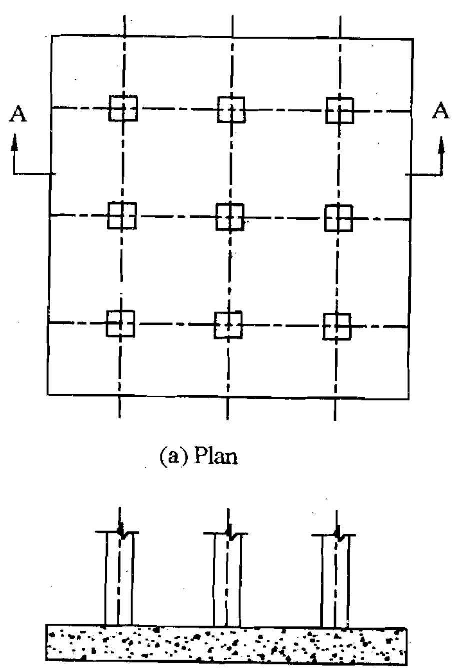
UNIT 18 RAFT FOUNDATIONS

Structure

18.1 INTRODUCTION

An R.C. foundation under a number of columns in a row or rows, in the form of a slab with or without beams is called a *raft foundation.* This type of foundation becomes essential when isolated footings under different columns or combined footing under columns in different rows would have overlapped one over the other due to soil of low bearing capacity. Raft foundation may be of the following types:

a) *Mat Type*: It consists of simple slab under the columns (Figure 18.1). This type is suitable for small loads having columns at nearly uniform and closer spacing.

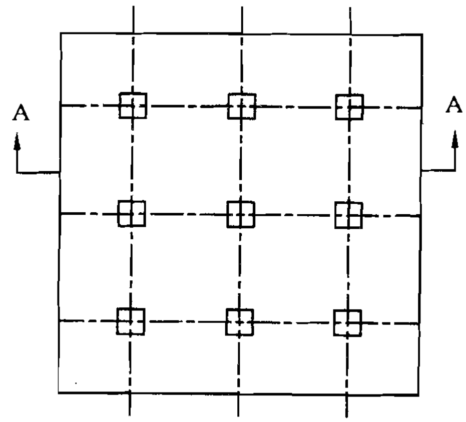


(a) Plan

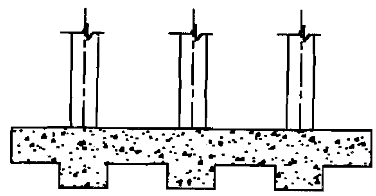
(b) Section at A- A

Figure 18.1: Mat Type Raft Foundation

b) *Slab with Protrusions Type*: It consists of simple slab with protrusions under the columns to resist shear and bending adequately (Figure 18.2). These protrusions may also be in the form of pedestal on top of the slab.



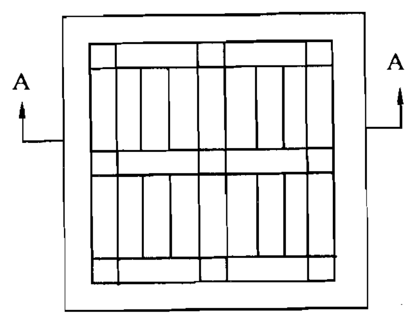
(a) Plan



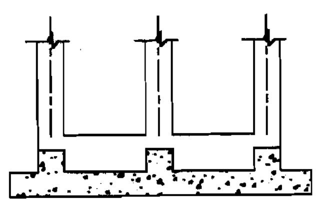
(b) Section at A- A

Figure 18.2: Flat Plate with Protrusions

c) *Slab and Beam Type*: It is just like *inverted roof* having slab supported on beams (Figure 18.3). Such footings are provided where the loads coming through columns are heavier and columns are generally at non-uniform spacings. The beams may be either on top or bottom of the slab depending on architectural requirement.



(a) Plan



(b) Section at A- A

Figure 18.3 : Slab and Beam Type Raft

d) *Cellular or Rigid Frame Type*: This type of foundation is suitable for *very* heavy loads on soil of relatively *high* bearing capacity (Figure 18.4). Sometimes, basement wall, if any, are used as beams of slab and beam type *or* as vertical side of cells in cellular or rigid frame type (Figure 18.5)

