UNIT-V

DESIGN OF COLUMN BASES

Column bases transmit the Load to the Foundation. The column Spreads the load on wider area so that the Intensity of bearing pressure on the Foundation block is within the bearing Strength. They are three types of column bases commonly used.

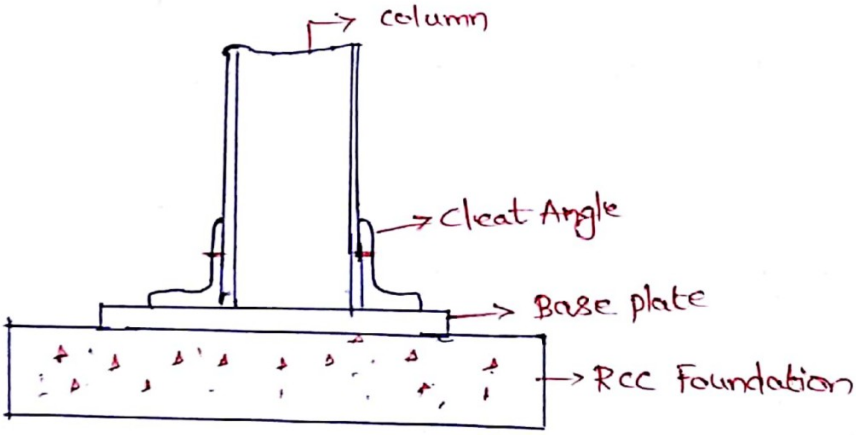
1) Slab base
2) Gusseted base → Axially Loaded / → Eccentrically loaded
3) Grillage foundation

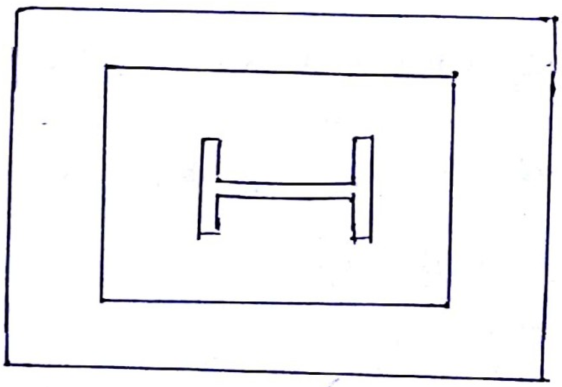
1 SLAB BASE:

→ These are used in columns carrying Large Loads.

→ In this type, the column is directly connected to the base plate through cleat angles.

→ The Load is transfered to the base plate through bearing





SLAB BASE

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P/Q

1) Design a slab base for a column ISHB 350 @ 710·2 N/m Subjected to a Factored axial load of 1500KN for the following Condition i) M20 grade concrete is used for the Foundation ii) provide Welded connection between column and base plate? and use Fe410 grade steel?

Sol Given,

Fe410 grade Steel, $f_u = 410 N/mm^2$

$f_y = 250 N/mm^2$

Partial safety factor $\gamma_{mo} = 1.10$, $\gamma_{mb} = 1.25$ } $P_u = 1500KN$

Column section ISHB 350 @ 710·2 N/m

Properties of section

Depth of section $h = 350mm$; Thickness of Flange $(t_f) = 11.6mm$

Thickness of web $(t_w) = 10.1mm$; Width of Flange $(b_f) = 250mm$

$$\text{Area required for base plate} = \frac{\text{Axial compressive load}}{\text{Bearing strength of concrete}}$$

Bearing strength of concrete $= 0.45 f_{ck}$

$= 0.45 \times 20$

$= 9 N/mm^2$

$$\therefore \text{Area (req)} = \frac{P_u}{0.45 f_{ck}}$$

$$= \frac{1500 \times 10^3}{9} = 166.67 \times 10^3 mm^2$$

Area of base plate required $= 166.67 \times 10^3 mm^2$

Let us Assume (or) provide Rectangular base plate.

Let the sides of base plate 'L & B'

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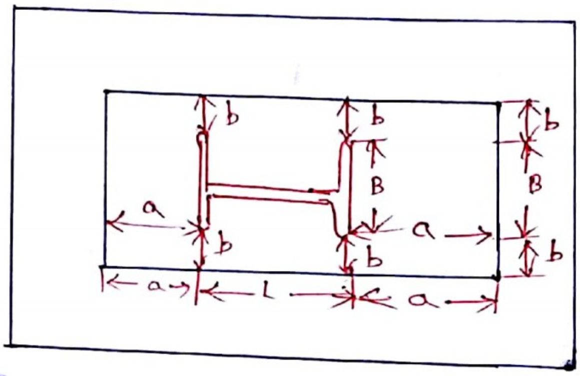
Assume projections of the base plate beyond the column flanges to be 'a' and 'b'.



