UNIT 9 DESIGN OF COLUMN BASES AND GRILLAGES

Structure

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9.1 INTRODUCTION

This unit introduces you to the design of (steel) column bases and grillage foundations. It also describes how columns are connected to their foundations, their detailing and design.

Objectives

After studying this unit, you should be able to

- design various types of column bases and gussetted bases,
- design grillage foundations, and
- design and detail the connections between the column bases and their foundations.

9.2 COLUMN BASES

9.2.1 Foundation Area

All loads which are carried by columns have ultimately to be transmitted to the earth. For proper transmission of loads the safe bearing capacity of the soil has to be determined. Usually the allowable bearing capacity of the foundation soil as determined by proper soil tests will be provided to the designer. This value is much smaller than the permissible stresses in the steel columns. Hence, for a given load, the area of the foundation is much larger than the area of the steel column or stanchion. There are special methods by which the column (or stanchion) is connected to the foundation, so that the axial loads or moments that it carries may be safely distributed over the larger foundation area. The region where the (smaller) column section gradually gets expanded so as to be adaptable for the larger foundation structures is known as the column base. There are various types of column bases depending upon the column and foundation type.
Column bases are important joints, not only from the structural safety point of view but from other considerations also. For example, it must

i) provide proper alignment of columns in plan;
ii) ensure verticality of the columns;
iii) limit the deflections of the columns;
iv) protect the base from floor/ground corrosion;
v) provide proper and firm anchorage to the column end.

### 9.2.2 Common Types of Column Bases

Some of the common types of column bases are shown in Figure 9.1.

**Figure 9.1: Column Bases**

- **Slab Base**: Figure 9.1(a) shows a slab base which is usually a reinforced concrete slab. The steel column is connected (either by rivet or welded joints) to a flat steel base plate which is laid down on that horizontal surface of the RC slab. The base plate is fixed to the slab by means of suitable hold fast (or anchor) bolts.

- **Gussetted Base**: Sometimes the base-plate in the above type of foundation becomes very thick as per the design requirements. In such a case the plate size can be reduced by using a combination of stiffeners.
gusset plates or wing plates as shown in Figure 9.1(b). Such a base is known as gusseted base.

c) **Pocket Bases**: In certain situations the foundations have to be deep foundations e.g., piles or well foundations. In these cases the steel columns are joined to the foundation by means of pocket bases as shown in Figure 9.1(c). As the foundation pressures are taken not by horizontally spread shallow foundations, but by the friction along the vertical well or pile surface, the column is also taken vertically deep instead of having horizontal spread.

d) **Grillage Foundations**: If the bearing capacity of the soil is quite low or the loads transmitted are very high, the foundation area required may be very large. In such situations the gradual expansion from the small column base plate to the large foundation sub-base is done by means of grillage foundations. As shown in Figure 9.1(d), the grillage consists of several tiers of steel beams, the topmost tier is connected with the column base and the lowermost is laid on the prepared foundation sub-base.

In the subsequent sections we shall see how these column bases are designed, after we know the external loads on the column and the safe allowable bearing pressures of the soil.

### 9.3 SLAB BASES

#### 9.3.1 Construction

A reinforced concrete slab base and simple base plate assembly is shown in Figure 9.2. The following points must be noted:

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**Figure 9.2: Bearing Area USBP**

a) column lower end is welded to the steel base plate,

b) grouting is done below the base plate for minor adjustment,

c) anchor bolt heads are provided with proper tightening arrangement (nuts) up to the proof stress level in bolts,

d) the bolt-shafts are encased in a ferrule or pipe which is embedded in the concrete and which is subsequently grouted,

e) for minor alignment of columns, a tolerance of about 2 mm is provided in the bolt-holes,

f) the alignment of columns and levels of the foundation base must be
properly checked before placing the grout below the base plate and in
the ferrule.

9.3.2 Design of Simple Slab Bases

The base plates are designed to withstand the axial load, horizontal shear or
external moments transmitted through the column and must be able to safely
transfer the same to the foundation. The pressure distribution of reaction from
concrete on the underside of the base-plate is assumed to be uniform as shown in
Figure 9.2(a), although theoretically, it may not be exactly so (see Figure 9.2(b)).

The following are the major design considerations:

i) **Size of the Base Plate**

The size of the plate must be large enough to ensure the pressure on the
base concrete \( w \) to be within limits. For columns carrying both axial
load and moment the bearing stress \( w \) on the concrete is

\[
w = \frac{P}{A_b} \pm \frac{M}{Z_b}
\]  

...(9.1)

where,

\( P \) = axial load on the column,

\( M \) = bending moment transmitted at the column bases,

\( A_b \) = area of base (= B.D. for rectangular bases), and

\( Z_b \) = section (= \( \frac{BD^2}{6} \) for rectangular bases).

Thus, if we denote load eccentricity \( e = \frac{M}{P} \), the equation (9.1) above can
be written as follows for rectangular plates

\[
w = \frac{P}{BD} + \frac{6M}{BD^2} = \frac{P}{BD} \left[ 1 + \frac{6e}{D} \right]
\]  

...(9.2)

The allowable bearing stress \( \sigma_{cbe} \) in concrete is given by

\[
\sigma_{cbe} = \sigma_{cc} \sqrt{\frac{A_c}{A_b}}
\]  

...(9.3)

where,

\( \sigma_{cc} \) = allowable compressive stress in concrete = 0.25 \( f_{ck} \) (where \( f_{ck} \)
is the characteristic strength of concrete),

\( A_b \) = area of bearing plate, and

\( A_c \) = area of cross-section of pedestal or area of the largest frustum
of a cone contained in the footing at a slope of 1 (vert.) in
2 (horiz.) in case of sloping/stepped pedestal. (Figure 9.2(c))

[Note: The factor \( \sqrt{\frac{A_c}{A_b}} \) cannot be greater than 2.0]

ii) **Thickness of the Base Plate:** (IS: 800 – 1084, §5.4.3)

When the slab alone distributes the load uniformly the minimum
thickness \( t \) of a rectangular slab shall be given by the following formula
where, \( \sigma_{bs} \) = the permissible bending stress in slab bases (for all steels, shall be assumed as (85 MPa)),

\[ a = \text{the greater projection of the plate beyond column, and} \]

\[ b = \text{the lesser projection of the plate beyond column.} \]

\( (t, a \text{ and } b \text{ are in mm; } w \text{ should be given in MPa}) \)

When the slab does not distribute the loading uniformly (or where the plate is not rectangular), special calculations shall be made to show that the stresses are within the specified limits.