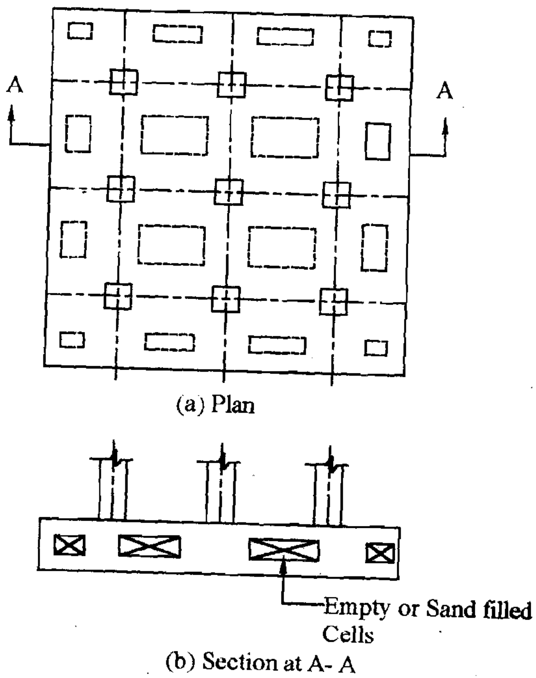


Figure 18.4: Cellular Type Raft Foundation

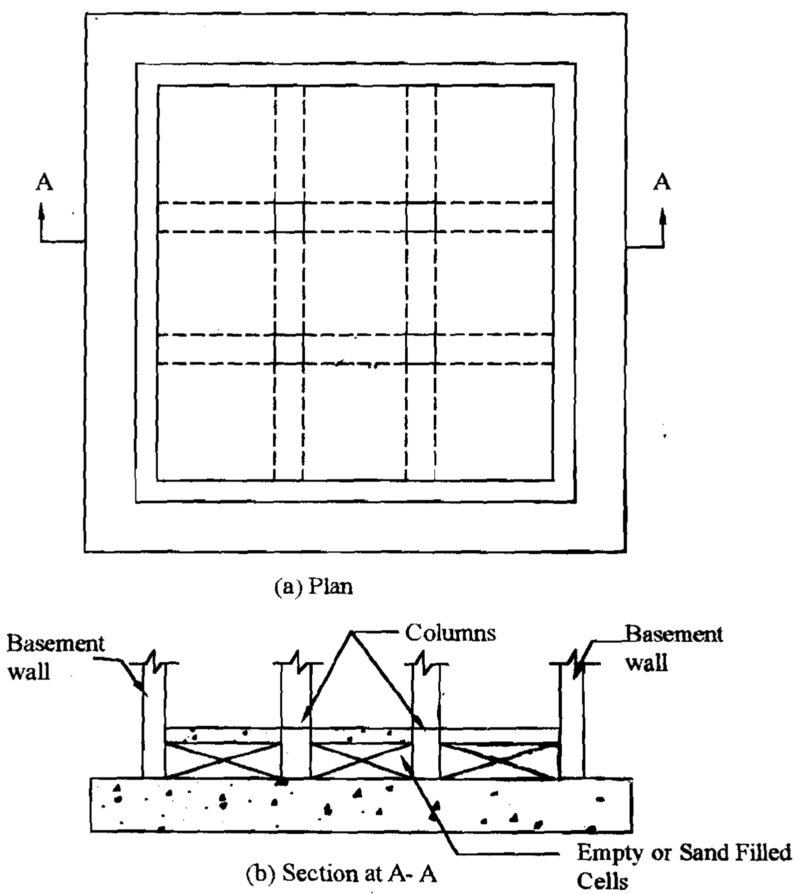


Figure 18.5: Rigid Frame Type Raft Foundation

Objectives

After studying this unit, you should be able to

- know different type of raft foundation, and illustrate *conventional* method of analysis and design of *simple* raft foundation.

18.2 GENERAL DESIGN CONSIDERATIONS

Approach to analysis and design of raft foundation may be either of the following two types:

1) *Rigid Foundation*

For this type, the foundation elements are considered as rigid producing *planar* distribution of soil pressure. In other words, the deflection of the *foundation or mat* does not influence the pressure distribution (Figure 18.6). The pressure distribution (p) under the raft shall be determined by the following formula (Figure 18.7):

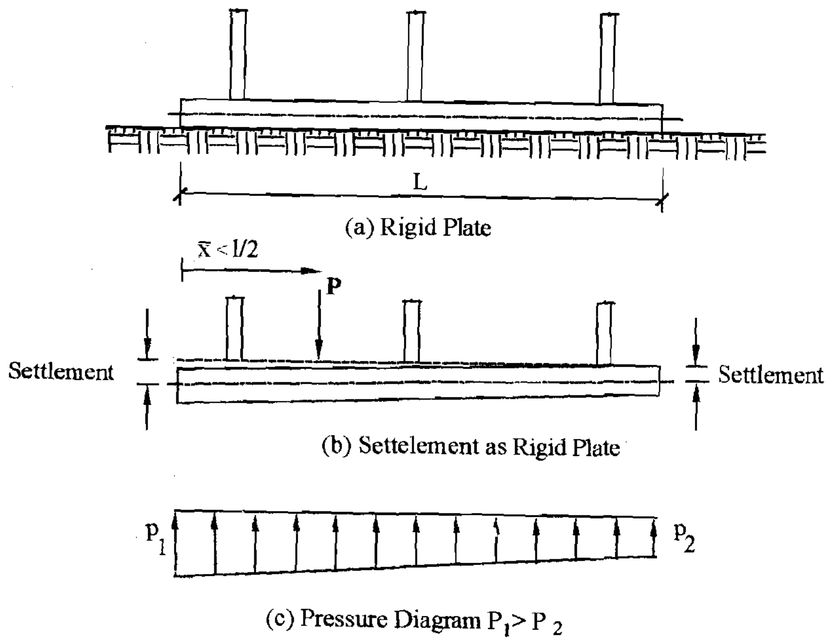


Figure 18.6: Rigid Foundation

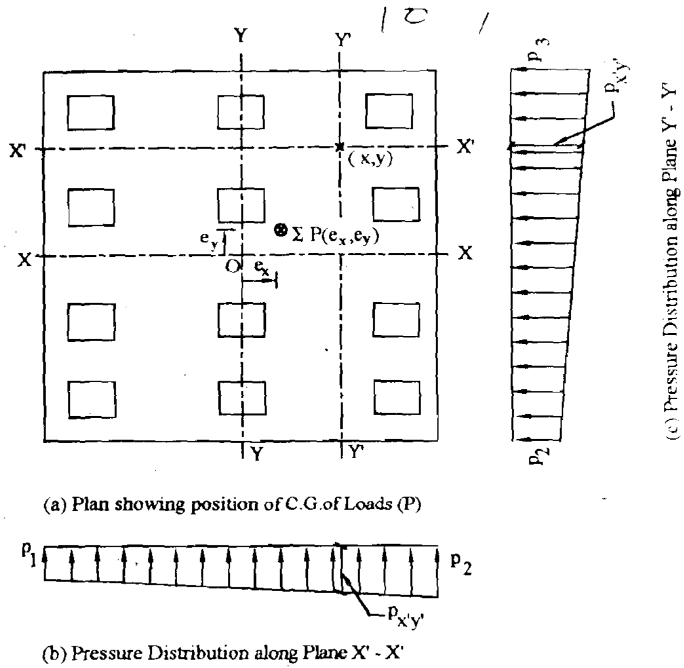


Figure 18.7: General Pressure Distribution Diagrams

$$p = \frac{P}{A} \pm \frac{Pe_y}{I_x} y' \pm \frac{Pe_x}{I_y} x' \qquad \text{.......(18.1)}$$

where,

P = total vertical loads ($\sum P_i$)

A = area of raft

e_x, e_y = eccentricities of total vertical loads w.r.t. X-X and Y-Y axes respectively.

I_x, I_y = moment of inertia about X-X and Y-Y axes respectively.

x', y' = coordinate of the point at which value of p is desired from X'-X' and Y'-Y' axes respectively.

This approach reduces the analysis and design of foundation system to that of an inverted roof–either flat slab type *or* slab and beam type with loading profile given by Equation (18.1).

2) *Flexible Foundation*

In this type the soil mass is considered to be made of springs of different stiffness or mass of dense liquid * (Figure 18.8). The analysis and design is done by finite difference or finite element method. This approach is, therefore, out of scope of this unit as it is not suited to manual design.

Hence only first approach of analysis and design will be illustrated here through the following examples.

