -102.345 \text{ kN (Tension)}$$

Design of anchor bolts

$$\text{Tensile strength of bolt } (T_{nb}) = 0.9 A_{nb} \cdot f_{ub} > A_{sb} f_{yb} \cdot \frac{\gamma_{mb}}{\gamma_{m0}}$$

Let 20 mm diameter bolts of grade 4.6 are provided.

$\therefore$ For 20 mm diameter bolts, $A_{nb} = 245 \text{ mm}^2 \left(\simeq 0.78 \times \frac{\pi}{4} \times 20^2\right)$

$$A_{sb} = \frac{\pi}{4}(20)^2 = 314.16 \text{ mm}^2$$

$$\therefore \quad T_{nb} = 0.9 (245)\, 400 \text{ N} = 88.2 \text{ kN}$$

Also

$$A_{sb} f_{yb} \cdot \frac{\gamma_{mb}}{\gamma_{m0}} = (314.16) 250 \times \frac{1.25}{1.1} \text{ N} = 89.25 \text{ kN}$$

$$\therefore \quad T_{nb} < A_{sb} f_{yb} \cdot \frac{\gamma_{mb}}{\gamma_{m0}} \text{ (O.K)}$$

$$\text{Design tensile strength of bolt, } T_{db} = \frac{T_{nb}}{\gamma_{mb}} = \frac{88.2}{1.25} = 70.56 \text{ kN}$$

$$\therefore \quad \text{No. of bolts required } (n) = \frac{F_b}{T_{db}} = \frac{102.345}{70.56} = 1.45 \approx 02 \text{ (say)}$$

$\therefore$ Provide 2 nos. 20 mm diameter anchor bolts.

Length of bolt

$$\text{Length of bolt,} \quad l = \frac{d}{4\tau_{bd}} \cdot \frac{f_y}{\gamma_{m0}} = \frac{20}{4 \times 1.5} \times \frac{250}{1.1}$$

$$= 757.58 \text{ mm} = 760 \text{ mm (say)}$$

Length of bolt required from pull out criterions,

$$N_u = k\sqrt{f_{ck}} \cdot l^{1.5}$$

$$\Rightarrow \quad 70.56 \times 10^3 = 15.5\sqrt{30} \cdot l^{1.5}$$

$$\Rightarrow \quad l = 88.4 \text{ mm} < 760 \text{ mm}$$

$\therefore$ Provide 2-20 mm dia anchor bolts 760 long.

Thickness of base plate

$$\text{Maximum moment} = 0.45 f_{ck} Bx\left(c_2 - \frac{x}{2}\right)$$

$$= 0.45(30)\,(750)(37.733)\left(150 - \frac{37.793}{2}\right) \text{ N.mm}$$

$$= 50.17 \text{ kNm}$$

$$\text{Moment capacity of base plate, } M_d = 1.2 \frac{f_y}{\gamma_{m0}} Z_e = 1.2 \times \frac{250}{1.1} \times \left(\frac{1}{6} \times 750 \times t^2\right) \text{ N.mm}$$

$$= 34090.91\, t^2 \text{ N.mm}$$

$$\therefore \quad 50.17 \times 10^6 = 34090.91\, t^2$$

$$\Rightarrow \quad t = 38.36 \text{ mm} \approx 40 \text{ mm (say)}$$

Design of column connection with base plate

The axial compressive load will be transferred directly due to full bearing available. Thus weld will be designed only for moment.

$$\text{Maximum tension in flange} = \frac{M}{D - t_f} = \frac{103 \times 10^6}{450 - 13.7} = 236.08 \text{ kN}$$

Length available for welding on one flange

$$= 250 + (250 - 9.8) = 490.2 \text{ mm}$$

Let size of weld (s) = 6 mm

$$\therefore \text{ strength of weld per mm length} = l_w t_t \frac{f_u}{\sqrt{3}\gamma_{mw}} = \frac{1 \times (0.7 \times 6) \times 410}{\sqrt{3} \times 1.5} = 662.8 \text{ N/mm}$$

$$\therefore \quad \text{Length of weld required} = \frac{236.08 \times 1000}{662.8} = 356.2 \text{ mm} < 490.2 \text{ mm}$$

∴ Provide 6 mm fillet weld around the flanges.

