UNIT 17 DESIGN OF COMBINED FOOTINGS

Structure

17.1 INTRODUCTION

A footing provided under two or more collinear columns is called combined footing. These footings are necessary under the following circumstances:

i) where single collinear footings under adjacent columns overlap due to restricted width of footing or due to low bearing capacity of soil, and

ii) to avoid non-uniform soil pressure* beneath isolated or single footing.

The design of a combined footing is more efficient and economical as well as the settlement of footing is uniform if the pressure distribution due to load is *uniform*. This condition may be achieved if the centroid of all applied loads and the centroid of the area of footing coincide. Generally, these footings may be of the following types :

i) a rectangular slab type *with* or *wihout* a beam connecting the columns,

ii) a trapezoidal slab type *with* or *without* a beam connecting the columns, and

iii) isolated footings connected by a beam (strap footing).

Objectives

After reading this unit, you will be able to

- recognise the conditions under which a combined r.c. footing is provided as a foundation; their various types,
- analyse the loads and stresses to which a combined footing is subjected, and
- design and detail such a footing.

17.2 DESIGN OF RECTANGULAR COMBINED FOOTING WITH OR WITHOUT BEAM

The design of a rectangular combined footing may be done in the following steps :

a) Determine column loads and self-weight of the footing.

b) Determine area of footing for the above load and known bearing capacity of soil. The width of footing is fixed. Keep in mind that the length should always be more than distance between the external faces of extreme columns. The projections of footing beyond the columns in the longitudinal direction may be fixed in such a way that the C.G. of column loads must coincide with the C.G. of area of footing to have uniform distribution of soil pressure.

* The soil pressure may be non-uniform due to restriction on required dimension of footing caused by property line or otherwise.

c) Draw S.F.D. and B.M.D. and mention their critical values and respective locations for design purposes.

If the rectangular combined footing is to be without beam.

i) Determine the depth of slab for maximum bending moment as well as for *one-way* and *two-way* shears and fix up the designed depth accordingly.

ii) Treating the slab as a *wide* and *inverted* beam spanning longitudinally, between the columns design and detail the main reinforcements. The shear reinforcement for one-way shear may also be designed, if necessary.

iii) In the near vicinity of columns, the slab bends in the form of a saucer, i.e., it bends in the transverse direction *as well*. Hence the load below a column is to be distributed across the full width and a *limited length equal to the dimension of column along the length of footing plus twice the effective depth of footing on either side of the column*. The reactive pressure on the above area is evaluated for the design of cantilever projections of the slab in the transverse directions in the same way as for isolated footing in the remaining portion of the length. Only distribution bars are provided in the transverse direction.

If the rectangular combined footing is to be provided with beam

i) Design and detail main as well as shear reinforcements for the beam, and

ii) The projected cantilever slab in the transverse direction may be designed in the usual way.

Example 17.1

Two columns having cross-section of 250 × 250 mm and 300 × 300 mm are loaded with 300 kN and 500 kN respectively. The c/c distance between the column is 4 m and the bearing capacity of soil is 100 kN/m^2. Design a *rectangular* combined footing *without beam*.

Solution

Loads

Super-imposed load = 300 + 500 = 800 kN

Self weight of footing (assuming 10% of superimposed load) = 80 kN

Total load = 880 kN

Size of Footing

$$\text{Required area of footing} = \frac{880}{100} = 8.8 \text{ m}^2$$

Hence provided area of footing = 6 m × 1.5 m = 9 m^2 > 8.8 m^2

Let the C.G. of loads be at x from the centre of column C_1 (Figure 17.1(a)). Taking moment of superimposed loads about centre of column C_1,

$$-(300 + 500)\,x + 500 \times 4 = 0$$

or $$x = 2.5 \text{ m}$$

For uniform soil pressure C.G. of loads must coincide with C.G. of footing. i.e. projection of footing on L.H.S. from centre of column C_1