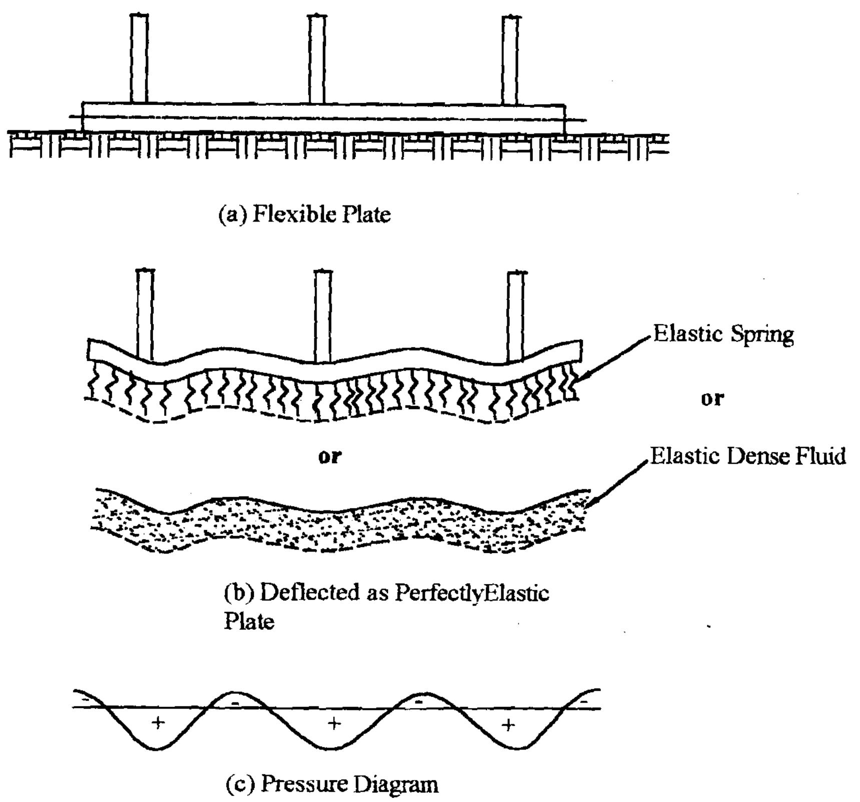


Figure 18.8: Flexible Foundation

* The foundation is assumed to be perfectly elastic. Therefore, the pressure at any point is linearly proportional to the deflection at that point. The spring constant *or* coefficient of subgrade reaction is taken as the pressure required to produce a unit settlement. The spring constant is the same in compression as well as in tension.

Example 18.1

Design a raft foundation for the arrangement of columns shown in the Figure 18.9. The bearing capacity of soil is 175 kN/m^2 and all columns are of the same size 350 × 350 mm. The loading on each outer column is 2000 kN whereas on each interior column it is 2500 kN each. Use M 20 concrete and Fe 415 steel.

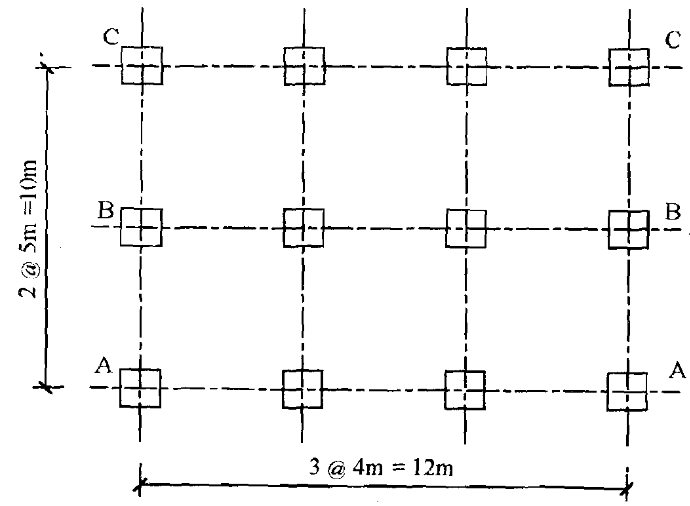


Figure 18.9: Layout of Columns

Solution

Design Constant

For M 20 concrete Fe 415 steel.

$\sigma_{cbc} = 7$ N/mm^2; $\sigma_{st} = 230$ N/mm^2; m = 13; $k_B = 0.283$; $j_B = 0.906$; $R_B = 0.897$; $p_B = 0.431\%$.

Size and General Arrangement of Foundation

Loads

Column loads	= 10 × 2000 + 2 × 2500	= 25000 kN
Self-weight of foundation (Assuming 10% of superimposed load)		= 2500 kN
Total load		= 27500 kN

$$\text{Required area of foundation} = \frac{27500}{175} = 157.14 \text{ m}^2$$

Providing 1m projection from centre line of columns on each side the area of foundation provided = 14 × 12 = 168 m^2 > 157.14 mm^2.

The arrangement of beams width 350 of connecting the columns have been shown in Figure 18.10.

$$\text{The intensity of pressure from soil } p = \frac{\Sigma P_i}{A} = \frac{25000}{168} = 148.8 \text{ kN/m}^2$$

Design of Slabs

Depth

i) **From B.M Considerations**

Cantilever span, a = 0.825 m

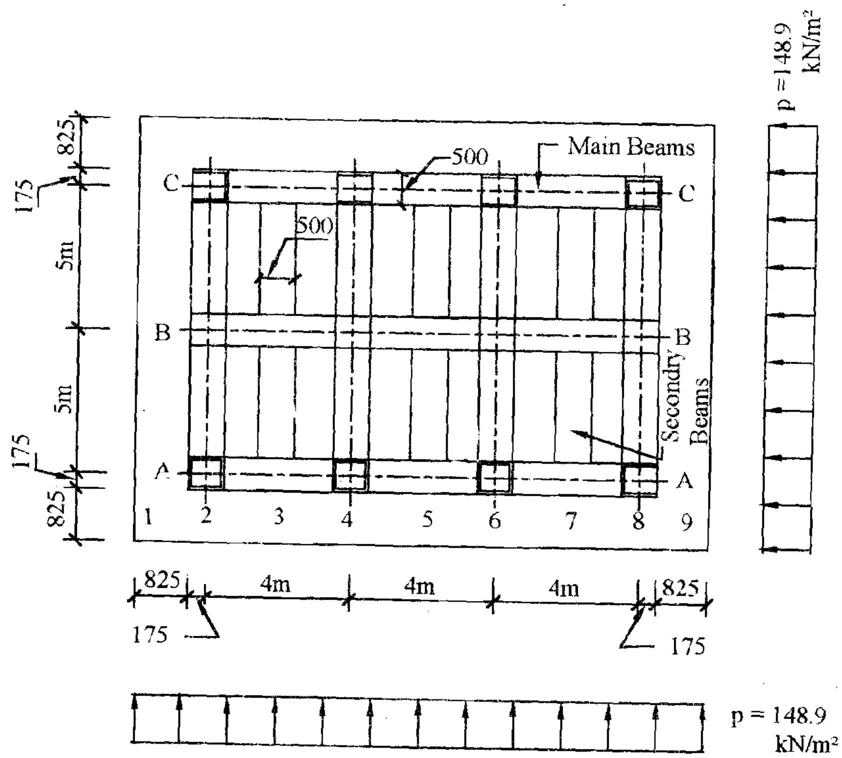


Figure 18.10: Arrangement of Beams and Pressure Diagram of the Raft Foundation

Maximum B.M in cantilever, $M_{max} = \frac{pa^2}{2} = \frac{148.9 \times 0.825^2}{2} = 50.67$ kNm