Example 7.3 A column section ISHB 350 @ 710 N/m support a roof truss and transfer factored loads (DL + LL) and (DL + WL). Due to the load combination (DL + LL), a compressive load of 105 kN acts downwards. Also due to uplift force of (DL + WL), tensile load of 63 kN acts on the column. Design the column cap. Use Fe410 grade of steel.

Solution:

For Fe410 grade of steel, $f_u = 410$ N/mm²,

$f_y = 250$ N/mm²

Partial factor of safety for material,

$\gamma_{m0} = 1.1$

Partial factor of safety for weld, $\gamma_{mw} = 1.25$

Partial factor of safety for site weld, $\gamma_{mw} = 1.5$

Propeties of ISHB 350 @ 410 N/m are as shown:

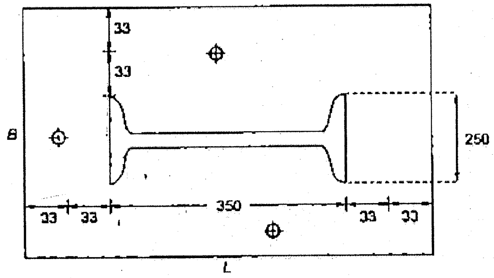
Depth of section,	$h = 350$
Cross-sectional area,	$A = 9221$ mm²
width of flange,	$b_f = 250$ mm
Thickness of flange,	$t_f = 11.6$ mm
Thickness of web,	$t_w = 10.1$ mm
MOI about Z-Z,	$I_z = 19802.8 \times 10^4$ mm⁴
MOI about Y-Y,	$I_y = 2510.5 \times 10^4$ mm⁴
Radius of gyration about Z-Z,	$r_z = 146.5$ mm
Radius of gyration about Y-Y,	$r_y = 52.2$ mm
Section modulus about Z-Z,	$Z_z = 1131.6 \times 10^3$ mm³
Section of modulus about Y-Y,	$Z_y = 199.4 \times 10^3$ mm³
Radius of rest,	$r_1 = 12$ mm
Plastic section modulus,	$Z_{pz} = 1268.7 \times 10^3$ mm³

Assuming truss shoe plate to be connected to column cap by 20 mm diameter bolts.

Minimum edge distance for 20 mm dia bolt (e) = 33 mm

Assuming all loads given are factored loads.

Size of plate



$$\text{Length of plate } (L) = 350 + 2 \times 33 + 2 \times 33 = 482 \text{ mm} \approx 500 \text{ mm}$$

$$\text{width of plate } (B) = 250 + 2 \times 33 + 2 \times 33 = 382 \text{ mm} \approx 400 \text{ mm}$$

∴ Provide a plate of 500 × 400 as column cap.

Thickness of plate

Thickness of base plate required is given by

$$t_s = \sqrt{2.5w(a^2 - 0.3b^2)\frac{\gamma_{m0}}{f_y}}$$

projection along length of column cap,

$$a = \frac{L - h}{2} = \frac{500 - 350}{2} = 75 \text{ mm}$$

Projection along width of column cap,

$$b = \frac{B - b_f}{2} = \frac{400 - 250}{2} = 75 \text{ mm}$$

Out of 105 kN and 63 kN, 105 kN is longer and thus,

$$\text{Unit pressure } (W) = \frac{P}{A_1} = \frac{105 \times 1000}{500 \times 400} = 0.525 \text{ N/mm}^2$$

$$\therefore \quad t_s = \sqrt{2.5(0.525)(75^2 - 0.3 \times 75^2)\frac{1.1}{250}} = 4.77 \text{ mm}$$

and $t_f = 11.6$ mm

Thus provide a column cap of 12 mm thickness. Thus size of column cap is 500 × 400 × 12 mm.

Connection of column cap with top of column

The column cap is required to be welded to the top of column.

Let size of weld (s) = 5 mm

Length available for welding along the column profile

$$= 2 \times 250 + 2 \times (250 - 10.1) + (350 - 2 \times 11.6) = 1306.6 \text{ mm}$$

There are 12 edges available for welding as shown.