$$x_1 = \frac{L}{2} - 2.5 = 3 - 2.5 = 0.5 \text{ m}$$

Similarly, projection of footing on R.H.S. from centre of column C_2,

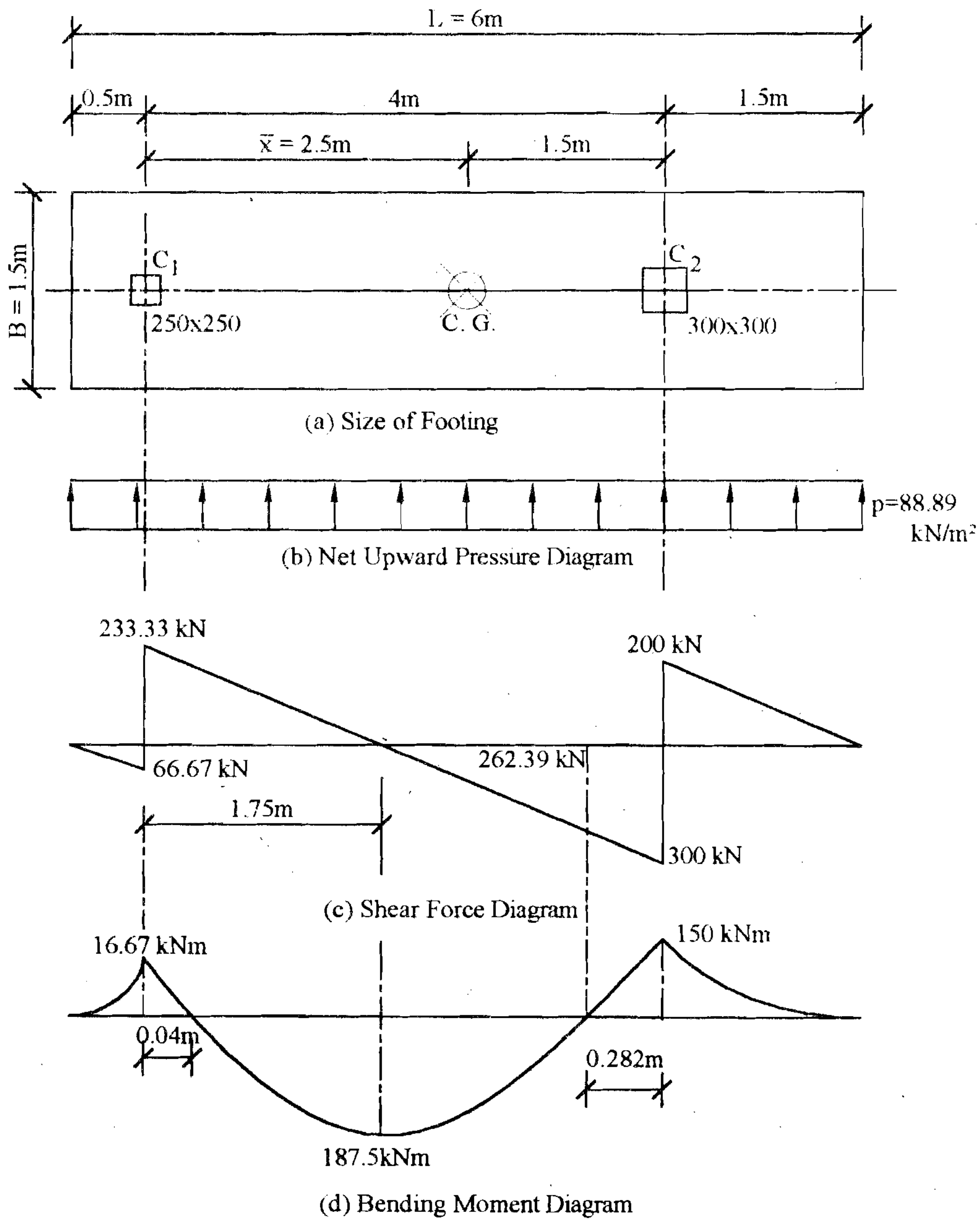


Figure 17.1: Size of Footing, S.F.D. & B.M.D.

$$x_2 = \frac{L}{2} - (4 - 2.5) = 3 - 1.5 = 1.5 \text{ m}$$

Net upward pressure on footing

$$\frac{300+500}{6 \times 1.5} = 88.89 \text{ kN/m}^2 \text{ (Figure 17.1 (b))}$$

The S.F.D. and B.M.D. have been drawn in Figure 17.1 (c&d).

Depth of footing fro Xm Bending Moment consideration

Let the distance of point from centre of C_1 where S.F. is zero be x, then

$$x = \frac{233.33}{1.5 \times 88.89} = 1.75 \text{ m}$$

$$\therefore \quad M_{max} = -300 \times 1.75 + 88.89 \times 1.5 \times \frac{(0.5+1,72)^2}{2} = -187.5 \text{ kNm}$$

$$\therefore \quad d = \sqrt{\frac{M}{R_b b}} = \sqrt{\frac{187.5 \times 16^6}{0.874 \times 1500}} = 378.18 \text{ mm}$$

For $\phi 16$ as main reinforcement

$$D = 378.18 + \frac{16}{2} + 40 = 426.18\text{mm}$$

Hence provided D = 430 mm

Assuming $\phi 16$ as main reinforcement

$$d = 430 - \frac{16}{2} - 40 = 382$$

*Checking D for Two-way shear**

Taking critical section for two-way shear at $\frac{d}{2}$ from the face of column C_2 (Figure 17.2)

$$V = 500 - 88.89\,(0.3 + 0.382)^2 = 458.655 \text{ kN}$$

$$b_0 = 4 \times (300 + 2 \times 191) = 2728$$

$$\therefore \quad \tau_v = \frac{V}{b_0 d} = \frac{458.655 \times 1000}{2728 \times 382} = 0.44 \text{ N/mm}^2$$

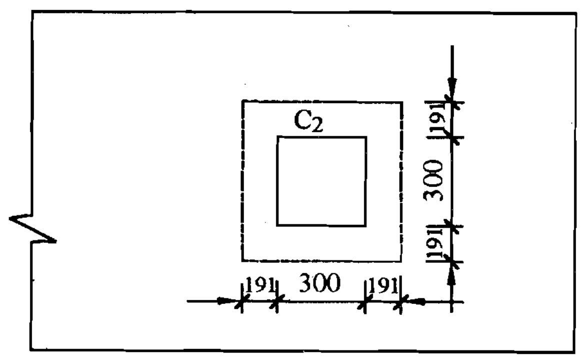


Figure 17.2: Critical Section for Two-way Shear

Permissible shear stress = $k_s\, \tau_c$,

where, $k_s = 0.5 + \frac{\text{Short side of column}}{\text{Long side of column}} = 0.5 + 1 = 1.5 > 1$

Hence k_s = 1

$$\tau_c = 0.16\sqrt{f_{ck}} = 0.16\sqrt{15} = 0.62 \text{ N/mm}^2$$

$\therefore$ Permissible shear stress, $k_s \tau_c = 1 \times 0.62 = 0.62 \text{ N/mm}^2 > \tau_v$ Hence O.K.

* In most of the cases bending moment or *two-way* shear determines the value of D for a footing. If the depth, so evaluated, is found inadequate, in *one-way shear* reinforcement shall be provided.

Longitudinal Tensile Reinforcement in Span

For maximum span hogging bending moment M = 187.50 kNm

$$A_{st} = \frac{M}{\sigma_{st} j_B d} = \frac{187.50 \times 10^6}{140 \times 0.865 \times 382} = 4053.16 \text{ mm}^2$$

$$A_{st,\,min} = \frac{0.85bd}{f_y} = \frac{0.85 \times 1500 \times 382}{250} = 1948.2 \text{ mm}^2 < A_{st}$$

Hence provided 21ϕ16 as longitudinal tensile reinforcement

Check for Development Length at point of Inflection

Let n be the number of ϕ16 bars required at point inflection at 0.282 m from centre line of column C_2

$$L_d = \frac{M_1}{V} + L_0$$

where, $M_1 = f_d A_{st} j_B d$

$= 140 \times n \times 201 \times 0.865 \times 382$

$= 9.298n$ kNm

V = 262.39 kNm

L_d greater of 12 × (dia of bar) = 12 × 16 = 192 mm or d = 382 . i.e. L_0 = 382 mm

$$L_d = \frac{\phi \sigma_s}{4\tau_{bd}} = \frac{16 \times 140}{4 \times 0.6} = 933.33 \text{ mm}$$

Substituting above values, the equation for L_d,

$$933.33 \le \frac{9.298n \times 10^6}{262.39 \times 1000} + 382$$

or $n > 15.56$

Hence all the 21 number ϕ16 may be extended beyond point of inflection for a distance of effective depth (382 mm) and, thereafter, only alternative bars may be extented upto the edge of footing as nominal reinforcement.