Maximum B.M. in the continuous slab

$$M_{max} = \frac{pl^2}{10} = \frac{148.9 \times 2^2}{10} = 59.56 \text{ kNm}$$

$$d = \sqrt{\frac{M}{R_b b}} = \sqrt{\frac{59.56 \times 10^6}{0.897 \times 1000}} = 257.68 \text{ mm.}$$

ii) **From S.F. Consideration**

Maximum S.F. in cantilever at distance d from support,

$p(a - d) = 148.9 \times 10^{-3} (825 - d) \times 1000 = 148.9 (825 - d)$ N

$$\text{Maximum S.F. in continuous slab} = 0.6p \times 10^{-3} \times \left(\frac{2 - 0.35}{2} - d\right) \times 1000$$

$$= 0.6 \times 148.9 \times 10^{-3} \left(\frac{2000 - 350}{2} - d\right) \times 1000$$

$$= 89.34 (825 - d) \text{ N} < 148.9 (825 - d)$$

Hence maximum S.F. for calculation of $d = 148.9 (825 - d)$ N

τ_c for $p_B = 0.431\%$

$$= 0.22 + \left(\frac{0.3 - 0.22}{0.25}\right) (0.431 - 0.25) = 0.278 \text{ N/mm}^2$$

$$\tau_c bd = 0.278 \times 1000 \times d$$

Equating maximum shear force with the permissible shear force

$$148.9\,(825 - d) = 0.278 \times 1000 \times d$$

or $\quad 122842.5 - 148.9 \times d = 0.278d \times 10^3$

or $\quad 122842.5 - 148.9d = 278\,d$

or $\quad d = \dfrac{122842.5}{(148.9 + 278)} = 287.75$

Hence provided $D = 340$ and $d = 290$ taking effective cover of 50 mm.

A_{st}

For cantilever slab,

$$A_{st} = \frac{M}{\sigma_{st}\, j_B d} = \frac{50.67 \times 10^6}{230 \times 0.906 \times 290} = 838.49 \text{ mm}^2\text{/m}$$

$$\text{Spacing of } \phi 10 = \frac{78.5 \times 1000}{838.49} = 93.62$$

For continuous slab,

$$A_{st} = \frac{M}{\sigma_{st}\, j_B d} = \frac{59.56 \times 10^6}{230 \times 0.906 \times 290} = 985.6 \text{ mm}^2\text{/m}$$

$$\therefore \text{ Spacing } s = \frac{78.5 \times 1000}{985.6} = 79.65 \text{ mm}^2$$

$$A_{st,min} = \frac{0.12bD}{100} = \frac{0.12 \times 1000 \times 340}{100} = 408 \text{ mm}^2\text{/m}$$

Maximum permissible spacing $s = 3d = 3 \times 290 = 870$ or 450 whichever is less

Hence provided $\phi 10$ @ 75 c/c in both orthogonal directions in cantilever as well as in continuous slabs for hogging and sagging bending.

The detailing of slab reinforcement have been shown in Figure 18.11.

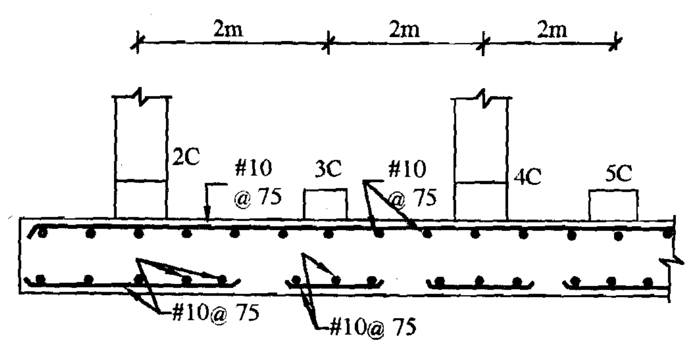


Figure 18.11: Section through Slab

Design of Secondary Beam 5A-5C.

Though the S.F.D. and B.M.D. for the beam shall not comply with those for a two-span continuous beam because of the *known* reaction of main beam*, still the S.F. and B.M. in this

* The reactions of main beams are the applied load through column which generally, do not satisfy statics due to planer distribution of soil pressure.

beam are taken as for *continuous beam* for design purposes to be on conservative side (Figure 18.12).

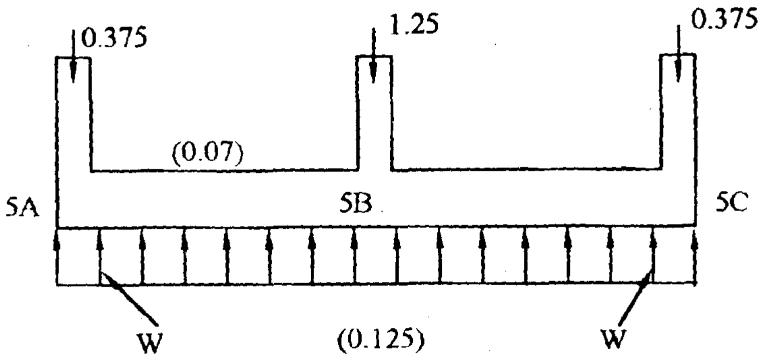


Figure 18.12: Showing Loading and Coefficient of Reactions and B.M.S.

Total Load on *one* span, $W = 148.9 \times 2 \times 5 = 1489$ kN

S.F. at the d from end support

$$= +0.375 \times 148.9 - 1489 \times 2\ (0.175 + 0.29) = 419.89 \text{ kN}$$

S.F. at d from middle support

$$= \frac{1.25}{2} \times 1489 + 148.9 \times 2\ (0.175 + 0.29) = -792.15 \text{ kN}$$

B.M. at mid span

$$= 0.07 \times 1489 \times 5 = 521.15 \text{ kNm}$$

B.M. at the face of middle support

$$= 0.125 \times 1489 \times 5 - 148.9 \times 2 \times \frac{0.175^2}{2} = 926.06 \text{ kNm}$$

$$d = \sqrt{\frac{M}{R_B b}} = \sqrt{\frac{926.06 \times 10^6}{0.897 \times 500}} = 1436.94$$

Taking $D = 1490$ and effective cover of 50, $d = 1440$

Hence provided $D = 1490$ and $d = 1440$

Area of steel A_{st}

For B.M. at the face of middle support

$$A_{st} = \frac{M}{\sigma_{st} j_B d} = \frac{926.06 \times 10^6}{230 \times 0.906 \times 1440} = 3086.17 \text{ mm}^2$$

$$A_{st} = \frac{0.85bd}{f_y} = 0.85 \times 500 \times \frac{1440}{415} = 1474.7 \text{ mm}^2 < 3086.17 \text{ mm}^2$$