### Example 9.1 (Simple Column Slab-base)

A column of ISMB 400 is subjected to an axial force of 750 kN. Design the base plate. (Assume M-15 pedestal concrete mix)

#### Solution

Axial force \( P = 750 \text{ kN} \)

Flange size of ISMB 400 = 140 mm

Assuming 24 mm bolts, bolt hole = 24 + 1.5 = 25.5 mm

Distance of centre of hole from outer edge = 44 mm

Distance of centre of hole from rolled section = 38 mm

Hence, minimum width of plate \( (B) = 140 + 2(44 + 38) = 304 \text{ mm} \)

Minimum length of plate \( (D) = 400 + 2 (44 + 38) = 564 \text{ mm} \)

Assuming 320 \( \times \) 580 mm size plate for slab base.

We go on to calculate its thickness.

Here both the projections \( a \) and \( b \) as shown in the Figure 9.3 are 90 mm and
the uniform pressure below the plate

\[ w = \frac{P}{B \times D} \]

\[ = \frac{750000}{320 \times 580} \]

\[ = 4.041 \text{ MPa} \]

Hence the thickness of the plate is given by Eq. (9.4).

\[ t = \sqrt{\frac{3 \times 4.04}{185} \left( 90^2 - \frac{90}{4} \right)} \]

\[ = 19.9 \text{ mm} \]

20 mm

Hence 320 \times 580 \times 20 \text{ mm} thick plate is provided.

Next to calculate the size of concrete pedestal, assume 40 mm as cover of reinforcements, dia of bars as 20 mm and ferrule dia as 75 mm; then

Width of pedestal \((B_c) = 240 + 2 \left( 40 + 20 + \frac{75}{2} \right) = 435 \text{ mm} \)

Length of pedestal \((D_c) = 580 + 2 \left( 40 + 20 + \frac{75}{2} \right) = 700 \text{ mm} \)

\(\sigma_{cc}\) for mix M-15 of concrete is 4 MPa

Steel plate area \((A_p) = B \times D = 320 \times 580 = 185600 \text{ mm}^2 \)

Concrete area \((A_c) = B_c \times D_c = 435 \times 700 = 304500 \text{ mm}^2 \)

Allowable bearing stress =

\[ \sigma_{cc} \sqrt{\frac{A_c}{A_p}} = 4 \sqrt{\frac{304500}{185600}} = 4 \times 1.28 = 5.12 \text{ MPa} > 4.04 \]

Hence the given pedestal size is safe.

**SAQ 1**

A column made of 2 ISMC 300 placed face to face at 350 mm outer to outer carries an axial load of 400 kN. Design the slab base foundation for the column.

### 9.4 DESIGN OF GUSSETTED BASES

When the load is very large and the base plate size is to be increased the thickness of the plate from Eq. (9.3) becomes excessive. Hence to reduce the projection lengths \(a\) and \(b\), fasteners like gusset plates, angle cleats and stiffeners etc. are used to (see Figure 9.1(c)). If the column ends are machined for complete bearing on the base plate, half of the axial load is assumed to be transferred to the plate by direct bearing and remaining half through the fasteners. In the case of unfaced
column end and gusset plates, the entire design load is assumed to be transmitted through the fasteners.

![Diagram of column bases and grillages](image)

**Example 9.2**

Design the gusseted base for a column, consisting of an ISHB 250 section with two cover plates $300 \times 25$ mm shown in Figure 9.5. It carries an axial load of 2500 kN and is supported on a concrete pedestal. Permissible bearing pressure of concrete is 4 MPa.

**Solution**

a) **Without Gussets**

Required area of slab base $= \frac{2500000}{4} = 625000$ mm$^2$

Assuming a square base $800 \times 800$ mm

Bearing pressure below base $(w) = \frac{2500000}{800 \times 800} = 3.9$ mm$^2$

The projections of base plate from column in either direction

$a = b = 250$ mm

:. Thickness of base plate $(t) = \sqrt{\frac{3 \cdot w}{\sigma_{bs}} \left(\frac{a^2 - b^2}{4}\right)}$  

... (9.5)

![Diagram of Example 9.2 Without Gussets](image)
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\[ \sqrt{\frac{3 \times 3.9}{185} \left( 250^2 - \frac{250^2}{4} \right)} = 54.4 \text{ mm say 556 mm} \]

b) With Gusset Plates

Using 16 mm gusset plates and ISA 200 × 150 × 12 mm gusset angles as shown in Figure 9.5(b).

Width of base plate = 250 + 2 × 25 + 2 × 16 + 2 × 150 = 632, (say 650 mm)

Length of base plate = \( \frac{625000}{650} \) = 962 mm, (say 1000 mm)

Adopted 650 × 1000 mm base plate as shown in Figure 9.5(b).

Case (i) Column faced for complete bearing

Load taken by fastenings on both faces of column = 0.5 × 2500 = 1250 kN

Using 22 mm dia power driven shop rivets:

Strength in single shear = \( \frac{\pi}{4} (23.5)^2 \times \frac{100}{1000} \) = 43.37 kN

Strength in bearing on 16 mm plate = 16 × 23.5 × \( \frac{300}{1000} \) = 112.8 kN

\[ \therefore \text{ Rivet value} = 43.37 \text{ kN} \]

Number of rivets required = \( \frac{1250}{43.37} \) = 28.8 (say 30 rivets)

Provide 30 rivets 15 on each face, use 3 rows of 5 each which can be accommodated within the 250 mm wide flange for connecting the gusset plate with the gusset angle a minimum of 15 rivets must be used.