Area of the base plate = L × B

= (L+2a)(B+2b) [From Figure]

If the projections are taken a=b then,

$A = (L+2a)(B+2a)$

$\Rightarrow 166.67 \times 10^3 = (L+2a)(B+2a)$

Assume L = 350 mm; and B = 250 mm

$\therefore 166.67 \times 10^3 = (350+2a)(250+2a)$

$\Rightarrow a = 55.65 \simeq 60\,mm$

∴ Projections are $a = b = 60mm$

Size of base plate

Length = L+2a = 350+2(60) = 470mm

Breadth = B+2b = 250+2(60) = 370mm

∴ Provide a base plate of 470×370mm.

Thickness of base plate is calculated from IS 800:2007

As per cl. 7.4.3.1/47, $t_s = \sqrt{\dfrac{2.5\,\omega(a^2-0.3b^2)\gamma_{mo}}{f_y}} > t_f$

Where ω = uniform pressure from below on the slab base under Factored load axial compression

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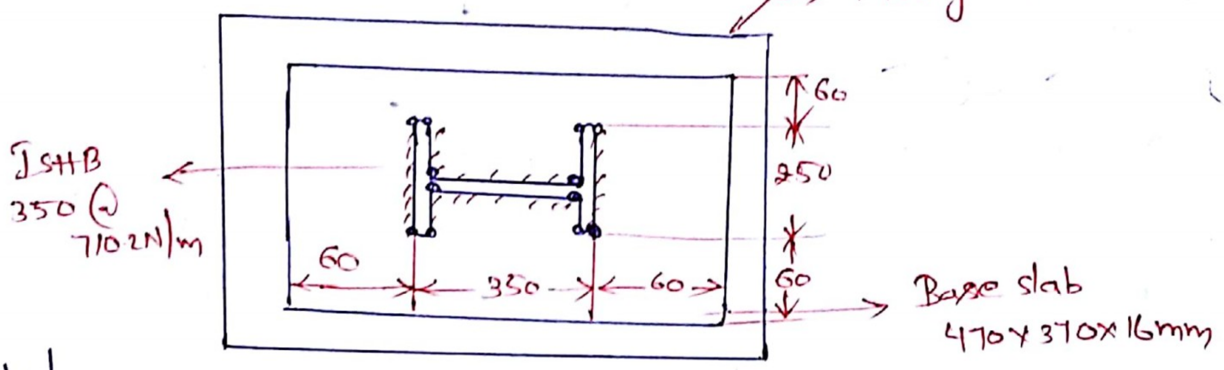
$$\omega = \frac{1500\times10^3}{470\times370} = 8.62\ N/mm^2$$

$$\therefore t_s = \sqrt{2.5\times8.62\left(60^2 - 0.3(60)^2\right)\times\frac{1.10}{250}} > t_f$$

$$t_s = 15.46 \simeq 16mm > 10.6mm \quad (\text{safe})$$

$\therefore$ Provide a base slab of 470 × 370 × 16mm.



Welded.

Connection between Base slab (470×370×16mm) with column Section (ISHB 350)

Total Length of the weld $L_a = (2\times250) + 2(250-10.1) + 2(350-2(11.6))$

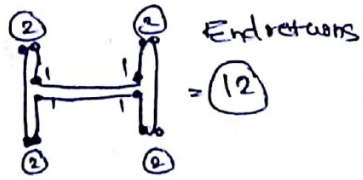
$$\boxed{L_a = 1633.40mm}$$

Let us provide size of fillet weld.

$S_{min} = 5mm$

$S_{max} = 11.6 - 1.5 = 10.1\ mm$ } Adopt S = 8mm fillet weld.

No. of end returns = 12



$\therefore$ Effective Length that can be provided

$= L_a - 2S(\text{No. of end returns})$

$= 1633.40 - 2\times8\times12$

$= 1441.58mm$

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Throat thickness $t_t = 0.7S = 0.7 \times 8 = 5.6mm$

Strength of the weld per (mm) $= l_w \times t_t \times \frac{f_u}{\sqrt{3}\,\gamma_{mw}}$
($l_w = 1mm$)

$= 1 \times 5.6 \times \frac{410}{\sqrt{3}(1.25)}$

$= 1060.47\ N/mm^2$

Length of the weld required to resist Axial load

$= \frac{\text{Axial load}}{\text{Strength of the weld}}$

$= \frac{1500 \times 10^3}{1060.47}$

$= 1414.46\,mm < 1441.58\,mm$ (Safe)

∴ Length of weld required = 1414.46mm ; But the Length of weld provided = 1441.58mm

∴ Provided Length of weld is more than the required Length of weld. (Safe)

2) Design a slab base for a column ISHB 300 @ 58.8kg/m Carrying Factored axial compressive load of 1000kN. The slab base is provided by M25 grade concrete foundation. Use Fe 410 grade Steel and also design welded connection b/w Column and base plate

Sol <-Assignment->

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GUSSETED BASE

→ For columns carrying Large Loads gusseted bases are used.