Longitudinal Tensile Reinforcement at column C_1

B.M. at L.H. face of column C_1

$$= p \times B \frac{(0.5-0.125)^2}{2} = 88.89 \times 1.5 \times \frac{0.375^2}{2} = 9.38 \text{ kNm}$$

B.M. at R.H. face of column C_1

$$= p \times B \frac{(0.5+0.125)^2}{2} - 300 \times 0.125$$

$$= 88.89 \times 1.5 \times \frac{0.625^2}{2} - 300 \times 0.125$$

$= -11.46$ kNm (hogging B.M.)

∴ Longitudinal tensile reinforcement for maximum sagging moment = 9.38 kNm will only be designed.

$$A_{st} = \frac{X}{\sigma_{st} j_B d} = \frac{9.38 \times 10^6}{140 \times 0.865 \times 382} = 202.77 \text{ mm}^2 < A_{stmin} (= 1948\text{mm}^2)$$

Hence provided 10ϕ16 (A_{st} = 2010mm^2)

Longitudinal Tensile Reinforcement at column C_2

B.M. at R.H. face of column C_2

$$= p \times B \frac{(1.5 - 0.15)^2}{2} = 88.89 \times 1.5 \times \frac{1.35^2}{2} = 121.5 \text{ kNm}$$

B.M. at L.H. face of column C_2

$$= p \times B \frac{(1.5 + 0.15)^2}{2} - 500 \times 0.15$$

$$= 88.89 \times 1.5 \times \frac{1.65^2}{2} - 500 \times 0.15$$

$$= 106.502 \text{ kNm}$$

∴ Longitudinal tensile reinforcement for maximum sagging moment

$$A_{st} = \frac{M}{\sigma_{st} j_B d} = \frac{121.5 \times 10^6}{140 \times 0.865 \times 382} = 2626.45\text{mm}^2 > A_{stmin} (=1948.2\text{mm}^2)$$

Hence provided 14 number ϕ16 (A_{st} = 2814 mm^2)

Check for one-way shear

In cantilever projection

Critical section for S.F. at distance d beyond the left face of column C_1 falls beyond edge of footing, hence no check is necessary.

The shear force at critical section d on R.H.S. of column C_2,

$$V = 88.89 \times 1.5 \times (1.5 - 0.15 - 0.382) = 129.068 \text{ kN}$$

$$\tau_v = \frac{129.068 \times 1000}{1500 \times 382} = 0.225$$

$$p\% = \frac{14 \times 201}{1500 \times 382} \times 100 = 0.49\%$$

$$\tau_c = 0.22 + \frac{(0.29 - 0.22)}{0.25} \times (0.49 - 0.25) = 0.287 \text{ N/mm}^2$$

Permissible shear stress = $k\tau_c$ = 1 × 0.287 N/mm^2 > τ_v

Hence no shear reinforcement is necessary.

In Central portion

Point of contraflexure is *nearer* to column face C_1, hence shear stress at this point

$$\tau_v = \frac{227.997 \times 10^3}{1500 \times 382} = 0.398 \text{ N/mm}^2$$

$$p\% = \frac{10 \times 201}{1500 \times 382} \times 100 = 0.351\,\%$$

$$\therefore \quad \tau_c = 0.22 + \frac{(0.29 - 0.22)}{0.25} \times (0.351 - 0.25) = 0.248 \text{ N/mm}^2$$

Permissible shear stress = $k\tau_c$ = 1 × 0.248 = 0.248 N/mm^2 < τ_v

Hence shear reinforcement will be provided for a shear force of

$V_s = V - V_c$ = 227.997 – 0.248 × 1500 × 382 × 10^{-3} = 85.893 kN

If ϕ8-8 legged stirrups are provided

$$s_v = \frac{\sigma_{sv} A_{sv} d}{V_s} = \frac{140 \times 8 \times 50 \times 382}{85.893 \times 10^3} = 249.05 \text{ mm c/c}$$

Spacing for minimum shear reinforcement is given by

$$s_v = \frac{0.87 f_y A_{sv}}{0.4b} = \frac{0.87 \times 250 \times 8 \times 50}{0.4 \times 1500} = 145 \text{ c/c}$$

The spacing shall also not exceed 0.75 × 382 = 286.5 c/c

Hence provided ϕ8-8legged stirrups @145 c/c

Point of contraflexure is *nearer* to column face C_2, hence shear stress at this point

$$\tau_v = \frac{262.39 \times 10^3}{1500 \times 382} = 0.458 \text{ N/mm}^2$$

$$p\% = \frac{14 \times 201}{1500 \times 382} \times 100 = 0.49\%$$

$$\therefore \quad \tau_c = 0.22 + \frac{(0.29 - 0.22)}{0.25} \times (0.49 - 0.25) = 0.287 \text{ N/mm}^2$$

Permissible shear stress = $k\tau_c$ = 1 × 0.287 = 0.287 N/mm^2 < τ_v

Hence shear reinforcement will be provided for a shear force of

$V_s = V - V_c$ = 262.39 – 0.287 × 1500 × 382 × 10^{-3} = 97.939 kN

If ϕ8-8 legged stirrups are provided

$$s_v = \frac{\sigma_{sv} A_{sv} d}{V_s} = \frac{140 \times 8 \times 50 \times 382}{97.939 \times 10^3} = 218.42 \text{ mm c/c}$$

Hence provided ϕ8-8legged stirrups @145 c/c

(as this is according to minimum shear reinforcement)

Transverse Reinforcement

i) **Under Column C_1**

Slab projected beyond the face of column $C_1 = \dfrac{(1.5 - 0.25)}{2}$ = 0.625m width over which column load is supposed to be distributed,

$$b' = 0.25 + 2 \times 0.382 = 1.014\text{m}$$

$\therefore$ Net upward pressure $= \dfrac{300}{1.014 \times 1.5} = 197.239$ kN/m^2

Considering *1m wide strip*

$$M_{max} \text{ at face of column} = 197.239 \times \frac{0.625^2}{2} = 38.52 \text{ kNm}$$

$$d = 430 - 40 - 16 - \frac{12}{2} = 368$$

$$\therefore \quad A_{st} = \frac{M}{\sigma_{st} j_B d} = \frac{38.52 \times 10^6}{140 \times 0.865 \times 368} = 864.36 \text{ mm}^2$$

$$A_{st,min} = \frac{0.15}{100} \times 1500 \times 430 = 967.5 \text{ mm}^2 > A_{st}$$