∴ Overall effective weld length available

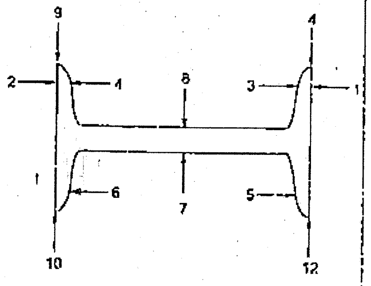
$$= \text{Total weld length available} - (2 \times S \times \text{Total no. of weld edges available}$$

$$= 1306.6 - (2 \times 5 \times 12) = 1186.6 \text{ mm}$$

Strength of weld per mm length

$$= l_w t_t \frac{f_u}{\sqrt{3}\gamma_{mw}} = 1 \times (0.7 \times 5) \times \frac{410}{\sqrt{3} \times 1.5}$$

$$= 552.33 \text{ N/mm}$$



∴ Length of weld required

$$= \frac{105 \times 1000}{552.33} = 190.1 \text{ mm} << 1186.6 \text{ mm}$$

Connection of column cap with roof truss shoe

Here the bolts will be designed for the tensile force of 63 kN.

Used bolts of grade 4.6 and thus f_{ub} = 400 N/mm²

$$\text{Tensile strength of 20 mm dia bolt } (T_{nb}) = 0.9 A_{nb} f_{bu} \geq \frac{f_{yb} A_{sb}}{\gamma_{m0}} \cdot \gamma_{mb}$$

$$\text{For 20 mm dia bolts, } A_{nb} = 245 \text{ mm}^2 \left(\simeq 0.78 \times \frac{\pi}{4} d^2\right)$$

$$A_{sb} = 314 \text{ mm}^2$$

$$\therefore \quad T_{nb} = 0.9\,(245)\,400 \text{ N} = 88.2 \text{ kN}$$

$$\frac{f_{yb} A_{sb}}{\gamma_{m0}} \cdot \gamma_{mb} = \frac{250(314)1.25}{1.1} \text{N} = 89.2 \text{ kN}$$

Thus

$$T_{nb} > \frac{f_{yb} A_{sb}}{\gamma_{m0}} \cdot \gamma_{mb}$$

$$\text{Design tensile strength of 20 mm dia bolt } (T_{bd}) = \frac{T_{nb}}{\gamma_{mb}} = \frac{88.2}{1.25} = 70.56 \text{ kN}$$

$$\therefore \quad \text{No. of bolts required} = \frac{63}{70.56} = 0.893 \simeq 4 \text{ bolts (say)}$$

Provide 4 nos. 20 mm diameter bolts of grade 4.6 at each corner of column cap.

Example 7.4 A column section ISHB 350 @ 710 N/m is required to carry a factored axial compression of 1800 kN. Design gusset base for this column. Assume pedestal concrete of grade M20. Use 24 mm diameter bolts of grade 4.6 for connections. Use steel of grade Fe410.

Solution:

For steel grade Fe410,

$$f_u = 410 \text{ N/mm}^2,$$
$$f_y = 250 \text{ N/mm}^2$$

Bearing strength of concrete, $\quad 0.45 F_{ck} = 0.45(20) = 9 \text{ N/mm}^2$

Properties of ISHB 350 @ 710 N/m are as follows:

Depth of section, h = 350 mm

Cross-sectional area, A = 9221 mm²

Width of flange, b_f = 250 mm

Thickness of flange, t_f = 11.6 mm

Thickness of web, t_w = 10.1 mm

MOI about Z-Z, $\quad I_z = 19802.8 \times 10^4 \text{ mm}^4$

MOI about Y-Y, $\quad I_y = 2510.5 \times 10^4 \text{ mm}^4$

Section modulus about Z-Z, $\quad Z_z = 1131.6 \times 10^3 \text{ mm}^3$

Section modulus about Y-Y, $\quad Z_y = 199.4 \times 10^3 \text{ mm}^3$

Plastic section modulus, $\quad Z_{pz} = 1268.7 \times 10^3 \text{ mm}^3$

For bolt of grade 4.6, f_{ub} = 400 N/mm²

Partial factor of safety for bolt material, γ_{mb} = 1.25

$$\text{For 24 mm diameter bolts, } A_{nb} = 353 \text{ mm}^2 \left(\simeq 0.70 \times \frac{\pi}{4} d^2\right)$$