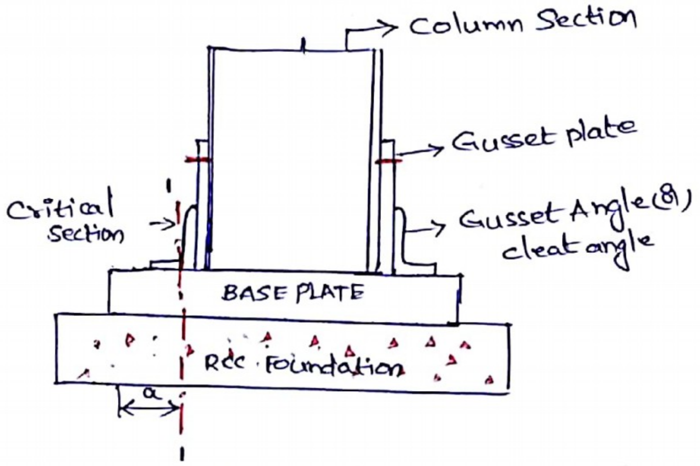
→ Bending moment is present along with Axial load.

There are two cases

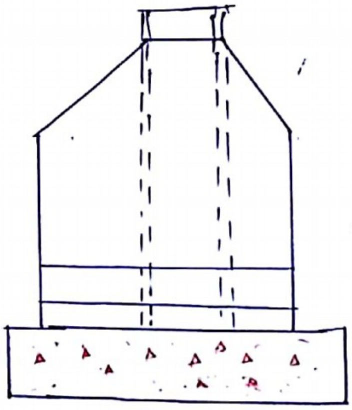
a) Axially Loaded column

b) Eccentrically Loaded Column

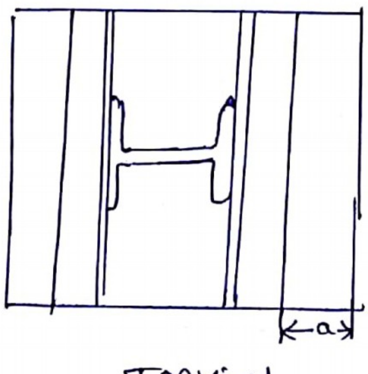
→ Load transfered to the base partly through bearing and partly through gussets.



Front view



Side view



Top view

In a Gusseted base, a part of the load is transmitted from the columns through the gussets to the base slab.

The Gussets and stiffeners bear the base slab against bending and therefore, a thinner base plate can be provided.

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A) AXIALLY LOADED COLUMN:

Design steps for Axially loaded column

Step 1: Calculate Area of base plate $A = \frac{P_u}{0.45 f_{ck}}$

where P_u = Axial compressive load

Allowable compressive stress (or) Bearing strength of concrete = $0.45 f_{ck}$ [f_{ck} = Grade of concrete]

Step 2: Select a suitable Angle section and thickness of gusset plate should not be less than 10mm.

Step 3: Find the size of Gusset plate (or) base plate

i) An unequal angle is generally provided

ii) Minimum width of base plate required

= Depth of section + 2(Thickness of gusset plate)
+ 2(Length of angle leg) + 2(overhang portion)

∴ [overhang should be approximately 1cm to 2cm]

iii) using $A = L \times B \Rightarrow$ calculate $B = \frac{A}{L}$

iv) calculate thickness of Base plate

a) uplift pressure on base plate (w) = $\frac{P_u}{\text{Area of base plate provided}}$

b) Calculate projection 'a'

$$a = \frac{\text{Width of base plate} - \text{Depth of section} - 2(\text{Gusset plate thickness}) - 2(\text{Thickness of angle section})}{2}$$

c) Bending moment @ section 1-1 critical section is calculated

$$M_{1-1} = \frac{Wa^2}{2}$$

$$M_d = \frac{Wa^2}{2}$$

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d) As per cl. 8.2.1.2/53, Moment capacity of base plate + Angle leg

$1.5 Z_e \frac{f_y}{\gamma_{m0}}$ [SSB] (or) $1.2 Z_e \frac{f_y}{\gamma_{m0}}$ [cantilevers]

Where Z_e = Section Modulus = $\frac{bd^2}{6}$ [b=1mm]

e) Equate Moment of resistance to Design Bending moment

$$M_R = M_d$$

$$1.2 \times Z_e \frac{f_y}{\gamma_{m0}} = \frac{wa^2}{2}$$

f) Finally Thickness of base plate is calculated

Step 4: Bolted connections

i) calculate strength of bolt in Single shear [Assume 24mm ϕ bolts]

ii) calculate strength of bolt in bearing.

iii) calculate strength of 24 mm dia bolt.

iv) Assuming end of the column is Machined, then 50% of the total load is transmitted to the base plate through Bearing and Remaining 50% shared by the two gusset plates

v) Load shared by each gusset plate is calculated.

vi) No. of bolts required are calculated.

These are steps involved in Axialy Loaded column of Gusset Base.