Figure 9.5(b): Example 9.2 (With Gussets)
Thickness of plate is checked at critical section X and Y cantilever projection at

\[ X = \frac{650}{2} - \left( \frac{250}{2} + 25 + 16 + 12 \right) = 147 \text{ mm} \]

Max. B.M. at section X = \(4 \times \frac{(147)^2}{2} = 43218 \text{ Nmm/mm}\)

Hence thickness of plate \(t_1 = \sqrt{\frac{43218 \times 6}{185}} = 37.4 \text{ mm}\)

B.M. at Section Y = \(\frac{4 \times 332}{8} - \frac{4 \times 159^2}{4} = 29831 \text{ Nmm/mm}\)

Hence thickness of plate \(t_2 = \sqrt{\frac{29831 \times 6}{185}} = 31.1 \text{ mm}\)

\(:: \text{ Adopt a 32 mm thick plate.}\

Thickness available at X = 32 + 12 = 44 > 37.4 \text{ mm} :: \text{ safe.}\

(Note: Here the plate is assumed to be cantilevered in the first case, and simply supported, with overhangs in the second case. Actually it is supported on three sides, and a bi-planar bending takes place. However by neglecting the bending in other plane our B.M. values are higher and therefore on the safe side.)

**Case (ii) Column not faced for complete bearing**

In this case the fastenings take the full column load.

\(: \text{ Load on fastenings} = 2500 \text{ kN} \)

\(\text{Number of rivets required} = \frac{2500}{43.37} = 57.6\)

Provide 58 rivets, 25 in each flange and 8 in the web

Rivets in web are in double shear

\(\text{Strength of rivet in double shear} = 2 \times 43.37 = 86.74 \text{ kN}\)

\(\text{Strength of rivet in bearing on 6.9 mm web} = 23.5 \times 6.9 \times \frac{300}{1000} = 48.65 \text{ kN}\)

\(\text{Rivet value} = 48.65 \text{ kN (smaller of the two)}\)

Hence safe load on all rivets = \(50 \times 43.37 + 8 \times 48.65 = 2557.7 \text{ kN} > 2500 \text{ kN}\)

The design of the base plate will remain the same as in case (i).

**SAQ 2**

Design a column base for a column carrying an axial load of 3500 kN. It consists of an ISWB 400 section with a 300 x 20 mm flange plate on either flange. The column rests on a concrete foundation (allowable bearing pressure on concrete is 3.5 MPa). Design

a) a gussetted base with machined facing of column end;

b) a gussetted base with unfaced column end with riveted connections.

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**9.5 MOMENT RESISTANT COLUMN BASES**

In bases for column carrying axial loads (P) only were discussed. In this case the
bearing pressure between the base plate and footing is assumed uniform \( w = \frac{P}{A_b} \)

Anchor bolts only hold the column in position and do not take up any loads. If the columns carry bending moments \((M)\) at its ends the bearing pressure follows the linear law

\[
w = \frac{P}{A_b} \pm \frac{M}{Z_b}
\]

and the anchor bolts are sometimes required to take up tensions.

### 9.5.1 Eccentrically Loaded Base Plates

Three cases of eccentrically loaded base plates may arise (Figure 9.6) depending upon the value of eccentricity \((e)\).

- **Case I:** Compression over entire area (Figure 9.6(a)(i))
  
  Here, \( e < L/6 \)

  Anchor bolts are theoretically not required but 16 to 20 mm diameter bolts are provided for keeping the column in position.

- **Case II:** Tension over an area of less than \( \frac{1}{3} \) of the base area (Figure 9.6(a)(ii))

  Here, \( L/6 < e < L/3 \)

  From dimension of column the width of base plate \((B)\) may be assumed. The length of the base plate \((L)\) is then given by the formula
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\[ L = \frac{4P}{3B \sigma_{c_b}} + 2e \]  \( \text{...}(9.6) \)

The tension in the bolt, for \( e < \frac{L}{3} \), may be neglected.

c) **Case III:** Tension over more than \( \frac{1}{3} \) of the base area (Figure 9.6(a)(iii))

In this case \( L \) can be found out by adopting the positive sign in Eq. (9.2)

\[ L = \frac{P}{2B \sigma_{c_b}} \sqrt{\left(\frac{\frac{P}{2B \sigma_{c_b}}}{}\right)^2 + \left(\frac{6P}{B \sigma_{c_b}}\right)} \]  \( \text{...}(9.7) \)

The area \( (A_s) \) and number \( (n) \) of the anchor bolts in tension can be determined by taking moments of all forces about the centre of the compressive zone in foundation block and increasing it by 40% for consideration of stability. Here,

\[ A_{s \text{(net)}} = \frac{1.4T}{n \sigma_f} \]  \( \text{...}(9.8(a)) \)

\[ A_{s \text{(gross)}} = A_{s \text{(net)}} \times 0.75 \]  \( \text{...}(9.8(b)) \)

The gross area of the bolt is obtained by dividing it with a factor 0.75

Here, \( \sigma_f \) = permissible tensile stress in bolt

and \[ T = \frac{M - Pa}{b} \]  \( \text{...}(9.9) \)

where,

\[ a = \frac{1}{2} - \frac{x}{3}, \quad b = L - e - \frac{x}{3} \]  \( \text{...}(9.10) \)

where, \( e \) = edge distance of the bolt holes

and \[ x = \frac{\sigma_{\text{conc(max)}}}{\sigma_{\text{conc(max)}} + \sigma_{\text{conc(min)}}} \cdot L \]  \( \text{...}(9.11) \)

The maximum and minimum concrete stresses are obtained from formula (9.2).

The base plates may be either (a) unattached to the column, or (b) attacked to it firmly by welding or riveting.

The latter may have either (i) initially tensioned bolts, or (ii) the bolts are untensioned.