Hence provided 10#20

A_{st} for mid span

Beam at this section will behave as T-beam

b_f will be least of the following

i) $\frac{l_0}{6} + b_w + 6D_f = \frac{0.7 \times 5}{6} + 0.35 + 6 \times 0.12 = 1.65$ m

ii) 2 m

$\therefore b_f = 1.65$ m

Assuming $j_B \approx 0.9$

$$A_{st} = \frac{M}{\sigma_{st} j_B d} = \frac{521.15 \times 10^6}{230 \times 0.9 \times 1440} = 1748.35 \text{ mm}^2$$

Hence provided 6#20 (A_{st} = 1884 mm^2)

Provision of Shear Reinforcement

Nominal shear stress at d from face of mid-support,

$$\tau_v = \frac{V}{bd} = \frac{792.15 \times 10^3}{500 \times 1440} = 1.1 \text{ N/mm}^2$$

$$p\% = \frac{A_{st}}{bd} \times 100 = \frac{3141}{500 \times 1440} \times 100 = 0.436$$

$$\tau_c = 0.22 + \frac{(0.3 - 0.22)}{(0.5 - 0.25)} \times (0.436 - 0.25) = 0.279 \text{ N/mm}^2$$

$\therefore$ $V_c = \tau_c bd = 0.279 \times 500 \times 1440 = 200.88$ kN

$\therefore$ $V_s = 792.15 - 200.88 = 591.27$ kN

Taking ϕ10-2 legged stirrups, $A_{sv} = 100$

$$s_v = \frac{\sigma_{sv} A_{sv} d}{V_s} = \frac{230 \times 156 \times 1440}{591.27 \times 10^3} = 87.38$$

s_v shall be the least of

i) $0.75d = 0.75 \times 1440 = 1080$

ii) 450

iii) $s_v = \frac{\sigma_{sv} \times 0.87 f_y}{0.4b} = \frac{100 \times 0.87 \times 415}{0.4 \times 500} = 180.25$

Nominal shear stress at d from face of exterior beam

$$\tau_v = \frac{V}{bd} = \frac{421.39 \times 10^3}{500 \times 1440} = 0.585$$

$$p\% = \frac{A_{st}}{bd} \times 100 = \frac{1884}{500 \times 1440} \times 100 = 0.26\%$$

$$\tau_c = 0.22 + \frac{(0.3 - 0.22)}{(0.5 - 0.25)} \times (0.26 - 0.25) = 0.22 \text{ N/mm}^2$$

$\therefore$ $V_c = \tau_c bd = 0.22 \times 500 \times 1440 \times 10^{-3} = 158.4$ kN

$\therefore \quad V_s = V - V_c = (421.39 - 158.4) = 262.99$ kN

$$s_v = \frac{\sigma_{sv} A_{sv} d}{V_s} = \frac{230 \times 156 \times 1440}{262.99 \times 10^3} = 196.46 > 180.52$$

Hence provided ϕ10-2 legged stirrups @180 throughout

The details have been shown in Figure 18.13.

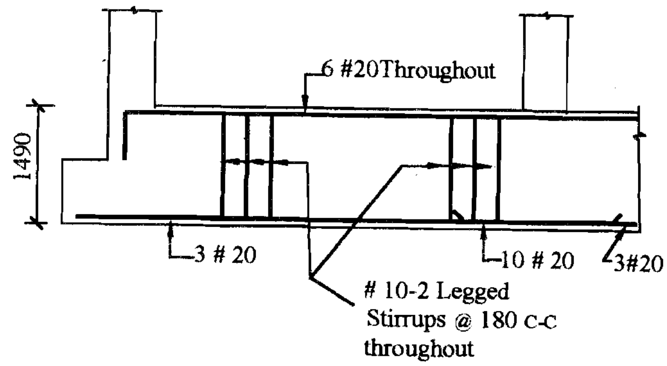


Figure 18.13: Details of Secondary Beam 5A-5C

Design of Main Beam C-C

Loads (Figure 18.14)

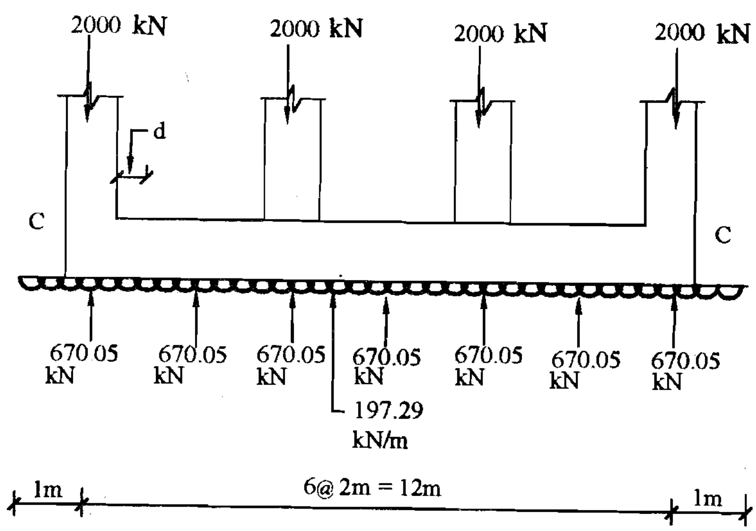


Figure 18.14: Beam C-C′

U.DL on this beam is due to projected slab and pressure directly on the beam over a total length of (0.825 + 0.5) = 1.325 m.

Therefore, *udl*, $w = 148.9 \times 1.325 \times 1 = 197.29$ kN/m

concentrated load due to reaction from the secondary beam* $= \frac{1}{2} \times 2 \times 148.9 \times (5 - 0.5)$

$= 670.05$ kN

* Though the reaction from secondary *continuous* beam is 0.375 W (Figure 18.12), it is taken equal to 0.5 W to be on conservative side due to the fact that beam will be analysed on the approximation that the reactions are the actual column loads.

Statical Check

For equilibrium of beam C-C

$\Sigma f_y = -4 \times 2000 + 670.05 \times 7 + 197.29 \times 14 = -547.59$ kN

$\therefore$ For two exterior main beams the *total* downward load = 2 × 547.59 = 1095.18 kN

For equilibrium of beam B-B (Figure 18.15)

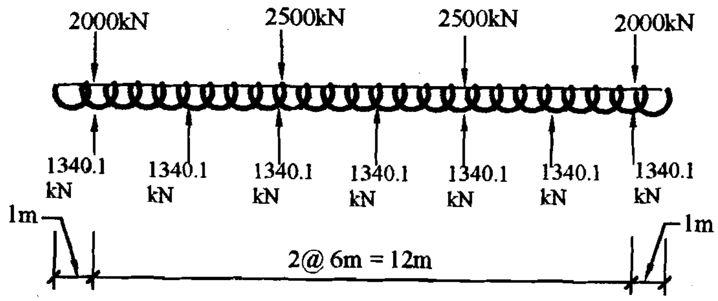


Figure 18.15: Loading on Beam B-B

The u.d.l will be only due to soil pressure on under-side of the beam itself

$$= 148.9 \times 0.5 = 74.45 \text{ kN/m}$$

conc. load due to secondry beam = 670.05 × 2 = 1340.1 kN

$\therefore \quad \Sigma f_y = -2 \times 2000 - 2 \times 2500 + 74.45 \times 14 + 1340.1 \times 7 = 1423.0$ kN (upward load)

Hence for the *whole* raft

Total upward load = 1423.0 − 1095.18 = 327.82 kN

If conc. loads, due to secondary beam, on main beam are taken according to reaction coefficients then,

$$2 \times \left(-4 \times 2000 + \frac{0.375}{0.5} \times 670.05 \times 7 + 197.29 \times 14\right) = -3440.35 \text{ kN}$$

$$-2 \times 2000 - 2 \times 2500 + 1.25 \times 1340.1 \times 7 + 74.45 \times 14 = 3768.17 \text{ kN}$$

$$\therefore \ \Sigma f_y = -3440.35 + 3768.17 = 327.82 \text{ kN}$$

From above it is clear that almost the same results for both the methods.