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AXIALLY LOADED COLUMN

(P/Q) 1) Design a Gusseted base for the column ISHB 350 @ 710 N/m with two plates 450mm x 20mm carrying a factored load of 3600KN. The column is to be supported on concrete pedestal to be built up with M20 concrete?

Sol Given Data,

Factored load (or) Axial compressive load $P_u = 3600kN$;

Cover plates of two = 450x20mm

Grade of concrete = M20

Column section = ISHB 350 @ 710 N/m

Details of ISHB 350 @ 710 N/m

Depth of section $(h) = 350mm$;

Thickness of Flange $(t_f) = 11.6mm$;

Thickness of web $(t_w) = 10.1 mm$

Width of Flange $(b_f) = 250mm$

STEP 1:

Required Area of the base plate $= \frac{P_u}{0.45 f_{ck}}$ (or) $\frac{\text{Axial comp. load}}{\text{Bearing strength of concrete}}$

$$= \frac{3600 \times 10^3}{0.45 \times 20}$$

$$\boxed{A_{req} = 400 \times 10^3 \, mm^2}$$

Step 2

Let us select an Angle section ISA 150x115x15mm and also Assume Thickness of Gusset plate = 16mm

Step 3: Size of Base plate

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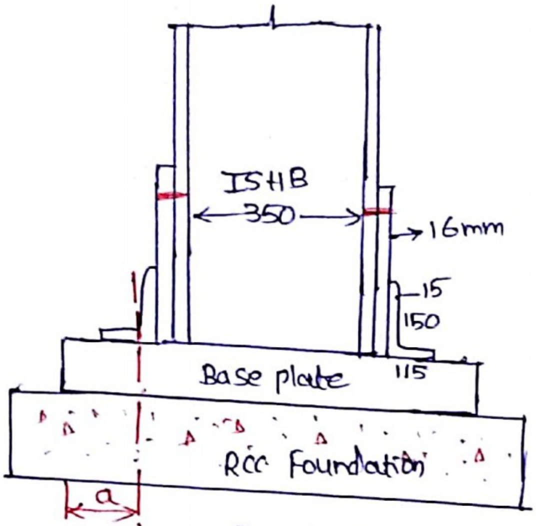
i) Minimum width of base plate required

$= \text{Depth of section} + 2(\text{Thickness of gusset plate})$

$+ 2(\text{Length of angle leg}) + 2(\text{overhang})$



Front view

$\therefore$ Minimum width of base plate $= 350 + 2(16) + 2(115) + 2(20)$

$= 652\,mm$

$\therefore$ provide 700mm wide Base plate

ii) Length of base plate

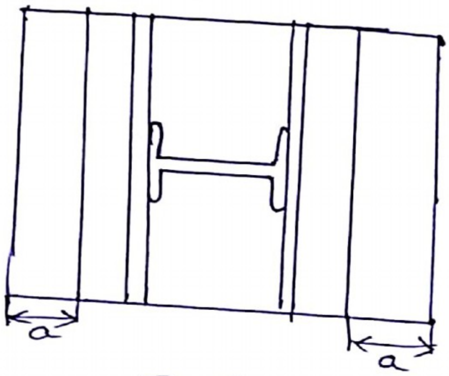
$\text{Area} = L \times B$

$\Rightarrow \text{Length} = \dfrac{\text{Area of baseplate}}{\text{Width of base plate}}$

$= \dfrac{400 \times 10^3}{700}$

$= 571.42\,mm$

$\therefore$ provide 600mm length of base plate



Top view

Area of base plate provided $= 700 \times 600 = 420 \times 10^3\,mm^2 > 400 \times 10^3\,mm^2$ (ok)

iii) Thickness of Base plate

a) uplift pressure on base plate $(\omega) = \dfrac{P_u}{\text{Area of base plate provided}}$

$= \dfrac{3600 \times 10^3}{700 \times 600}$

$\omega = 8.57\,N/mm^2 \cancel{<} 9\,N/mm^2$ (safe)

b) calculate projection

$a = \dfrac{\text{Width of base plate} - \text{Depth of section} - 2(\text{Gusset plate thick}) - 2(\text{Thickness of angle section})}{2}$

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$$a = \frac{700 - 350 - 2(16) - 2(15)}{2}$$

$\boxed{a = 144 \text{ mm}}$

c) Bending moment @ section 1-1 critical section is provide

$$M_{1-1} = \frac{\omega a^2}{2} = \frac{8.57 \times 144^2}{2} = 88.85 \times 10^3 \text{ Nmm.}$$

d) As per cl. 8.2.1.2/53,

Moment capacity of base plate + Angle leg $= 1.2 \times Z_e \frac{f_y}{\gamma_{mo}}$