Some of these types are discussed here.

**9.5.2 Attached Base Plate with Initially Tensioned Bolts**

If \( T \) is the total initial pretensioning force in all the anchor bolts, the pressure between the base plate and footing when column is unloaded will be \( T/bd \). On application of an axial load \( P \) and a bending moment \( M \) the pressure distribution is given by (see Figure 9.6(b))

\[ p = \frac{T + P}{bd} \pm \frac{6M}{bd^2} \]  \( \text{...}(9.12) \)

If the permissible base pressure is \( w \), and no tensile (negative) pressure is allowed we have
This gives
\[ \frac{bd}{w} = \frac{12M}{P} \] ...
\[(9.13)\]

and
\[ T = \frac{wbd}{2} - P \] ...
\[(9.14)\]

The first equation helps us to choose the size of rectangular footing \( b \times d \) and the second gives us the value of the pretensioning force \( T \). If the anchor bolts are tightened to their permissible stress \( f \), the area of bolts is given by \( \frac{T}{f} \) and knowing their diameter and their numbers can be calculated.

The exact design considering the actual support conditions of the base plate (along the gussets) may be complicated. For simplicity the following assumptions are
made, which were towards safety, and can therefore, be safely adopted.

i) The downward reaction due to bolt-tension are ignored and only the upward reaction from footing is considered.

ii) The critical section for maximum B.M. is assumed at 0.95D and 0.8B of the column section as shown in Figure 9.7.

Example 9.3 (Column End with Bending Moment)

A column made up of ISMB 400 section is subjected to an axial load of 480 kN and a bending moment of 60 kNm at its base. Design the slab base for the column.

Figure 9.8 (Example 9.3)

Solution

Assuming 32 mm dia bolts, we have

Distance of bolt centre from plate edge = 57 mm
Distance of bolt centre from column edge = 51 mm

Minimum plate width \((B)\) = 140 + 2 (57 + 51) = 356 mm
Minimum plate length \((D)\) = 400 + 2 (57 + 51) = 616 mm

To allow for the stresses due to B.M., we assume a larger plate size e.g. 400 x 700 mm

Bearing pressure \((w)\) under plate is not uniform

\[ w = \frac{P}{A} \pm \frac{M}{Z} = \frac{P}{BD} \pm \frac{6M}{BD^2} \]
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\[
\frac{480000}{400 \times 700} \pm \frac{6 \times 50000000}{400 \times (700)^2} = 1.71 \pm 1.84
\]

= 3.55 MPa (comp) & 0.13 MPa (tensile)

The latter has almost negligible value and is within safe limit. Using mix M-15 of concrete for which the allowable compressive stress \(\sigma_{ec} = 4\) MPa.

Maximum B.M. in base plate due to bearing pressure at the edge of column (B) is due to the trapezoidal loading of 3.55 to 2.79 MPa over a length of 150 mm.

Hence, Maximum B.M. at point (B) for each mm width of plate (m)

\[
= \frac{2.79 + 2 \times 3.55}{6} \times (150)^2 = 37088 \text{ Nmm/mm}
\]

Allowable bending stress in steel = 185 MPa

\[.: \text{Thickness of plate required} = \sqrt{\frac{6M}{B\sigma_{st}}} = \sqrt{\frac{6 \times 37099}{1 \times 185}} = 35 \text{ mm}\]

(Sometimes two plates 20 mm & 15 mm thick welded together may give 35 mm.)

**Example 9.4**

Design a column base for a column of ISHB 300 section. It carries an axial load of 300 kN and a bending moment of 50 kNm in the plane of the web.

The bolts are having a pretension of 150 MPa. (Allowable bearing pressure on footing is 4 MPa.)

**Solution**

\[w = 4 \text{ MPa} \quad M = 50 \text{ kNm} = 5000000 \text{ N.cm}\]

We have \(bd^2 = \frac{12M}{w} = \frac{12 \times 50000000}{4} = 150000000 \text{ mm}^2\)

Assuming \(b = 500\) we have \(d = 547\) (say 550 mm)

Concrete base may be 500 x 550 mm square in plan.

\[T = \frac{1}{2} wbd - P = \frac{1}{2} \times 4 \times 500 \times 550 - 300000 = 250000 \text{ N} = 250 \text{ kN}\]

Area of bolts = \(\frac{250000}{142} = 1760 \text{ mm}^2\)

Use 4 Nos. of 25 mm dia bolts.

The critical section of the base plate will be

\[\frac{1}{2} \times 0.95 \times 300 = 142.5 \text{ mm from the centre or}\]

\[250 - 142.5 = 107.5 \text{ mm from the edge}\]

To calculate actual pressure distribution below plate

\[w = \frac{T + P}{A} \pm \frac{M}{Z}\]
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(a) Elevation

(b) Bearing Pressure Diagram

(c) Plane of Base Plate

Figure 9.9: Example 9.4

\[ \frac{B.M.}{\text{per mm width}} = (3.14 + 2 \times 4) \times \frac{(107.5)^2}{6} = 21456 \text{ Nmm} \]

Use 500 $\times$ 550 $\times$ 30 mm base plate.

9.5.3 Attached Base Plates with Untensioned Bolts

If the bending moment is large and a post of the pressure distribution below base plate in the Eqn. \( w = \frac{P}{A} \pm \frac{M}{Z} \) is negative (tensile), the anchor bolt on the tension side is expected to take the tension as concrete cannot take tension. Compressive stress in the anchor bolt on compression side is ignored and the section is analysed as in similar reinforced concrete sections (see Figure 9.10).

From the strain diagram in Figure 9.10.

\[ \frac{w}{E_c} \left(1 - \frac{n}{E_s} \right) = \frac{f_s}{E_s} \]

\[ \text{...}(i) \]
Also \( \Sigma \) vertical forces \( \equiv 0 \) \( \Rightarrow \) \( C - T = P \) \( \ldots \)(ii)

\[
\frac{1}{2} n b w - T = P
\]

Taking moments about tensile bolt (A)...

\[
\frac{1}{2} n b w \left( h - \frac{1}{3} h b \right) = P a + M
\]

or

\[
\frac{1}{2} n \left( 1 - \frac{n}{3} \right) b h^2 w = P a + M \ldots \)(iii)

From these equations assuming the dimensions \( b \) and \( h \) of the plate we can get \( n \) and \( T \) and the area of the bolts.