Hence $A_{st} = 967.5$ mm^2/m

spacing $= \dfrac{1000 \times 113}{967.5} = 116.80$ mm c/c

Hence provided ϕ12 @115 c/c in a width of 1.014 m.

ii) **Under column C_2**

Slab projected beyond the face of column $C_2 = \dfrac{(1.5 - 0.3)}{2} = 0.6$ m

$\therefore$ $b' = 0.3 + 2 \times 0.382 = 1.064$ m

Net upward pressure $= \dfrac{500}{1.064 \times 1.5} = 313.28$ kN/m^2

Considering 1m wide strip

$$M_{max} = \frac{313.28 \times 0.6^2}{2} = 56.39 \text{ kNm}$$

$$d = 430 - 40 - 16 - \frac{12}{2} = 368 \text{ mm}$$

$$A_{st} = \frac{56.39 \times 10^6}{140 \times 0.865 \times 368} = 1265.35 \text{ mm}^2$$

$$\text{spacing} = \frac{1000 \times 113}{1265.35} = 89.30$$

Hence provided ϕ12 @85 for a width of 1.064 m under column C_2

The detailings of reinforcement have been shown in Figure 17.3.

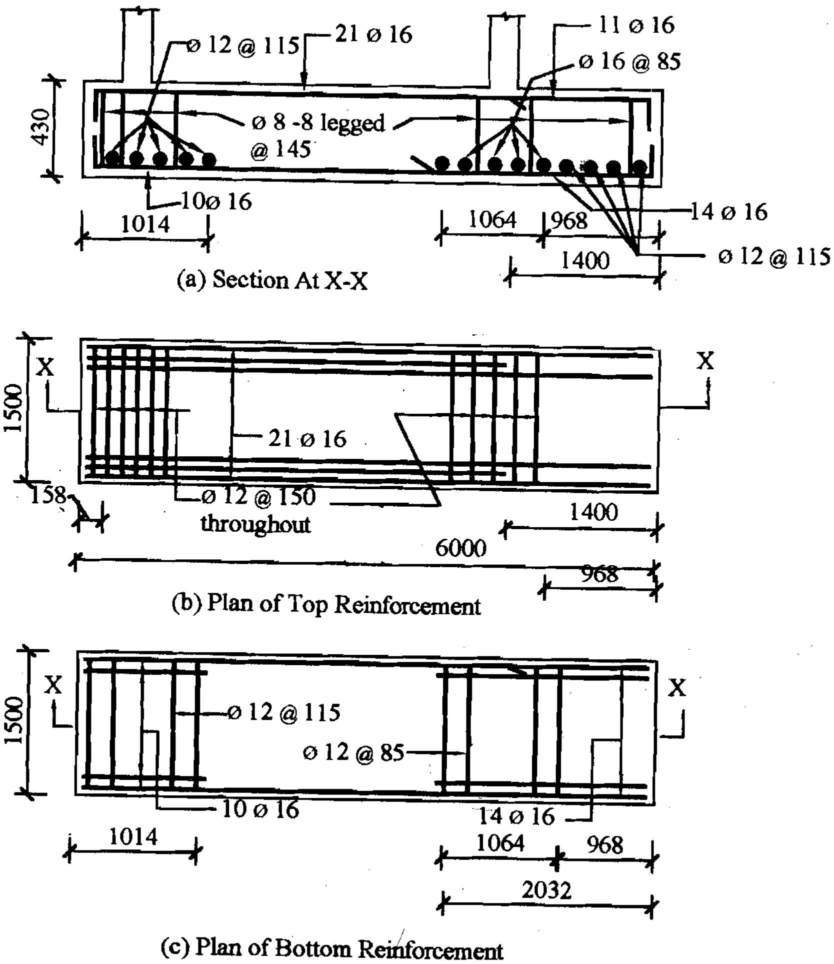


Figure 17.3: Detailing of Footing

Example 17.2

Two columns having cross-section of 500 × 500 mm and 600 × 600 mm are transmitting loads of 1250 kN and 1750 kN respectively. The c/c distance between the columns is 5 m and the bearing capacity of soil is 300 kN/m^2. Design a combined rectangular footing with beam joining the columns.

Solution

Loads

Super-imposed load	= 1250 + 1750	= 3000 kN
Self-weight of footing (assuming 10% of superimposed load)		= 300 kN
Total load		= 3300 kN

Size of Footing

$$\text{Required area of footing} = \frac{3300}{300} = 11 \text{ m}^2$$

Hence provided size of footing = 6.5 m × 1.7 m = 11.05 m^2 > 11 m^2

Let the C.G. of loads be at $\bar{x}$ from the centre of column C_1 (Figure 17.4(a)). Taking moment of superimposed loads about centre of column C_1,