B.M at critical section

At the face of support next to the end support

$$+197.29 \times \frac{\left(1+4-\frac{0.35}{2}\right)^2}{2} - (2000-670.05) \times \left(4-\frac{0.35}{2}\right) + 670.05 \times \left(2-\frac{0.35}{2}\right) = -1567.7 \text{ kNm}$$

$$d = \sqrt{\frac{M}{R_b b}} = \sqrt{\frac{1567.7 \times 10^6}{0.897 \times 500}} = 1869.6$$

Provided $D = 1920$ and $d = 1870$

$$A_{st} = \frac{M}{\sigma_{st} j_B d} = \frac{1567.7 \times 10^6}{230 \times 0.906 \times 1870} = 4023.15 \text{ mm}^2$$

Hence provided 9#25 (A_{st} = 4410 mm^2)

B.M. will be the maximum in the span where S.F. = 0

or $+2000 - 670.05 \times 2 - 197.29x = 0$

or $x = 3.34$ m

$$\therefore M_{max} = -(2000 - 670.05) \times (3.34 - 1) + 670.05 \times (3.34 - 3) + 197.29 \times \frac{3.34^2}{2}$$
$$= -1783.82 \text{ kNm}$$

At this section the beam will act as T-beam and, therefore, assuming $j_B \approx 0.9$

$$A_{st} = \frac{M}{\sigma_{st} j_B d} = \frac{1783.82 \times 10^6}{230 \times 0.9 \times 1870} = 4608.28 \text{ mm}^2$$

Hence provided 10#25 (A_{st} = 4900mm^2)

S.F. at Critical Sections

At distance d from inner face of exterior column

$$= -197.29 \times (0.825 + 0.35 + 1.87) + 2000 - 670.05 = 729.2 \text{ kN}$$

At d L.H. of support next to end support

$$= +2000 - 2 \times 670.05 - 197.29 \times (1 + 4 - \frac{3.34}{2} - 1.87) = -371.85 \text{ kN}$$

$\therefore$ Nominal shear stress at d from inner face of exterior column

$$\tau_v = \frac{V}{bd} = \frac{729.2 \times 10^3}{500 \times 1870} = 0.779 \frac{N}{\text{mm}^2}$$

$$p\% = \frac{A_{st}}{bd} \times 100 = \frac{4900}{500 \times 1870} \times 100 = 0.52$$

$$\tau_c = 0.3 + \frac{(0.35 - 0.3)}{(0.75 - 0.5)} \times (0.52 - 0.5) = 0.304 \frac{N}{\text{mm}^2}$$

$\therefore \quad V_c = \tau_c bd = 0.304 \times 500 \times 1870 \times 10^{-3} = 284.24$ kN

$\therefore \quad V_s = V - V_c = (729.2 - 284.24) = 444.96$ kN

Taking #10-2 legged stirrups

$$s_v = \frac{\sigma_{sv} A_{sv} d}{V_s} = \frac{230 \times 156 \times 1870}{444.96 \times 10^3} = 150.79$$

s_v shall not be greater than

i) $0.75d = 0.75 \times 1870 = 1402.5$

ii) 450

iii) $\dfrac{A_{sv} \times 0.87 f_y}{0.4b} = \dfrac{100 \times 0.87 \times 415}{0.4 \times 500} = 180.52$

Hence provided #8-two legged stirrups @150 c/c throughout

The details of reinforcement have been shown in Figure 18.16.

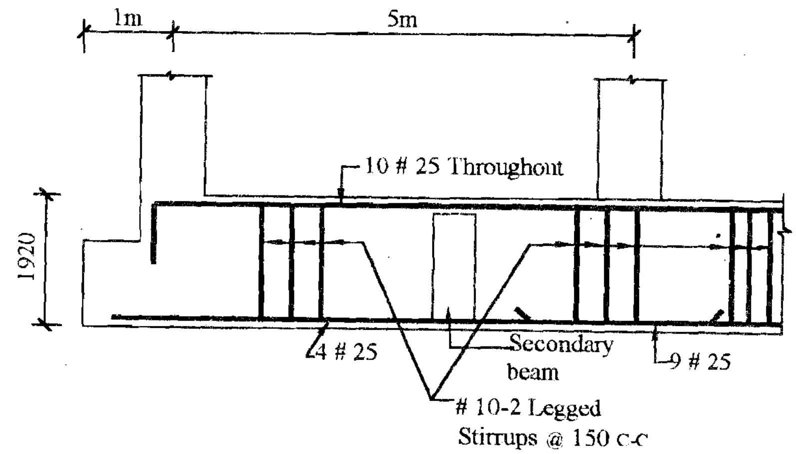


Figure 18.16: Details of Main Beam

Example 18.2

Design a *mat foundation* for the data of Example 18.1 (Figure 18.17).