**Example 9.5**

Design a column base for an ISHB 300 section column carrying an axial load of 250 kN and a B.M. of 75 kN in the plan of the web. (bearing pressure on footing is 4 MPa).

**Solution**

Area reqd. for axial load \( \frac{250000}{4} = 62500 \text{ mm}^2 \), but due to large load

\[
\text{eccentricity } \frac{M}{P} = \frac{75 \times 10^6}{250 \times 10^3} = 300 \text{ mm}, \text{ a base plate of size } 600 \times 500 \text{ mm may
Taking moment about the tensile anchor bolt (B) from Eq. (iii)

\[ \frac{1}{2} n \left(1 - \frac{n}{3}\right) \times 500 \times (525)^2 \times 4 = 250000 \times 225 + 75000000 \]

or \( n^2 - 3n + 1.32 = 0 \)

giving \( n = 0.535 \)

\[ \text{Depth of pressure triangle} = nh = 0.535 \times 525 = 281 \text{ mm} \]

From Eq. (ii)

Tension in bolts, \( T = \frac{1}{2} \times 281 \times 500 \times 4 - 250000 = 3100 \text{ N} \)

Taking the modular ratio (m) \( \frac{E_s}{E_c} = 18 \), we get from Eq. (i)

\[ f_s = 4 \times \frac{1 - n}{n} \times 18 = 4 \times \frac{1 - 0.5356}{0.535} \times 18 = 62.58 \text{ MPa} \]

Area of the bolts = \( \frac{31000}{62.58} = 49.4 \text{ mm}^2 \)

\[ \therefore \text{ Provide 2 bolts of 25 mm dia on each side. Thickness of plate is} \]

determined by taking moments of compressive forces and tensile forces about the respective critical sections which are 0.95 of the depth of column, (i.e. \( 0.95 \times 300 = 285 \text{ mm} \))

The distance of critical sections (\( X \)) or (\( Y \)) from the beam edge
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\[ \frac{1}{2} (600 - 0.95 \times 300) = 157.5 \text{ mm} \]

Hogging moment at section \((X)\) = \[31000 \times (157.5 - 75) = 2557500 \text{ Nmm}\]

Sagging moment of section \((Y)\)

\[ = \frac{1.76 + 2 \times 4}{6} = (157.5)^2 \times 500 = 20175750 \text{ Nmm} \]

The latter being larger, it is taken for calculating thickness \((t)\)

\[ t = \sqrt{\frac{20175750 \times 6}{500 \times 185}} = 36.2 \text{ mm (say 38 mm plate)} \]

500 x 600 mm size is adopted.

SAQ 3

A column made of 2 ISMC-300 channel sections placed face to face at 350 mm outer to outer edge carries an axial load of 300 kN and a bending moment of 80 kNm at base. Design the slab base.

9.6 JOINTS BETWEEN COLUMN AND BASE PLATES

The column may carry axial load only or it may be subjected to end moments. The joints may be either riveted or welded. Here two examples of welded column-base plate joints are given.

Example 9.6

An ISMB 400 column is subjected to an axial load of 750 kN. Design a joint between the column and base plate of size 360 x 620 x 28 mm.

Solution

Assuming a 6 mm fillet-weld connection gross length available for welding along the periphery of the ISMB 400 joint is

![Figure 9.12: Example 9.6](image-url)
Effective weld length = Gross length – 2 × end returns of weld (2 sizes at each end)

\[ = 1310 - 2 (4 + 2) 2a = 1310 - 24a \]

\[ = 1310 - 24 \times 6 = 1166 \text{ mm} \]

Allowable stress in weld in direct compression \((\sigma_c) = 150 \text{ MPa}\)

Weld capacity \(F_w = 0.707 \sigma_c = 0.707 \times 6 \times 150 = 636 \text{ N/mm}\)

Weld length required

\[ = \frac{P}{F_w} = \frac{750000}{636} = 1180 \text{ mm} > 1166 \text{ mm} \]

∴ Weld size is not adequate.

∴ Use 8 mm weld and check.

\[ F_w = 0.707 \times 8 \times 150 = 850 \text{ N/mm} \]

\[ L_{\text{reqd}} = \frac{750000}{850} = 890 \text{ mm} \]

\[ L_{\text{provided}} = 1310 - 24 \times 8 = 1118 \text{ mm} > 890 \text{ mm} \:
∴ \text{OK} \]

Example 9.7

An ISMB 400 column is to be connected to a rectangular base plate of size 400 x 800 x 25 along with two parallel stiffeners 160 x 25 mm as shown in Figure 9.13. The column carries an axial load of 480 kN and a bending moment of 250 kNm (due to earthquake forces).

Design a weld size for the connection.

Solution

The load can be transferred either (i) to the gusset plates first and then to the base plate through the gusset or (ii) to the base plate directly if the gusset plates are stiffeners of base-plate only.

Flanges being thicker are having heavier weld sizes \((a = 16 \text{ mm})\).

Web-welds are 8 mm size and gusset plates have 12 mm welds.