$$-(1250 + 1750)\ \bar{x} + 1750 \times 5 = 0$$

or $\bar{x} = 2.916$ m

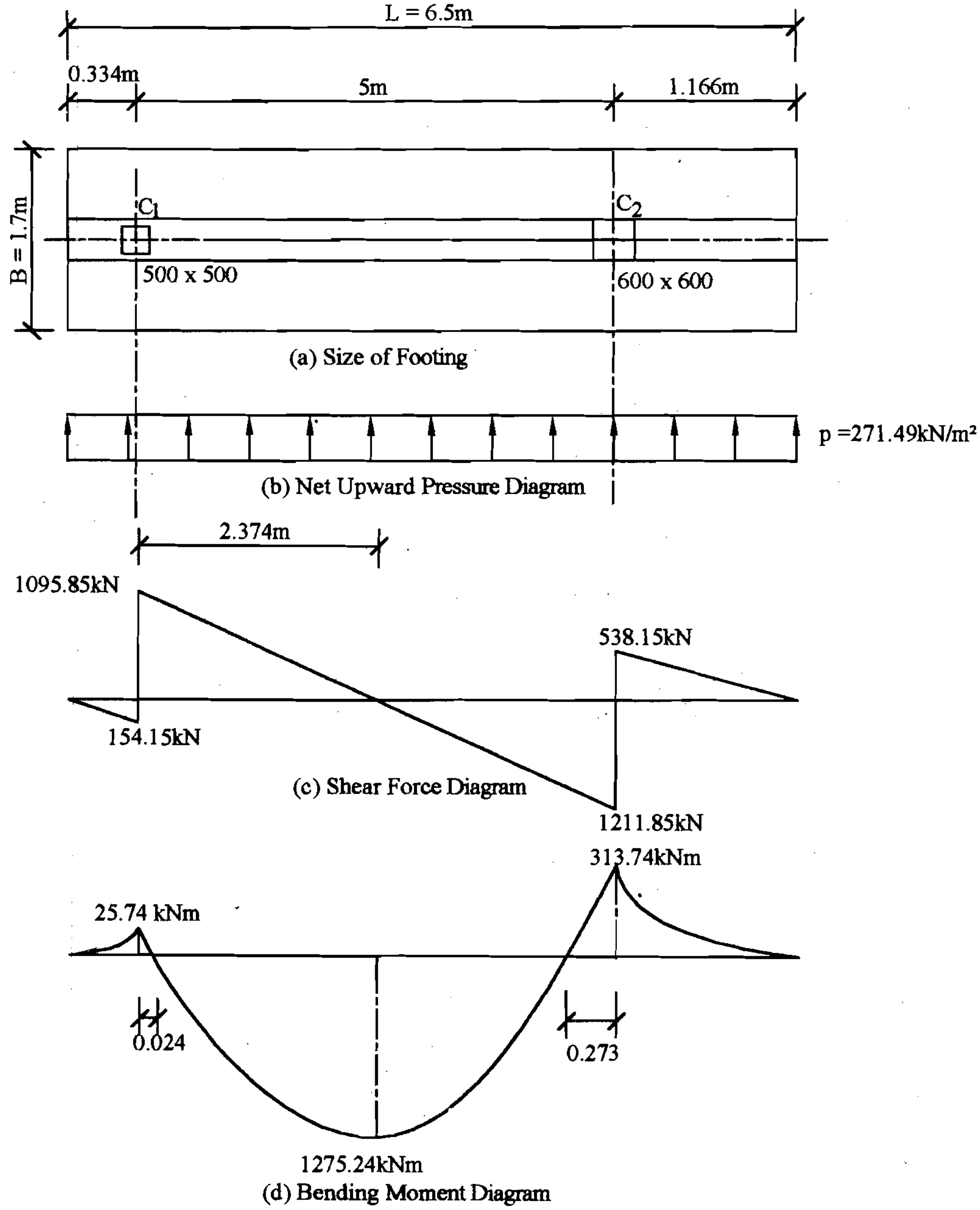


Figure 17.4: Size of Footing, S.F.D. & B.M.D.

For uniform soil pressure, C.G. of loads must coincide with C.G. of footing;

Projection of footing on L.H.S. of centre of column C_1

$$x_1 = \frac{L}{2} - 2.916 = 3.25 - 2.916 = 0.334 \text{ m}$$

Similarly, projection of footing on R.H.S. of centre of column C_2,

$$x_2 = \frac{L}{2} - (5 - 2.916) = 3.25 - 2.084 = 1.166 \text{ m}$$

Net upward pressure on footing

$$\frac{1250 + 1750}{6.5 \times 1.7} = 271.49 \text{ kN/m}^2 \text{ (Figure 17.4(b))}$$

The S.F.D. and B.M.D. have been drawn in Figure 17.4 (c & d).

Design of Slab

Let the width of the beam = 600 mm

$\therefore$ Projection of slab beyond the longitudinal face of beam = $\frac{1.7-0.6}{2} = 0.55$ m

$$\therefore\ M_{max} = 271.49 \times \frac{0.55^2}{2} = 41.06 \text{ kNm/m width}$$

$$\therefore\ d = \sqrt{\frac{M}{R_b b}} = \sqrt{\frac{41.06\times10^6}{0.874\times1000}} = 216.75 \text{ mm}$$

$$\therefore\ D = 216.75 + 40 + \frac{12}{2} = 262.75 \text{ mm}$$

Provided D = 350 mm

$$d = 350 - 40 - 16\frac{12}{2} - = 288 \text{ mm}$$

$$A_{st} = \frac{M}{\sigma_{st} j_B d} = \frac{41.06\times10^6}{140\times0.865\times288} = 1177.29 \text{ mm}^2$$

$\therefore$ Provided ϕ12@95 c/c (=1189.47 mm²/m)

Check for shear

S.F. at critical section (i.e. at d from face of beam) = 271.49 (0.55 – 0.288)

= 71.13 kN/m width

$$\tau_v = \frac{71.13\times10^3}{1000\times288} = 0.247 \text{ N/mm}^2$$

$$p\% = \frac{1189.47}{1000\times288}\times100 = 0.41\%$$

$$\tau_c = 0.22 + \frac{(0.29-0.22)}{0.25}(0.41 - 0.25) = 0.265 \text{ N/mm}^2$$

Permissible shear stress = $k\tau_c$ = 1 × 0.265 = 0.265 N/mm² > τ_v

Hence no shear reinforcement is necessary.

Check for developmet length

$$L_d = \frac{\phi\sigma_s}{4\tau_{bd}} = \frac{12\times140}{4\times0.6} = 700 \text{ mm}$$

Straight length available beyond the face of the beam = 550 – 40 = 510 < L_d

With standard U-hook total lenth = 510 + 16 × 12 = 702 > 700 Hence O.K.

Design of Beam

The beam will act as a T-beam in the span between points of contraflexture and as rectangular beam in the projected portion

Design of beam between C_1 & C_2

l_0 for effective width calculation of *isolated* T-beam = 5 – (0.024 + 0.273) = 4.703 m

$$b_f = \frac{l_0}{\frac{l_0}{b}+4} + b_w = \frac{4.703}{\frac{4.703}{1.7}+4} + 0.6 = 1.295 \text{ m} < 1.7 \text{ m}$$

For balanced section

assuming $j_B = 0.9$

$$M = 0.45\, D_f\, d_f\, \sigma_{cbc} \left(\frac{2k_B d - D_f}{k_B}\right)$$

or $$1275.24 \times 10^6 = 0.45 \times 1295 \times 350 \times 5 \left(\frac{2 \times 0.404 \times d - 350}{0.404}\right)$$

or $$d = 1058.4 \text{ mm}$$

Hence provided D = 1150 mm

$\therefore$ $$d = 1150 - 40 - 32 - \frac{32}{2} = 1062 \text{ (Assuming } \phi 32 \text{ reinforcement in two layers)}$$

Areas of steel (A_{st})

i) **For critical B.M. in the span**

$$M = \sigma_{st} \times A_{st} \times l_a$$

$$l_a \approx 0.9d = 0.9 \times 1062 = 955.8 \text{ mm}$$

$$\therefore\ A_{st} = \frac{M}{\sigma_{st} l_a} = \frac{1275.24 \times 10^6}{130 \times 955.8} = 10263.17 \text{ mm}^2$$