Equating both. [b = 1mm]

$$\Rightarrow 88.85 \times 10^3 = 1.2 \times \frac{bd^2}{6} \times \frac{250}{1.1}$$

$$\Rightarrow 88.85 \times 10^3 = 1.2 \times \frac{1 \times d^2}{6} \times \frac{250}{1.1}$$

$$\Rightarrow 45.45 d^2 = 88.85 \times 10^3$$

$$d^2 = 1954.89$$

$\boxed{d = 44.21 \text{ mm}}$

∴ Minimum thickness of base plate required $= 44.21 - 15$
$= 29.21$ mm
$\simeq 30$ mm

∴ Provide a base plate of 700 × 600 × 30 mm

Step 4: Connections

Assume 24 mm φ bolts

i) Strength of bolt in single shear $V_{dsb} = \frac{V_{nsb}}{\gamma_{mb}}$
(10.3.3/75)

$$V_{nsb} = \frac{f_u}{\sqrt{3}}(n_n A_{nb} + n_s A_{sb})$$

$$= \frac{410}{\sqrt{3}}\left(0 + 0.78 \times \frac{\pi}{4} \times 24^2\right)$$

$\boxed{V_{nsb} = 83.52 \text{ KN}}$

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$$V_{nsb} = \frac{83.52\times10^3}{1.25} = 66.816\,KN$$

$$\boxed{V_{nsb} = 66.816KN}$$

Strength of bolt in bearing $V_{dpb} = \frac{V_{npb}}{\gamma_{mb}}$
(10.3.4/75)

$V_{npb} = 2.5\,K_b\,d\,t\,f_u$

$d = 24mm$

$d_n = d_0 = 24+2 = 26mm;$

$t = t_f$

$e \nless 1.5d_n = 1.5(26) = 39 \simeq 40mm$

$p \nless 2.5d = 2.5\times24 = 60mm;$

K_b is smaller of $\frac{e}{3d_0}, \frac{p}{3d_0} - 0.25, \frac{f_{ub}}{f_u}, 1.0$

$\frac{40}{3(26)}, \frac{60}{3(26)} - 0.25, \frac{400}{410}, 1.0$

$0.512, 0.519, 0.97, 1.0$

$$\boxed{K_b = 0.512}$$

$\therefore V_{npb} = 2.5\times0.512\times24\times11.6\times410$

$V_{npb} = 146.10KN$

$\therefore V_{dpb} = \frac{146.10}{1.25} = 116.88\,KN$

$$\boxed{V_{dpb} = 116.88KN}$$

$\therefore$ Strength of bolt is smaller of two $= 66.81KN$

Assuming end of the column is Machined, then 50% of total load will be transmitted to the base plate through bearing ie, $3600\times\frac{50}{100} = 1800KN$

Remaining 50% load (1800KN) will be shared by two gusset plates

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Load shared by each gusset plate = $\frac{1800}{2}$ = 900 KN.

Hence the Connection has to be designed for a load of 900 KN

No. of bolts required = $\frac{900}{66.81}$ = 13.47 ≃ 16 bolts.

∴ Provide 16 no. of bolts to connect.

2) For same problem → ISHB 300 @ 58.8 kg/m with 2 plates 350×16mm carrying a factored load of 2000 KN. The column is to be supported on concrete pedestal to be built up with M20 grade concrete?

Sol Assignment

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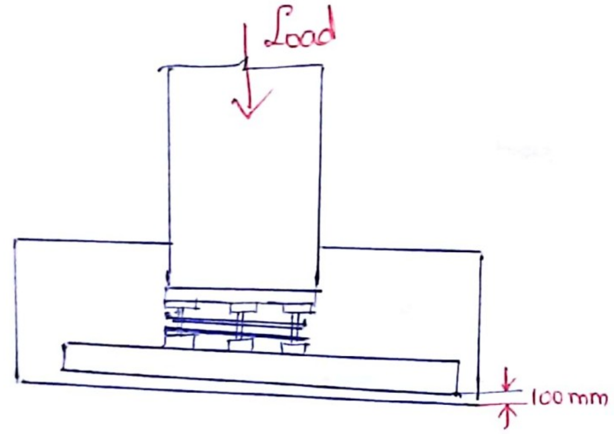
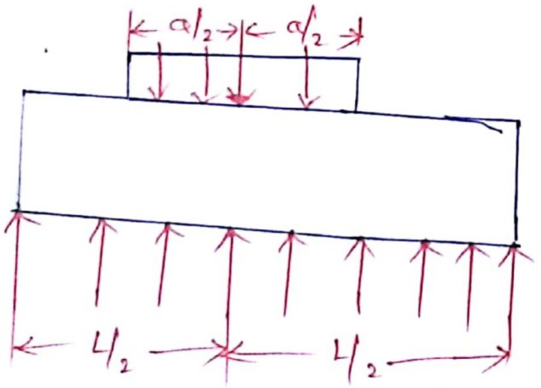
GRILLAGE FOUNDATION

High rise buildings are built with steel ~~beam~~ columns encased in concrete. Such columns carry very heavy loads and they need special foundation to spread the load to soil. The foundation consists of one tier or more tier of I section beams. Such foundation is called Grillage foundation.