Length of weld on each flange \(b_f = 140 - 32 + 140 - 32 = 175 \text{ mm}\)

(allowing for the returns)

Length of weld for web \(b_w = 2 [400 - 32 - 2 \times 2 \times 8] = 672 \text{ mm}\)
Sectional properties of the weld:

Cross-sectional area ($A_w$) = $2 \times 175 \times 16 + 2 \times 336 \times 8 = 10976 \text{ mm}^2$

M.I. of welds about section major axis ($I_w$)

\[
I_w = \frac{1}{12} \times 2 \times 8 \times 336^3 + 2 \times 175 \times 16 \left( \frac{400}{2} \right)^2 = 50577408 + 224 \times 10^6
\]

\[
= 27457.7 \times 10^4 \text{ mm}^4
\]

Distance of outer weld of flange ($y$) = $200 + 8 = 208$

Stresses due to axial force & B.M.

\[
\sigma = \frac{P}{A_w} + \frac{M_y}{I_w} = \frac{480000}{10976} + \frac{250 \times 10^6 \times 208}{274577 \times 10^4} = 44 + 189 = 233 \text{ N/mm}^2
\]

Stress on the throat of the weld $\sigma_t = \frac{\sigma}{0.707} = \frac{233}{0.707} = 303 \text{ N/mm}^2$

Allowable stress in seismic load condition = $1.33 \times 165 = 219 \text{ N/mm} < \sigma_t$

**SAQ 4**

Design a suitable welded connection for the fastenings of the gussetted base for the column shown in Example 9.2.

### 9.7 DESIGN OF POCKET BASES

In this type of column base the steel column base instead of spread horizontally, is taken vertically downwards. The stress is transmitted directly through the **end plate** and **shear connectors** welded to the column flanges. The shear connectors and end plates are embedded in concrete. The load is resisted by bearing pressure and two-way shear stress acting on a prism of concrete having an area equal to the end bearing plate.

The total bearing surface area ($A_b$) is given by

\[
A_b = A + n.d.L. \quad \text{...(9.16)}
\]

where, $A = \text{area of end plate}$

$n = \text{number of shear connectors}$

$d = \text{diameter of shear connector}$

$L = \text{length of shear connector}$

Bending is absent in the case of pocket bases.

**Example 9.8 (Pocket Base)**

Design a pocket base type of foundation for a steel column ISMB-400 carrying an axial load of 750 kN. (Assume mix M-15 of concrete in the foundation pocket)
Solution

The beam cross-section is: \( B = 180; D = 400 \) mm

Allowing an overhang of 20 mm all round provide an end plate of 180 x 480 mm x 20 mm thick.

Assuming the diameter of shear connectors as 20 mm, their length,

\[ L = \text{depth of I-section} - 2 \times \text{flange thickness} \]

\[ = 400 - 2 \times 16 \text{ mm} = 368 \text{ mm} \]

If \( n \) = the number of shear connectors, then total bearing surface area provided

\[ = A + ndL \]

\[ = (180 \times 480 + n \times 20 \times 368) \text{ mm}^2 \]

Provide a concrete trench area of 200 x 500 mm for the pocket base.

For mix M-15 concrete \( \sigma_{cc} = 4.0 \) MPa

Allowable bearing stress \( = \sigma_{cc} \sqrt{\frac{A_c}{A_p}} = 4.0 \sqrt{\frac{200 \times 500}{180 \times 480}} = 4.3 \text{ MPa} \)
:. Total resistance provided by the pocket base
\[ = 4.3 \times [180 \times 480 + n \times 20 \times 368] \text{ Newtons} \]

Equating this to the external column load
\[ P = 750 \text{kN} = 750 \times 10^3 \text{ Newtons} \]
we have
\[ 750000 = 4.3 \times [180 \times 480 + n \times 20 \times 368], \]
giving
\[ n = 11.9 \text{ say 12 members.} \]

Provide 6 shear connectors on each side of the web of the column.
(see Figure 9.14)

Spacing of connectors = \(5d = 5 \times 20 = 100\) mm
End cover = 200 mm of concrete below end plate.

Size of the pocket hole provided 200 \(\times\) 500 mm \(\times\) 1000 mm deep to be filled by mix M-15 cement concrete.

9.8 GRILLAGE FOUNDATION

When the foundation area required becomes too large, either due to heavy loads, or due to low bearing capacity of soil (or both), base slab type foundations become unsuitable.

One of the methods is to spread the column load to the foundation through a series of steel beams arranged at right angles to each other in two or more tiers.

Finally the beams are encased in lean concrete. Such foundations are called grillage foundations.

For a two tier grillage which is more common, the upper tier beams are connected to the column through a base plate. The lower tier beam which receives the load from the upper tier transfers it finally to the soil.

IS : 800 – has allowed an increase of \(33\frac{1}{3}\%\) over the usual permissible bending stresses in the case of grillage beams. If the loads include wind/earthquake or erection stresses an increase of 50% is allowed. Normally the following conditions have to be fulfilled.

a) Beams are to be unpainted and solidly encased in at least M-15 concrete with 10 mm aggregate,
b) Adjacent beam flanges are to be separated clear at least 75 mm apart by means of suitable separators,

c) Side and top concrete cones should be at least 100 mm all round the edges of beams/flanges.

The process of design will be clear by the following example.

Example 9.9

Design a grillage foundation for a ISHB 400 column with a 300 x 20 mm plate on each flanges.

Total load on the column is 2000 kN. The base plate is 650 x 800 mm. The bearing capacity of the soil is 120 kN/m².

(Permissible bending stress for beams is 165 MPa which may be increased by 33 1/3% for the grillage beams.)

Solution

Total load on column = 2000 kN

Self weight of foundation (assumed 10%) = 200 kN

Total load transmitted to soil = 2200 kN

Area of foundation required = \( \frac{2200}{120} = 18.33 \text{ m}^2 \)

Assuming a 4.3 x 4.3 m square foundation of area = 18.49 m²
Top Tier Beams

Assuming 3 beams in the top-tier, which will be placed at right angles to the longer dimension (800 mm) of the base plate.