Hence provided 13ϕ32 in two layers (A_{st} = 10452 mm^2)

Check for Development Length

$$L_d \le \frac{M_1}{V} + L_0$$

where $$M_1 = \sigma_{st} A_{st} j_B\, d$$

$$= 140 \times n \times 804 \times 0.9 \times 1062$$

$$= n \times 107.58 \times 10^6 \text{ Nmm}$$

$$V = 1085.86 \text{ kN}$$

L_0 = greater of 12ϕ (=12x32=384) or d = 1062

Substituting all values in the above equation

$$L_d = 58.3 \times 32 \le \left(\frac{n \times 107.58 \times 10^6}{1085.86 \times 10^3} + 1062\right)$$

or $$n \ge 8.11$$

Hence all 13ϕ32 have been extended upto the edge of footing on R.H.S. and on L.H.S. all the bars may be bent at 90° to make up L_0 at the edge.

ii) **For B.M. at the face of column on R.H.S. projection**

Load per m run = 271.49 × 1.7 = 461.53 kN/m

$$\therefore \quad M = 461.53 \times \frac{(1.166-0.3)^2}{2} = 173.064 \text{ kNm}$$

$$d = 1150 - 40 - \frac{32}{2} = 1094$$

$$\therefore \quad A_{st} = \frac{173.064 \times 10^6}{130 \times 0.865 \times 1094} = 1406.792 \text{ mm}^2$$

$$A_{st,min} = \frac{0.85bd}{f_y} = \frac{0.85 \times 600 \times 1094}{250} = 2231.76 \text{ mm}^2$$

Hence provided 8ϕ20 (A_{st} = 2513.27 mm^2)

These bars may be bent at 90° at the edge to make up for development length

iii) **For B.M. at the face of column on L.H.S. projection**

$$M = 461.53 \times \frac{(0.334-0.25)^2}{2} = 1.628 \text{ kNm}$$

$A_{st,min}$ will only be sufficient.

Hence provided 8ϕ20 and the bars may be bent at 90° at the edge to make up for development length

Provision of Shear Reinforcement

S.F. at *d* from interior face of R.H. column

$$V = \frac{1211.85}{(5-2.374)} \times (5-2.374-1.062) = 721.757 \text{ kN}$$

$$\tau_v = \frac{V}{bd} = \frac{721.757 \times 10^3}{600 \times 1062} = 1.133 < 1.6 \text{ N/mm}^2 (\tau_{max})$$

$$p\% = \frac{A_{st}}{bd} \times 100 = \frac{13 \times 804}{600 \times 1062} \times 100 = 1.64\%$$

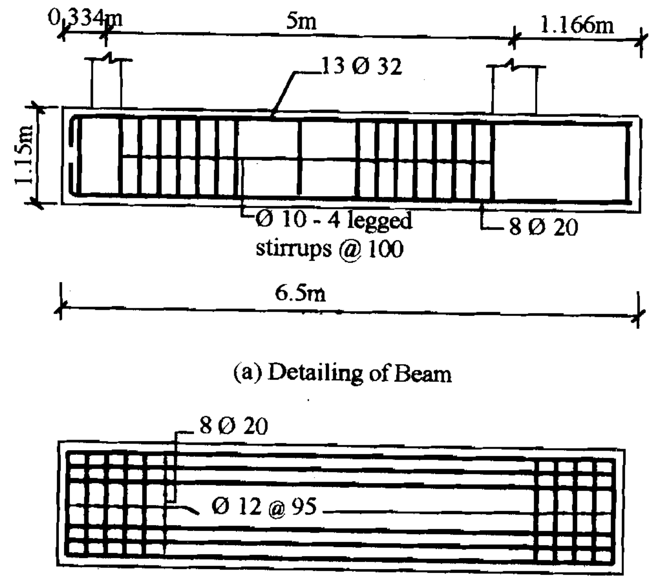
$$\tau_c = 0.42 + \frac{(0.44-0.42)}{0.25} \times (1.64 - 1.5) = 0.43 \text{ N/mm}^2$$

$$V_s = 721.757 - 0.431 \times 600 \times 1062 \times 10^{-3} = 447.124 \text{ kN}$$

$$s_v = \frac{\sigma_{sv} A_{sv} d}{V_s} = \frac{140 \times 4 \times 78 \times 1062}{447.124 \times 10^3} = 103.75 \text{ mm}$$

Hence provided ϕ10-4 legged stirrups @ 100 c/c

The details of reinforcements have been shown in Figure 17.5.



(a) Detailing of Beam

(b) Detailing of Slab

Figure 17.5: Details of the Designed Footing with Beam

SAQ 1

i) Define a combined footing. Under which conditions it is essential ?

ii) What are the types of combined footings ?

iii) Enumerate the steps for design of combined rectangular footing.

iv) Design and detail a combined rectangular footing *without* beam to transmit 800 kN and 1200 kN through column at sizes 400 × 400 mm and 500 × 500 mm respectively. The distance between the columns is 3.5 m and the bearing capacity of soil is 250 kN/m^2.

v) Design and detail the rectangular footing as given in (ii) *with* beam.

17.3 DESIGN OF TRAPEZOIDAL COMBINED FOOTING WITH OR WITHOUT BEAM

A trapezoidal footing becomes a necessity when the dimension along the length of footing is limited due to property line or due to some other reasons.

Example 17.3

Two columns having cross-sections of 240 × 240 mm and 300 × 300 mm are loaded with 300 kN and 500 kN respectively. The c/c distance between the column is 4m. The bearing capacity of soil is 100 kN/m^2. The footing is restricted to 120 mm from centre of first column and 150 mm from that of second column. Design a *trapezoidal* combined footing *without* beam.