If a beam carries a load 'P' pressure under beam is $\frac{P}{L}$ and on it is $\frac{P}{a}$.

$$M = \frac{P}{L} \times \frac{L}{2} \times \frac{L}{4} - \frac{P}{a} \times \frac{a}{2} \times \frac{a}{4}$$

$$\boxed{M = \frac{P(L-a)}{8}}$$

Maximum shear force occurs @ distance 'a' from center of beam

$$\therefore V = \frac{P}{L}\left(\frac{L}{2} - \frac{a}{2}\right) \Rightarrow \boxed{V = \frac{P(L-a)}{2L}}$$

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Design of Grillage foundation

P/o

1) Design a two-tier grillage foundation for a column carrying 2000KN. The size of the base plate is 800 x 800mm. Safe bearing capacity of soil is 200 KN/m²

Sol Given data,

Axial load from column = 2000 KN

Size of the base plate = 800 x 800mm

Safe bearing capacity of soil = 200 KN/m²

The Grillage Foundation is 2-Tier

Load acting on soil:

Loads from column = 2000KN

Assume load from foundation = 200KN

∴ Total load acting on the soil = 2000 + 200 = 2200KN

Safe Bearing of soil = 200 KN/m²

∴ Area of Foundation required = $\frac{\text{Total load acts on soil}}{\text{Safe bearing of soil}}$

$= \frac{2200}{200}$

$= 11 m^2$

Let us assume the foundation is square

Area of the square foundation (L×B) = 11 m²

Assume L = B

$\Rightarrow L^2 = 11 \Rightarrow L = \sqrt{11} = 3.316 \simeq 3.5m$

∴ L = B = 3.5m

∴ Provide size of foundation as 3.5 x 3.5m.

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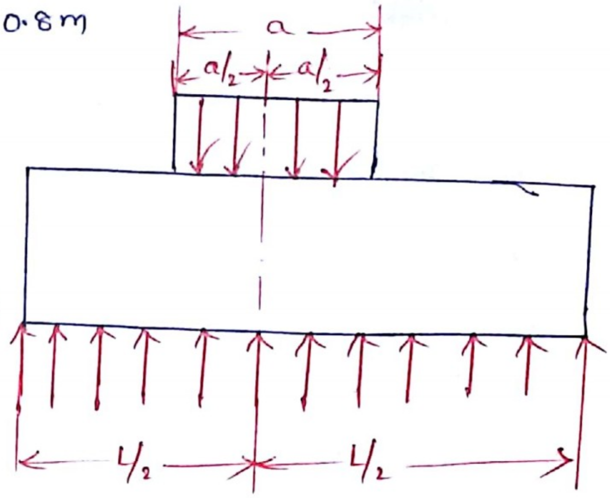
Given side of base plate $a = 800mm = 0.8m$



I. Design of Beam in Upper Tier:

1) Calculation of Factored SF & Factored BM:

Load from column = 2000 KN

Factored load $(P) = 1.5 \times 2000 = 2000 KN$

Maximum Factored shear Force $V_{max} = \frac{P(L-a)}{2L}$

$$= \frac{3000(3.5-0.8)}{2 \times 3.5}$$

$$\boxed{V_{max} = 1157.14 KN}$$

Maximum Factored Bending moment, $M_{max} = \frac{P(L-a)}{8}$

$$= \frac{3000(3.5-0.8)}{8}$$

$$\boxed{M_{max} = 1012.5 KNm}$$

Selection of Trial section:

Section modulus required $(Z_p) = \frac{M_p \gamma_{m0}}{f_y}$

$$= \frac{1012.5 \times 10^6 \times 1.10}{250}$$

$$\boxed{Z_{p(req)} = 4455 \times 10^3 mm^3}$$

Let us provide 3 beams.

$\therefore$ Section Modulus for each beam $= \frac{4455}{3} \times 10^3 = 1485 \times 10^3 mm^3$

Now select a Trial section ISMB 450 @ 72.4 kg/m, having

$Z_p = 1533.4 \times 10^3 mm^3 > (Z_p)_{req} (1485 \times 10^3 mm^3)$

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Properties of ISMB 450 @ 72.4 kg/m

Sectional Area $A = 92.27 cm^2 = 9227 mm^2$

$h = 450mm$; $b_f = 150mm$; $t_f = 17.4mm$; $t_w = 9.4 mm$; $r_{yy} = 30.1 mm$

$r_1 = 15mm$; $I_{xx} = 30390.8 \times 10^4 mm^4$; $I_{yy} = 834 \times 10^4 mm^4$

2) Check for clear spacing:

Width of flange $= b_f = 150mm$

clear spacing $= \left[\frac{800 - 3\times150}{3}\right] = 116.67mm > 75mm$

Hence adequate

3) Classification of section

i) $b = \frac{b_f}{2} = \frac{150}{2} = 75mm$

ii) $\varepsilon = \left(\frac{250}{f_y}\right)^{1/2} = \left(\frac{250}{250}\right)^{1/2} = 1$

iii) $\frac{b}{t_f} = \frac{75}{17.4} = 4.31 < 9.4\varepsilon$

iv) $\frac{d}{t_w} = \frac{385.2}{9.4} = 40.97 < 84\varepsilon$

Hence it belongs to class-I (plastic)

4) Check for Bending strength

Total Factored Bending moment $= 1012.5 KN\text{-}m.$

Bending moment on each beam $= \frac{1012.5}{3} = 337.5 KNm$

But, the design Bending strength $M_d = \beta_b Z_p \frac{f_y}{\gamma_{mo}}$

$= 1 \times 1533.36 \times 10^3 \times \frac{250}{1.1}$

$\boxed{M_d = 348.49 KNm} > M'$ (1012.5KNm)

Hence it is Adequate

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5) Check for Design shear Strength

As per cl. (8.4/59), $V \le V_d$; $V_d = \frac{V_n}{\gamma_{mo}}$

$$V_n = V_p = \frac{A_v f_{yw}}{\sqrt{3}}$$

$$A_v = h t_w = 450 \times 9.4$$

Total Factored shear Force = 1157.14 KN

Shear Force on each beam = $\frac{1157.14}{3}$ = 385.71 KN

$$V_d = \frac{A_v f_{yw}}{\sqrt{3}\,\gamma_{mo}} = \frac{450 \times 9.4 \times 250}{\sqrt{3}(1.1)} = 555\ KN$$

$\underset{(555KN)}{V_d} > \underset{(385.71 KN)}{V}$ (Safe)

Check for Low shear and Low High shear cases

$0.6 V_d$ = ~~0.6(385.71)~~ = 0.6(555) = 333 KN

$\therefore \underset{(385.71)}{V} < \underset{(333)KN}{0.6 V_d}$ → "High shear case"

6) check for Moment capacity in High shear case

As per cl. 9.2.2/70,

$$M_{dv} = M_d - \beta(M_d - M_{fd}) \le \frac{1.2 Z_e f_y}{\gamma_{mo}}$$

Where M_d = Design Bending strength

$$\beta = \left(\frac{2V}{V_d} - 1\right)^2$$

$$= \left(\frac{2 \times 385.71}{555} - 1\right)^2$$

$$\boxed{\beta = 0.152}$$

Z_e = Elastic section Modulus

$= 1350.7\ cm^3$

$Z_e = 1350.7 \times 10^3\ mm^3$

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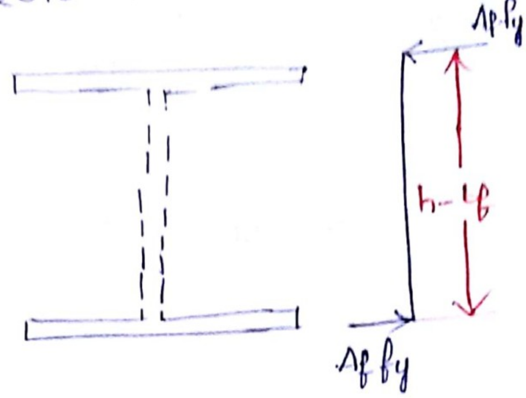
$$M_{fd} = f_y A_f (h - t_f) \times \frac{1}{\gamma_{mo}}$$

Where, $f_y = 250\ N/mm^2$; A_f = Width of the Flange × Thickness of flange ;

$$A_f = 150 \times 17.4 = 2610 mm$$



$$M_{fd} = 250 \times 2610 (450 - 17.4) \times \frac{1}{1.10}$$

$$\boxed{M_{fd} = 256.61\ KNm}$$

$$\therefore M_{dv} = M_d - \beta(M_d - M_{fd}) \leq 1.2\, Z_e \frac{f_y}{\gamma_{mo}}$$

$$= 348.49 - 0.159 (348.49 - 256.61) \leq 1.2 \times 1350.1 \times 10^3 \times \frac{250}{1.1}$$

$$\boxed{M_{dv} = 334.52\ KNm < \underset{(M)}{368.37\ KNm}}$$ Hence safe

7) Check for Web crippling:

As per cl. 8.7.4/67, $F_W = (b_1 + n_2)\, t_w \cdot \frac{f_{yw}}{\gamma_{mo}}$

b_1 = stiff bearing length ~~< t_f (150mm)~~

= width of base plate (a)

$b_1 = 800mm$; $n_2 = [2.5 (t_f + r_1)] 2$

$n_2 = [2.5 (17.4 + 15)] 2 = 162 mm$;

$$\therefore F_W = (800 + 162) \times 9.4 \times \frac{250}{1.1}$$

$$F_W = 2055\ KN > \frac{3000 (V)}{3}$$

$$\boxed{F_W = 2055\ KN > 1000\ KN}$$ Hence safe

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(II) Design of beam in Lower Tier:

1) Calculation of SF & BM

Total Maximum factored shear force $V_{max} = \frac{P(L-a)}{2L}$

$= \frac{3000(3.5-0.8)}{2 \times 3.5}$

$\boxed{V_{max} = 1157.143\ KN}$

Maximum Factored Bending moment $M_{max} = \frac{P(L-a)}{8}$

$= \frac{3000(3.5-0.8)}{8}$

$\boxed{M_{max} = 1012.5\ KNm}$

NOTE: Bottom Tier & Lower Tier beams placed in 3.5m width.