Max. total B.M. \(= \frac{W}{8} (l - l_1) = \frac{2000000}{8} (4300 - 650) = 912,500,000 \text{ Nmm}\)

Max. B.M. per beam \(= \frac{912,500,000}{3} = 304,167,000 \text{ Nmm}\)

Permissible bending stress \(= 165 \times \frac{4}{3} = 220 \text{ MPa}\)

Required section modules \(Z = \frac{M}{f} = \frac{304,167,000}{220} = 1383000 \text{ mm}^3 = 1383 \text{ cm}^3\)
Adopt ISLB 500 section (Z = 1543.2 cm³)

Max. total shear force for top tier beams = \( \frac{W}{l} \left( \frac{l - l_1}{2} \right) \)

\[
= \frac{2000000}{4300} \left( \frac{4300 - 650}{2} \right) = 848800 \text{ N}
\]

S.F. per beam = \( \frac{848800}{3} = 282930 \text{ N} \)

Actual shear stress in beam web = \( \frac{282930}{500 \times 9.2} \)

\[
= 61.5 \text{ MPa} < 94.5 \text{ MPa permissible} \quad \therefore \text{OK.}
\]

Check for Vertical Web Buckling

Height of web (h) = 500 – 2 × 14.1 = 471.8 mm

Web thickness (t) = 9.2 mm

Slenderness ratio = \( \frac{h}{t} \sqrt{\frac{t}{h}} = \frac{471.8}{9.2} \sqrt{\frac{9.2}{471.8}} = 88.8 \)

Corresponding permissible compressive stress = 93.8 MPa

Length of spread for load (B) = length of bearing plate + \( \frac{1}{2} \) depth of beam

\[
= 650 + \frac{500}{2} = 900 \text{ mm}
\]

\therefore \quad \text{Corresponding bearing area for each beam} = B \cdot t = 900 \times 9.2 = 8280 \text{ mm}²

Total bearing area provided by three upper tier beams = 3 × 8280 = 24840 mm²

\therefore \quad \text{Bearing stress} = \frac{2000000}{24840} = 80.5 \text{ MPa} < 93.8 \text{ MPa (permissible)}

Check for Diagonal Buckling

For diagonal compression slenderness ratio = \( \frac{h}{t} \sqrt{\frac{t}{h}} = \frac{471.8}{9.2} \sqrt{\frac{9.2}{471.8}} = 125.6 \)

Corresponding allowable compressive stress = 63 MPa > 61.5 MPa (actual shear stress) \therefore \text{OK.}

Design of Bottom Tier Beams

Using 10 Beams in the bottom tier. Maximum bending moment for the bottom tier beams = \( \frac{W}{8} \left( l - l_2 \right) \)

\[
= \frac{2000000}{8} \left( 4300 - 800 \right) = 875,000,000 \text{ Nmm}
\]

\therefore \quad \text{Maximum B.M. per beam} = \frac{875,000,000}{10} = 87,500,000 \text{ Nmm}

Permissible bending stress = 220 MPa

Section modulus required = \( \frac{87,500,000}{220} = 397727 \text{ mm}³ = 397.7 \text{ cm}³ \)

Provide ISMB 250 (section modulus = 410.5 cm³)
Check for Shear

Maximum total shear force for bottom tier beam \( \frac{W}{l} \left( \frac{l - l_2}{2} \right) \)

\[
= \frac{2,000,000}{4,300} \left( \frac{4300 - 800}{2} \right) = 813900 \text{ N}
\]

Shear force per mean \( \frac{813900}{10} = 81390 \text{ N} \)

Shear stress \( = \frac{81390}{250 \times 6.9} = 47.2 \text{ MPa} < 94.5 \text{ MPa} \) (permissible)

Check for Vertical Buckling

Slenderness ratio of web \( \frac{h}{t} \sqrt{3} = \left( \frac{250 - 2 \times 12.5}{6.9} \right) \sqrt{3} = 56.5 \)

Corresponding permissible compressive stress \( = 114.5 \text{ MPa} \)

Effective length for load spread \( = \) length of bearing plate + \( \frac{1}{2} \) depth of beam

\[
= 800 + \frac{250}{2} = 925 \text{ mm}
\]

Corresponding bearing area (each beam) \( = 925 \times 6.9 = 6383 \text{ mm}^2 \)

Corresponding all ten webs \( = 63830 \text{ mm}^2 \)

\[
\therefore \text{ Bearing stress} = \frac{2,000,000}{63,830} = 31.3 \text{ MPa} < \text{ Permissible stress} \therefore \text{ OK.}
\]

Example 9.10 (Combined Grillage Foundation)

A combined grillage foundation is to be designed for two columns (A) & (B) given the following data:

<table>
<thead>
<tr>
<th>Column</th>
<th>Section</th>
<th>Load</th>
</tr>
</thead>
<tbody>
<tr>
<td>(A)</td>
<td>2 x ISLC-350 channels back to back (spacing 220 mm)</td>
<td>1000 kN</td>
</tr>
<tr>
<td>(B)</td>
<td>2 x ISMC-350 channels placed back to back (spacing 220 mm)</td>
<td>1500 kN</td>
</tr>
</tbody>
</table>

Spacing between the columns is 4.5 m centres.

Permissible stresses may be taken as follows:

- bending (in steel beams) \( = 220 \text{ MPa} \)
- direct compression (in steel beams) \( = 252 \text{ MPa} \)
- shearing stress (in steel beams) \( = 94.5 \text{ MPa} \)
- bearing (below base-plates) \( = 4 \text{ MPa} \)
- safe bearing capacity of soil \( = 150 \text{ kN/m}^2 \)

Solution

Load on column (A) \( \ldots \) 1000 kN

Load on column (B) \( \ldots \) 1500 kN
Self weight of foundation (10%) 250 kN

Total load 2750 kN

Safe bearing capacity (soil) = 150 kN/m²

\[ \therefore \text{Area of foundation required} = \frac{2750}{150} = 18.33 \text{ m}² \]

The plan of the foundation is so chosen that the C.G. of the foundation coincides with the C.G. of the loads. This is shown in Figure 9.17. If we assume a minimum distance of 0.5 m to the left of the lighter-loaded column (A), the total length of the foundation comes as 6.4 m

\[ \therefore \text{Width of foundation} = \frac{18.33}{6.4} = 2.86 \text{ m} (= 2.9 \text{ m say}) \]

Area of base plate below column (B) = \( \frac{1,500,000}{4} = 375,000 \text{ mm}² \)

Assuming a base plate of 600 x 650 mm size and similar plate for column (A) also.