Solution

Loads

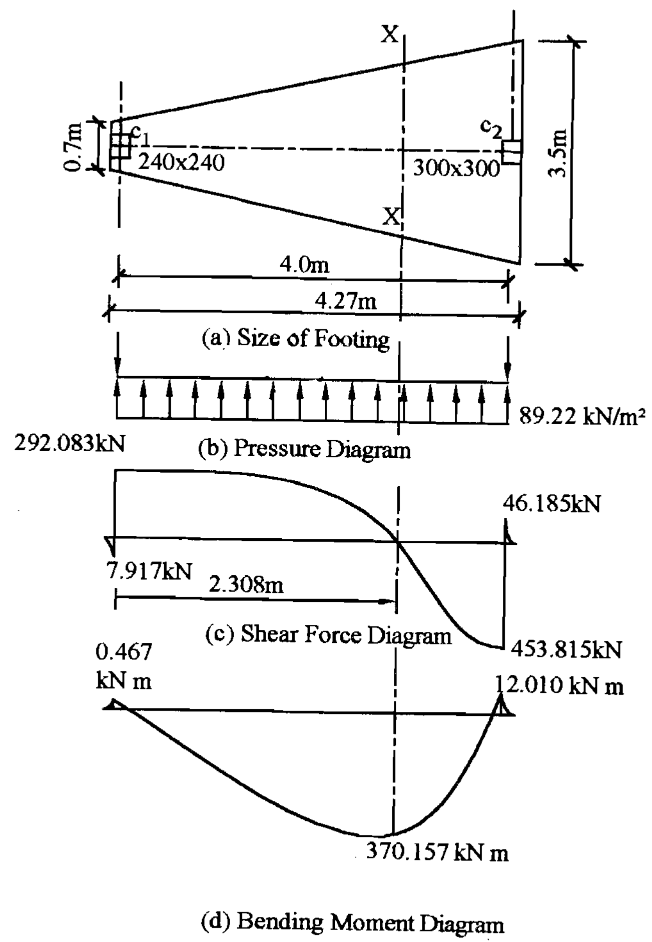
Super-imposed load	= 300 + 500	= 800 kN
Self-weight of footing (assuming 10% of superimposed)		= 80 kN
Total load		= 880 kN

Size of Footing

$$\text{Required area of footing} = \frac{880}{100} = 8.8 \text{ m}^2$$

$$\frac{(B_1 + B_2)}{2} \times (4 + 0.12 + 0.15) = 8.8$$

or $B_1 + B_2 = 4.12$ m ...(17.1)



|1 .6: Size of Footing, S.F.D. & B.M.D.

Let the C.G. of loads be at $\bar{x}$ from the centre of column C_1 (Figure 17.6(a)). Taking moment of superimposed loads about centre of column C_1,

$$-(300 + 500)\ \bar{x} + 500 \times 4 = 0$$

or $$\bar{x} = \frac{500 \times 4}{(300 + 500)} = 2.5 \text{ m}$$

For uniform soil pressure C.G. of loads must coincide with C.G. of footing, i.e., C.G. of footing from side B_1 is given by

$$\frac{(B_1 + 2B_2)}{B_1 + B_2} \times \frac{4.27}{3} = (2.5 + 0.12)$$

or $\quad B_1 + 2B_2 = 1.84\,(B_1 + B_2)$

or $\quad -0.84\,B_1 + 0.16\,B_2 = 0 \qquad$ ...(17.2)

Solving simultaneous equations 17.1 and 17.2

$B_1 = 0.7$ m, and $B_2 = 3.5$ m (Figure 17.6(a))

∴ Net upward pressure on footing

$$\frac{800}{\frac{(0.7+3.5)}{2} \times 4.27} = 89.22\frac{\text{kN}}{\text{m}^2} \quad \text{(Figure 17.6(b))}$$

Calculation for S.F.D. and B.M.D.

$$p = 89.22 \text{ kN/m}^2$$

Let he breadth be B_x at distance x from L.H.S. edge

$$B_x = \left(B_1 + \frac{B_2 - B_1}{L}x\right)$$

S.F. at centroid of C_1 (L.H.S.) Breadth B_x at the centroid of column C_1

$$= 0.7 + \frac{3.5 - 0.7}{4.27} \times 0.12$$

$$= 0.779 \text{ m}$$

∴ S.F. at centroid of C_1

$$= -89.22 \times \frac{0.7 + 0.779}{2} \times 0.12$$

$$\mathbf{= -7.917\ kN}$$

S.F. at R.H.S. of centroid of C_1

$= 300 - 7.917 =$ **292.083 kN**

S.F. at R.H.S. of centroid of C_2: Breadth at centroid of C_2

$$= \left\{0.7 + \frac{2.8}{4.27}(4.27 - 0.15)\right\}$$

$$= 3.402 \text{ m}$$

∴ S.F. at R.H. of centroid of C_2

$$= 89.22 \times \left(\frac{3.402 + 3.5}{2}\right) \times 0.15$$

$$\mathbf{= 46.185\ kN}$$

∴ S.F. at L.H. of cen troid of C_2

$$= -500 + 46.185 = -453.815 \text{ kN}$$

Let S.F. = 0 at x, then

$$B_x = \left(0.7 + \frac{2.8}{4.27}x\right)$$

$$89.22 \times \left[\frac{0.7 + \left(0.7 + \frac{2.8}{4.27}x\right)}{2}\right]x - 300 = 0$$

or $$89.22 \times \left[\frac{1.4 + 0.656x}{2}\right]x - 300 = 0$$

or $62.454x + 29.264x^2 - 300 = 0$

or $x^2 + 2.134x - 10.252 = 0$

or $$x = \frac{-2.134 \pm \sqrt{4.554 + 41.008}}{2} = 2.308 \text{ m}$$

B.M. at centroid of C_1

$$+7.917 \times \frac{(2 \times 0.7 + 0.779)}{(0.7 + 0.779)} \times \frac{0.12}{3}$$

= 0.467 kNm

B.M. at centroid of C_2

$$+46.185 \times \frac{(3.402 + 2 \times 3.5)}{2} \times \frac{0.15}{3}$$

= 12.010 kNm

Max. B.M. at x = 2.308

$B_x = 0.7 + 0.656 \times 2.308$

$= 2.214$ m

$$\therefore \quad M_{max} = 300 \times \frac{(2 \times 0.7 + 2.214)}{(0.7 + 2.214)} \times \frac{2.308}{3} - 300 \times (2.308 - 0.12)$$

$= 286.243 - 656.4 = -370.157$ kNm

The S.F.D. and B.M.D. have been drawn in Figure 17.6 (c&d).