Let us assume width of each beam = width of beam in upper Tier

= 150 mm

clear spacing between beams is not less than 75mm

No. of beams $= \frac{3.5}{\text{width of each beam + clear spacing}}$

$= \frac{3500}{150+75} = 15.5 \simeq 16$ Nos

∴ Assume 16 beams in Lower tier. Hence clear spacing is sufficient

Factored shear force for each beam $= \frac{1157.14}{16} = 72.32\ KN$

Factored Bending moment for each beam $= \frac{1012.5}{16} = 63.28\ KNm$

Section modulus required $Z_p = \frac{M_p\, \gamma_{m0}}{f_y} = \frac{63.28 \times 10^6 \times 1.1}{250}$

$\boxed{Z_{p(req)} = 278.43 \times 10^3\ mm^3}$

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Selection of Trial section {

Try ISMB 225, having $Z_p = 348.27 \times 10^3 > 278.43 \times 10^3 mm$

$h = 225 mm$; $b_f = 110 mm$; $t_f = 11.8 mm$; $t_w = 6.5 mm$; $r_1 = 12 mm$

2) check for clear spacing:

Width of Flange $b_f = 110 mm$

Clear spacing $= \dfrac{3500 - 16 \times 110}{15} = 116 mm > 75 mm$

Hence safe.

3) classification of section:

i) $b = \dfrac{b_f}{2} = \dfrac{110}{2} = 55 mm$

ii) $\varepsilon = \left(\dfrac{250}{f_y}\right)^{1/2} = \left(\dfrac{250}{250}\right)^{1/2} = 1$

iii) $\dfrac{b}{t_f} = \dfrac{55}{11.8} = 4.66 < 9.4\varepsilon = 4.66 < 9.4$

iv) $\dfrac{d}{t_w} = \dfrac{177.4}{6.5} = 27.29 < 84\varepsilon = 27.29 < 84$

Hence it belongs to Class-I and it is plastic

4) check for Bending strength

Total Factored Bending moment $= 1012.5 \, KNm$

Bending moment on each beam $= \dfrac{1012.5}{16} = 63.28 \, KNm$

But, the design Bending strength,

$$M_d = \beta_b Z_p \frac{f_y}{\gamma_{m0}}$$

$$= 1 \times 348.27 \times 10^3 \times \frac{250}{1.1}$$

$\boxed{M_d = 79.15 \, KNm} > 63.28 \, KNm$

Hence safe.

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5) Check for Design Shear Strength:

As per cl. (8.4/59), $V \leq V_d$; $V_d = \frac{V_n}{\gamma_{m0}}$

$V_n = V_p = \frac{A_v f_{yw}}{\sqrt{3}}$

$A_v = h t_w = 225 \times 6.5$

Total Factored shear Force = 1157.14 KN

Shear Force on each beam = $\frac{1157.14}{16} = 72.32$ KN

$\therefore V_d = \frac{A_v f_{yw}}{\sqrt{3} \gamma_{m0}} = \frac{225 \times 6.5 \times 250}{\sqrt{3}(1.1)} = 191.90$ KN

$V_d > V$ (Safe)

(191.90) (72.32) KN

Check for Low shear/High shear case

$0.6 V_d = 0.6(191.90) = 115.14$ KN

$V > 0.6 V_d \longrightarrow$ "Low shear case"

(72.32) (115.14 KN)

6) Check for Web crippling

As per cl. 8.7.4/67, $F_w = (b_1 + n_2) t_w \frac{f_{yw}}{\gamma_{m0}}$

$b_1 = a = 800$ mm; $n_2 = 2[2.5(t_f + r_1)]$

$= 2[2.5(11.8 + 12)]$

$n_2 = 119$ mm

$F_w = (800 + 119)\, 6.5 \times \frac{250}{1.10}$

[Factored load on each beam $= \frac{3000}{16}$]

$= 1357.61 > \frac{3000}{16}$

$\boxed{F_w = 1357.61 > 187.5 \text{ KN.}}$ Hence safe

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Details:

∴ Designed two tier grillage Foundation

1) Upper Tier Designed with 3 NO's of beams of ISMB450

2) Lower Tier Designed with 16 NO's of beams of ISMB 225

3) The concrete Foundation block is of size 3.5 x 3.5m

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