Diameter of bolt hole for 24 mm dia bolt, $\quad d_0 = 24 + 2 = 26$ mm

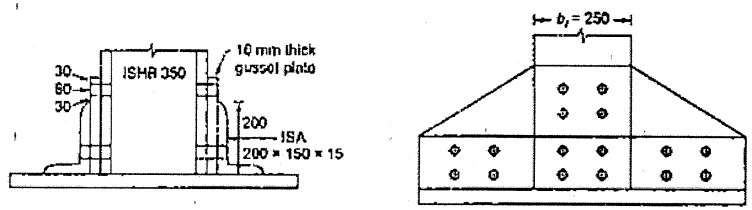
Minimum edge distance for 24 mm diameter bolt,

e = 39 mm

Axial compressive load (P) = 1800 kN

$$\therefore \quad \text{Area of base plate required } (A) = \frac{1800 \times 10^3}{9} = 200 \times 10^3 \text{ mm}^2$$

Let thus 16 mm thick guest plates are provided on each side of the column flange along with two gusset angles 200 × 150 × 15 mm.



Let pitch of bolts (p) = 55 mm

Minimum width of gusset plate required

$$= 350 + (2 \times 16) + (2 \times 150)$$
$$= 682 \text{ mm} \simeq 700 \text{ mm (say)}$$

∴ Projection of base plate beyond flange angle toe

$$= \frac{700 - 682}{2} = 9 \text{ mm}$$

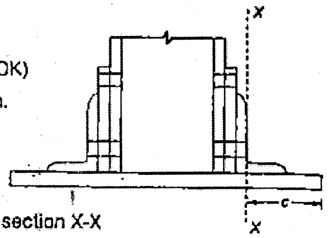
$$\therefore \quad \text{length of base plate} = \frac{200 \times 10^3}{700} = 285.7 \text{ mm} \simeq 300 \text{ mm (say)}$$

Thus provide a base plate of size 700 × 300 mm

Bearing pressure of concrete (W)

$$= \frac{P}{A} = \frac{1800 \times 1000}{300 \times 700}$$

$$= 8.57 \text{ N/mm}^2 < 9 \text{ N/mm}^2 \quad \text{(OK)}$$



The critical section for base plate is the section X-X as shown.

Length of base plate acting as cantilever (c)

$$= 9 + (150 - 15) = 144 \text{ mm}$$

Total thickness of base plate and gusset angle at critical section X-X

Maximum bending moment at $X - X$,

$$M_x = \frac{wc^2}{2} = \frac{8.57 \times 144^2}{2} = 88853.76 \text{ N.mm}$$

Combined moment capacity of base plate and angle is given by,

$$M_d = \frac{1.2 f_y Z_e}{\gamma_{m0}} = \frac{1.2(250)}{1.1}\left(1 \times \frac{t^2}{6}\right) = 45.455\, t^2$$

Equating moment capacity with maximum bending moment,

$$45.455\, t^2 = 88853.76$$

$$\Rightarrow \quad t = 44.212 \text{ mm}$$

$\therefore$ Thickness of base plate $= 44.212 - 15 = 29.212 \text{ mm} \simeq 32$

The thickness of base plate $= 32 \text{ mm} > t_f (= 11.6 \text{ mm})$ (o.k)

$\therefore$ Provide a base plate of $700 \times 300 \times 32$ mm.

Bolted connections

The bolts provided are in single shear and bearing.

$$\text{Strength of bolt in single shear } (V_{sb}) = \frac{A_{nb} f_u}{\sqrt{3}\gamma_{mb}} = \frac{353 \times 400}{\sqrt{3} \times 1.25} N = 65.22 \text{ kN}$$

$$\text{Strength of bolt in bearing } (V_{pb}) = 2.5 k_b \frac{dt f_u}{\gamma_{mb}}$$

where $k_b >$ minimum of

$$\frac{e}{3d_0} = \frac{36}{2 \times 26} = 0.75$$

$$\frac{p - 0.25}{3d_0} = \frac{55 - 0.25}{3 \times 26} = 0.455$$

$$\frac{f_{ub}}{f_u} = \frac{400}{410} = 0.976 = 1.0$$

$$V_{pb} = 2.5(0.455)(24)\frac{32(410)}{1.25} N = 286.5 \text{ kN}$$

Let column end and gusset material have 100% bearing and thus 50% of axial load will get transferred directly and remaining 50% through bolts.