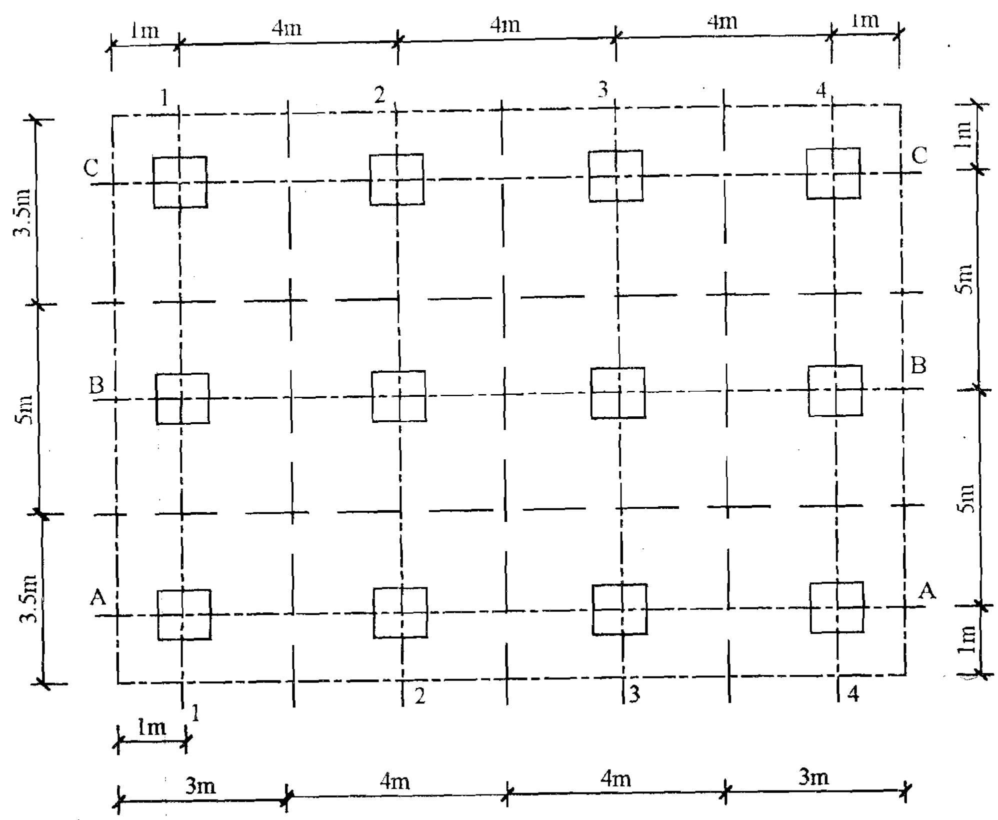


Figure 18.17: Longitudinal and Cross Beams

Solution

According to BIS 2950 (Part - I) - 1981, when the variations in adjacent column loads and their spacing do not exceed 20% of the higher value, the raft may be divided into perpendicular strips of widths equal to the distance between mid spans and each strip may be analysed as an independent beam with *known* column loads and *known* contact pressures.

Based on above mentioned principles, the slab has been divided into 3 longitudinal beams and 4 cross beams as follows;

Two longitudinal beams A-A and C-C each of width 3.5 and length 14 m

One longitudinal beam B-B of width 5 m and length 14 m

Two cross beams 1-1 and 4-4 each of width 3 m and length 12 m

Two cross beams 2-2 and 3-3 each of width 4m and length 12 m

Design of beam B-B

Loads (Figure 18.18)

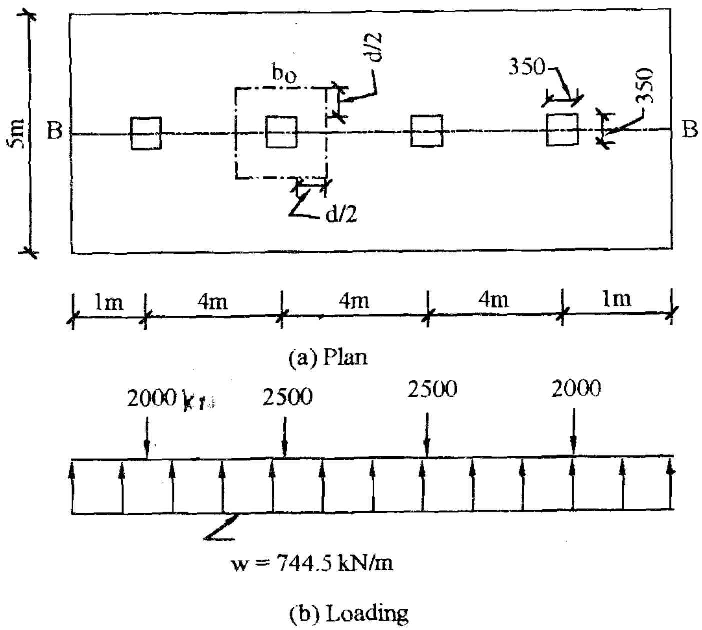


Figure 18.18: Beam B - B

udl, $w = 148.9 \times 5 = 744.5$ kN/m

B.M. at L.H. face of exterior column, $M = 744.5 \times \dfrac{(1-0.175)^2}{2} = \mathbf{253.36\ kNm}$

Maximum B.M. at span

For this let the shear force be zero at x from the edge of the mat

S.F. $= -744.5x + 2000$

or $x = 2.69$ m

$$\therefore\ M_{max} = 744.5 \times \frac{2.69^2}{2} - 2000\,(2.69 - 1) = \mathbf{-686.36\ kNm}$$

B.M. at L.H., face of column next to end column

$$M = 744.5 \times \frac{\left(1+4-\dfrac{0.35}{2}\right)^2}{2} - 2000 \times \left(4 - \frac{0.35}{2}\right) = \mathbf{+1016.21\ kNm}$$

B.M. at mid span of middle span, $M = 744.5 \times \dfrac{7^2}{2} - 2000 \times 6 - 2500 \times 2 = \mathbf{1240.25\ kNm}$

For deciding depth, the maximum B.M. for beam 2-2 is also calculated

Loads (Figure 18.19)

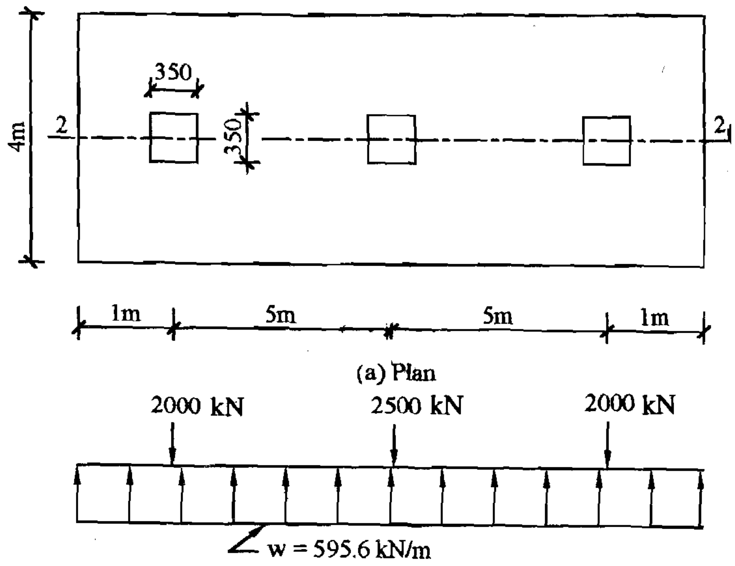


Figure 18.19: Beam 2 - 2

udl, $w = 148.9 \times 4 = 595.6$ kN/m

Maximum B.M. in the span

Let S.F. = 0 at x from edge of the mat

$V = 0 = -595.6\,x + 2000$

or $x = 3.36$ m

$$\therefore \quad M_{max} = 595.6 \times \frac{3.36^2}{2} - 2000 \times (3.36 - 1) = \mathbf{-1357.95\ kNm}$$