**Design of Top Tier Beams**

The Top Tier Beams are parallel to the length of the foundation.

The sub-soil reaction per metre run = \( \frac{1000 + 1500}{6.5} = 390.6 \text{ kN/m} \).

This can be assumed as an inverted overhanging beam supported at the column points (A) & (B). The B.M. and S.F. diagrams are drawn in Figure 9.18, which must be verified by you.

Maximum B.M. for top tier = 780 kNm

Assuming three beams in the top tier.

B.M. per beam = \( \frac{780}{3} = 260 \text{ kNm} \)

Section modulus (z) of each beam = \( \frac{260,000,000}{220} = 1,182,000 \text{ mm}² = 1182 \text{ cm}³ \)

Choose ISMB 450 section (\( Z = 1350.7 \text{ cm}³ \)).
Check for Vertical Buckling under Column (B)

Column load on each beam = $\frac{1500}{3} = 500$ kN.

Effective spread length (B) = Length of base plate + half the depth of beam

$$= 600 + \frac{450}{2} = 825 \text{ mm}$$

Buckling stress = $\frac{500,000}{825 \times 9.4} = 64.5 \text{ MPa}$

[Flange thickness ($t_1$) = 17.4 mm & web thickness ($t_2$) = 9.4 mm]

Slenderness ratio of web = $\frac{h}{t} \sqrt{3} = \left( \frac{450 - 2 \times 17.4}{9.4} \right) \sqrt{3} = 76.5$
Corresponding safe compressive stress = 103 MPa > 64.5 MPa (:. Safe)

Check for Shear

Maximum S.F. = 836.7 kN

S.F. per beam = \( \frac{836.7}{3} = 278.9 \text{ kN} \)

Shear stress = \( \frac{278,900}{450 \times 9.4} = 65.9 \text{ MPa} < 94.5 \text{ MPa (permissible)} (:. \text{ safe}) \)

Check for Diagonal Buckling

Slenderness ratio = \( \frac{h}{t} \sqrt{\frac{15.2}{9.4}} \sqrt{6} = 108.1 \)

Corresponding safe stress = 77 MPa > 65.9 MPa (actual) (:. safe).

Design of Bottom Tier Beams

Maximum B.M. for bottom tier beam = \( \frac{W}{8} (l - l_2) \)

\[ = \frac{2,500,000}{8} [2900 - 650] = 703100000 \text{ Nmm} \]

Providing 12 beams in the lower tier

B.M. per beam = \( \frac{703,100,000}{12} = 58,592,000 \text{ Nmm} \)

Section modulus :required per beam (\( Z \)) = \( \frac{58,592,000}{200} = 266400 \text{ mm}^3 = 266.4 \text{ cm}^3 \)

Use ISLB 250 (\( Z = 297.4 \text{ cm}^3 \))

Check for Vertical Buckling

Load per beam \( \frac{2500000}{12} = 208,300 \text{ N} \)

clear web height, \( h = 250 - 2 \times 8.2 = 233.6 \); thickness \( t = 6.1 \)

slenderness ratio = \( \frac{h}{t} \sqrt{3} = \frac{233.6}{6.1} \sqrt{3} = 66.3 \)

corresponding safe compressive stress = 109.6 MPa

Effective length of beam for vertical buckling

\( = \text{Length of base plate} + \text{half the depth of beam} \)

\( = 650 + \frac{250}{2} = 775 \text{ mm} \)

Buckling stress = \( \frac{208300}{775 \times 601} = 44.1 \text{ MPa} < 109.6 \text{ (permissible stress)} (:. \text{ safe}) \)

Check for Shear

Maximum S.F. for bottom tier = \( \frac{W}{1} \left( \frac{l - l_2}{2} \right) = \frac{2,500,000}{2900} \left( \frac{2900 - 650}{2} \right) = 969400 \text{ N} \)

S.F. per beam = \( \frac{969400}{12} = 8079 \text{ N} \)

Shear stress = \( \frac{8079}{250 \times 6.1} = 53 \text{ MPA} < 94.5 \text{ (permissible shear stress)} (:. \text{ OK}) \)
Check for Diagonal Buckling

Slenderness ratio for diagonal buckling \( \frac{h}{t} \sqrt{\frac{E}{6}} = \frac{233.6}{6.1} \sqrt{6} = 94 \)

Corresponding safe compressive stress = 89.3 MPa > 53.0 MPa

(actual shear stress) safe.

9.9 SUMMARY

All loads carried by columns are ultimately transferred to the Soil. As soil bearing pressures are much lower than steel strength, the relatively smaller column sections are to transfer the loads to a larger soil area. For this purpose suitable column bases and their connections are to be provided. The most common type is a slab base. For heavier loads gussetted bases are to be provided. Special arrangements are to be made for Moment resistant column bases. Grillage foundations are to be provided for very heavy column loads or poor bearing capacity soils, requiring a very large foundation area. The grillage beams which may be in several tiers are designed as (upward) loaded (cantilever) beams supported at the centre (column point).

9.10 ANSWERS TO SAQs

SAQ 1
Refer Example 9.1

SAQ 2
Refer Example 9.2

SAQ 3
Refer Example 9.3
FURTHER READING

*Design of Steel Structures* by M. Raghupathi

*Design of Steel Structures* by Edwin H. Gaylord, Jr. Charles N. Gaylord

*Design of Steel and Timber Structures* by S. Ramanrutham

*Design of Steel Structures—Vol. 1* by Dr. Ramchandra

*Design of Steel Structures* by S. M. A. Kazmi & R. S. Jindal

*Design of Steel Structures* by P. Dayarathnam

*Design of Steel Structures* by Prof. A. S. Arya & J. L. Ajmani

*Steel Structures and Timber Structures Analysis, Design and Details of Structures* by V. N. Vazirani & M. M. Ratwani