$$\text{No. of bolts required} = \frac{0.5 \times 1800}{65.22} = 13.8 \simeq 16 \text{ bolts (say)}$$

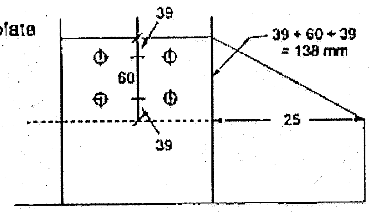
$\therefore$ provide 16 nos. 24 mm diameter bolts on each column flange as shown.

Height of gusset plate $= 200 + 2 \times 39 + 60 = 338 \text{ mm} \simeq 340 \text{ mm}$ (say)

Length of gusset plate $=$ Length of base plate $= 300$ mm

$\therefore$ Provide a gusset plate of size $340 \times 300 \times 16$ mm.

Check for buckling of compression edge of gusset plate



$$\epsilon = \sqrt{\frac{250}{f_y}} = \sqrt{\frac{250}{250}} = 1$$

$$\text{Outstand of gusset} = \frac{300 - 250}{2} = 25 \text{ mm}$$

$$\text{Slant edge of outstand } (S_0) = \sqrt{138^2 + 25^2} = 140.25 \text{ mm}$$

But So $\quad 13.66 \epsilon\, t_g = 13.66 \times 1 \times 16 = 218.56$ mm

$\therefore \quad S_0 = 140.25 > 13.66\, \epsilon\, t_g$ (OK)

Objective Brain Teasers

Q.1 The length of anchor bolt is given by:

(a) $\frac{df_y}{4\tau_{bd}\gamma_{m0}}$ (b) $\frac{df_y}{2\tau_{bd}\gamma_{m0}}$

(c) $\frac{Af_y}{4\tau_{bd}\gamma_{m0}}$ (d) $\frac{Af_y}{\tau_{bd}\gamma_{m1}}$

Q.2 The purpose of anchor bolts in column base is to

(i) act as reinforcement in the concrete pedestal of the steel column

(ii) to resist tension forces

(iii) to keep the column in place as per plan

Of the above statements, the correct one(s) is(are):

(a) (i) only (b) (i) and (iii)

(c) (ii) and (iii) (d) (i), (ii) and (iii)

Q.3 The fasteners in gusseted base plate are designed for

(a) 25% column load

(b) 100% of column load

(c) 30% of column load

(d) 50% of column load

Q.4 The minimum thickness of the rectangular base plate is given by,

(a) $t = \sqrt{\frac{2.5 w\gamma_{m0}}{f_y}\left(a^2 - 0.3b^2\right)}$

(b) $t = \sqrt{\frac{1.2 M\gamma_{m0}}{f_y}\left(a^2 - 0.3b^2\right)}$

(c) $t = \sqrt{\frac{M\gamma_{m0}}{f_y}}$

(d) $t = \sqrt{\frac{3wa^2\gamma_{m0}}{f_y}}$

Q.5 The thickness of base plate is assessed from:
(a) Shear capacity of the plate
(b) Flexural capacity of the plate
(c) Bearing strength of concrete
(d) Two way shear strength of concrete

Q.6 The outstand of gusset plate from the column flange edge is limited to ______ because of buckling consideration.
(a) 9.4εt
(b) 16.6εt
(c) 8.4εt
(d) 13.6εt

Answers

1. (a) 2. (c) 3. (d) 4. (a) 5. (b) 6. (d)

Conventional Practice Questions

Q.1 Design a suitable base plate for steel column carrying a factored axial load of 2500 kN. The effective length of the column is 4.2 m. Use concrete of grade M25 for pedestral supporting the column.

Q.2 Design a column cap for a truss to transfer a reaction of 100 kN to a column section ISHB 400.

Q.3 Design a suitable base plate for a column section ISHB 350 @ 661 N/m supporting an axial load of 450 kN. Use concrete of grade M20 for pedestral supporting the column.

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