B.M. at L.H. face of column next to exterior column

$$595.6 \times \frac{\left(1+5-\frac{0.35}{2}\right)^2}{2} - 2000 \times \left(5 - \frac{0.35}{2}\right) = \mathbf{454.54\ kNm}$$

$$\therefore \quad d = \sqrt{\frac{M}{R_b b}} = \sqrt{\frac{454.54 \times 10^6}{0.897 \times 5000}} = \mathbf{318.35}$$

Depth of Foundation from Two-way shear considerations

$b_0 = 4\,(b + d) = 4\,(350 + d)$

$k_s = (0.5 + \frac{350}{350}) = 1.5 > 1$

$\tau_c = 0.16\sqrt{f_{ck}} = 0.16\sqrt{20} = 0.716$

$\therefore$ Permissible shear stress $= k_s \tau_c = 1 \times 0.716 = 0.716$ N/mm^2

$$\tau_v = \frac{V}{b_0 d} = \frac{2500 \times 10^3}{4(350+d)d} = k_s \tau_c = 0.716$$

or $2500 \times 10^3 - 1002.4d - 2.864d^2 = 0$

or $d^2 + 350d - 872905 = 0$

or $$d = \frac{\left(-350 \pm \sqrt{\left(350^2 + 4 \times 872905\right)}\right)}{2} = 775.54$$

Hence provided $D = 900$ and $d = 875$

Area of steel (A_{st})

$$A_{st} = \frac{M}{\sigma_{st} j_B d} = \frac{1240.25 \times 10^6}{230 \times 0.906 \times 875} = 6257.96 \text{ mm}^2$$

$$\text{Spacing of \#16, } s = \frac{201 \times 5000}{6257.96} = 160.59$$

spacing shall be the least of the following for the *beam*

i) 160.59

ii) $3d = 3 \times 180 = 540$

iii) 450

iv) $A_{s,\min} = \frac{0.85}{f_y} \times bd = \frac{0.85 \times 5000 \times 875}{415} = 8960.84 \text{ mm}^2 > 6257.96 \text{ mm}^2$

$$s = \frac{201 \times 5000}{8960.84} = 112.15$$

Hence provided #16 @110

A_{st} for span

$$A_{st} = \frac{M}{\sigma_{st} j_B d} = \frac{686.36 \times 10^6}{230 \times 0.906 \times 875} = 3764.33 \text{ mm}^2$$

$$\text{Spacing of \#16, } s = \frac{201 \times 5000}{3764.33} = 266.98$$

Hence provided #16 @110 throughout both at supports and in span in both directions.

Check for One-way Shear

At distance d from the inner face of exterior column,

$$V = -744.5 \times \left(1 + \frac{0.35}{2} + 0.875\right) + 2000 = 473.78 \text{ kN}$$

At distance d from L.H. face of the column next to end column

$$V = -744.5 \times \left(1 + 4 - \frac{0.35}{2} - 0.875\right) + 2000 = -940.78 \text{ kN}$$

$$\therefore \quad \tau_{vmax} = \frac{V_{max}}{bd} = \frac{940.78 \times 10^3}{5000 \times 875} = 0.21 \frac{N}{\text{mm}^2} < 0.22 \frac{N}{\text{mm}^2}$$

The details of reinforcements have been shown in Figure 18.20.

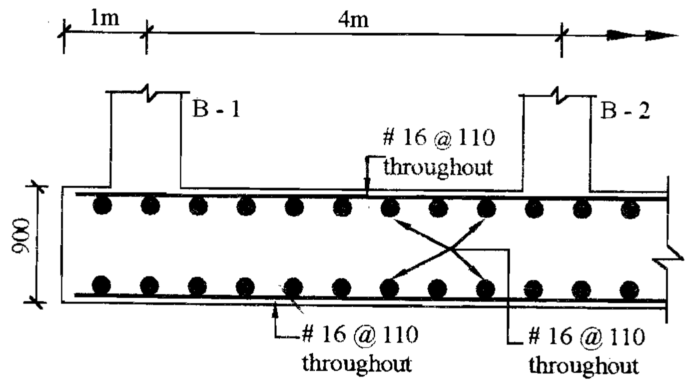


Figure 18.20: Details of Beam B - B

SAQ 1

i) Define a raft foundation.

ii) What are the types of raft footings ? Explain with sketches.

SAQ 2

i) Which are the methods of analysis for design of a raft foundation? Why flexible method of analysis has not been discussed in this unit?

ii) Design and detail the central span of the main beam B-B of Example 18.1.

iii) Design and detail the beam 2-2 of the mat foundation of Example 18.2.

18.3 SUMMARY

A raft type foundation is proposed when isolated footings for individual columns overlap each other for a structure. Selection of the type of raft is dependent on type of soil, type of loading, site plan, environmental factors etc. Manual design is possible only by assuming the raft to be *rigid* one and, therefore, the contact pressure variation is assumed as planar (Eqn 18.1). The raft is analysed as a whole in each of the two perpendicular directions which is based on principle of statics for *known* column loads and *known* contact pressure.

18.4 ANSWERS TO SAQs

SAQ 1

i) Refer text 18.1

ii) Refer text 18.1

SAQ 2

i) Refer text 18.2

ii) Refer Example 18.1

iii) Refer Example 18.2

* The reinforcements have been provided at closer interval to match the curtailment

FURTHER READINGS

1) BIS : 456-1978, "*Code of Practice for Plain and Reinforced Concrete*", Bureau of Indian Standards, Manak Bhawan, 9 Bahadur Shah Zafar Marg, New Delhi – 110002

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3) Mallick S. K. & Gupta A. P. , "*Reinforced Concrete*", Oxford & IBH Publishing Co. Pvt. Ltd.

4) S. N. Sinha, "*Reinforced Concrete Design*," Tata MC Graw-Hill Publishing Company Limited.

5) SP 24 :1983 "*Explantary Handbook on IS :456-1978*", Bureau of Indian Standards, Manak Bhawan, 9 Bahadur Shah Zafar Marg, New Delhi - 110002

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7) Punmia B.C., Jain Ashok K., & Jain Arun K., "*Reinforced Concrete Structures - Volume I*", Laxmi Publications, 7/21, Ansari Road, Daryaganj, New Delhi-110002.

8) IS : 2950 (Part - I) - 1981, "*Code of Practice for the Design and Construction of Raft Foundation*". Bureau of Indian Standards, Manak Bhawan, 9 Bahadur Shah Zafar Marg, New Delhi - 110002

9) Jai Krishna and Jain O.P., "Plain and Reinforced Concrete, Volume I", Nemchand & Bros", Roorkee.

10) Teng W.C., "*Foundation Design, Prentice Hall of India (Pvt.) Ltd*".

11) Varyani U.H., "*Structural Design of Multistoreyed Buildings*", South Asian Publishers, New Delhi.