Depth of Footing from Bending Moment Consideration

$$d = \sqrt{\frac{M}{R_b b}} = \sqrt{\frac{370.157 \times 10^6}{0.865 \times 2214}} = 439.639$$

Depth of Footing from Two-way Shear Consideration

i) Under column C_1 (Figure 17.7)

$$b_0 = (b_1 + 2 \times \frac{d}{2}) + 2 \times (b_1 + \frac{d}{2})$$

$= (0.240 + d) + (0.48 + d)$

$= (0.72 + 2d)$

Shear force on critical section

$$= P_1 - p\,(b_1 + d)\,(b_1 + \frac{d}{2})$$

$$= 300 - 89.22\,(0.24 + d)\,(0.24 + \frac{d}{2})$$

$$= 300 - 89.22 \times (0.0576 + 0.36d + \frac{d^2}{2})$$

$$= 294.861 - 32.119d - 44.61d^2$$

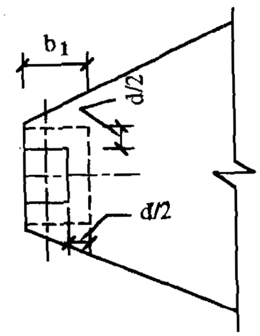


Figure 17 : Two-way Shear Under Column C_1

$$\therefore\ \tau = \frac{\{294.861 - 32.119d - 44.61d^2\}}{b_0 d}$$

$$= \frac{\{294.861 - 32.119d - 44.61d^2\} \times 10^3}{(0.72 + 2d)d \times 10^6}$$

$k_c = (0.5 + 1) = 1.5 > 1$

Hence $k_c = 1$

$\therefore\ \tau_c = k_c\, 0.16\sqrt{15} = 1 \times 0.619 = 0.619$ N/mm^2

Equating τ and τ_c

$$\frac{\{294.861 - 32.119d - 44.61d^2\}}{(0.72 + 2d)d \times 10^3} = 0.619$$

or $294.861 - 32.119d - 44.61d^2 = 445.68d + 1238.0d^2$

or $1282.61d^2 + 477.8d - 294.861 = 0$

or $d^2 + 0.372d - 0.229 = 0$

or $d = \frac{-0.372 \pm \sqrt{0.138 + 0.916}}{2} = 0.3206$ m $= 320.6$ mm

ii) Under column C_2 (Figure 17.8)

$$b_0 = (b_2 + 2 \times \frac{d}{2}) + 2 \times (b_2 + \frac{d}{2})$$

$= (0.3 + d) + (0.6 + d)$

$= (0.9 + 2d)$

Shear force on critical section

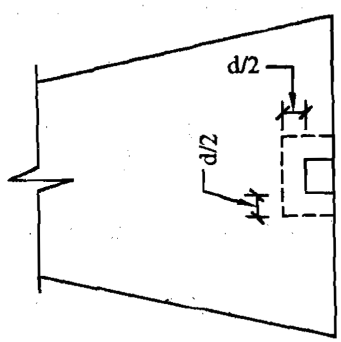


Figure 17.8: Two-Way Shear Under Clumn C_2

$$= P_2 - p\,(b_2 + d)\,(b_2 + \frac{d}{2})$$

$$= 500 - 89.22 \times (0.3 + d)\,(0.3 + \frac{d}{2})$$

$$= 500 - 89.22 \times (0.09 + 0.45d + \frac{d^2}{2})$$

$$= 491.97 - 40.149d - 44.61d^2$$

$$\therefore\ \tau = \frac{\{491.97 - 40.149d - 44.61d^2\}}{b_0 d}$$

$$= \frac{\{491.97 - 40.149d - 44.61d^2\} \times 10^3}{(0.9 + 2d)d \times 10^6}$$

$$k_c = (0.5 + 1) = 1.5 > 1$$

Hence $k_c = 1$

$$\therefore\ \tau_c = k_c \times 0.16\sqrt{f_{ck}} = 0.16 \times \sqrt{15} = 0.619 \text{ N/mm}^2$$

Equating τ and τ_c

$$\frac{\{491.97 - 40.149d - 44.61d^2\}}{(0.9 + 2d)d \times 10^3} = 0.619$$

or $\quad 557.1d + 1238d^2 = 419.97 - 40.149d - 44.61d^2$

or $\quad 1282.61d^2 + 597.249d - 419.97 = 0$

or $\quad d^2 + 0.466d - 0.327 = 0$

or $\quad d = \dfrac{-0.466 \pm \sqrt{0.217 + 1.308}}{2} = 0.384 \text{ m} = 384$

Therefore maximum value of d from above considerations = 439.639

$$D = 439.639 + 40 + \frac{20}{2} = 489.639$$

Hence provided $D = 520$

$$d = 520 - 40 - \frac{20}{2} = 470$$

$$A_{st} = \frac{M}{\sigma_{st} j_B d} = \frac{370.157 \times 10^6}{140 \times 0.865 \times 470} = 6503.45 \text{ mm}^2$$

Hence provided 21ϕ20

Check for development length near column C_1

Let 14ϕ20 are only extended upto the column C_1

$\therefore \quad M_1 = \sigma_{st} A_{st} j_B d = 140 \times 14 \times 314 \times 0.865 \times 470 = 250.21 \times 10^6$ Nmm

$\quad V = 292.083$ kN

Providing 90° bend,

$$L_0 = 120 - 40 + 8\phi = 120 - 40 + 8 \times 20 = 240$$

$$L_d \le \frac{1.3M_1}{V} + L_0$$

or $$58.3\phi \le \frac{1.3 \times 250.21 \times 10^6}{292.083 \times 10^3} + 240$$

or $\phi \le 23.22$ Hence O.K.

Check for development length under column C_2

Let all 21ϕ20 extend upto the R.H. edge of footing

$\therefore$ $M_1 = \sigma_{st} A_{st} j_B d = 140 \times 21 \times 314 \times 0.865 \times 470 = 375.31$ kNm

$V = 453.815$ kN

Providing 90° bend at the edge,

$$L_0 = 150 - 40 + 8 \times 20 = 270$$

$$L_d = 58.3\,\phi$$

$$L_d \le \frac{1.3M_1}{V} + L_0$$

or $$58.3\phi \le \frac{1.3 \times 375.31 \times 10^6}{453.815 \times 10^3} + 270$$

or $\phi \le 23.07$ Hence O.K.

Check for one-way shear

i) *Near column C_1*

The shear force at critical section d from inner face of column C_1

Width at this section is given by

$$B_x = B_1 + \frac{B_2 - B_1}{L}x = 0.7 + 0.656 \times (0.24 + 0.47) = 1.165 \text{ m}$$

$$V = p\frac{B_1 + B_x}{2}(b_1 + d) + 300 = -89.22 \times \frac{(0.7 + 1.165)}{2}(0.24 + 0.47) + 300$$

$$= 240.93 \text{ kN}$$

$$\tau_v = \frac{V}{bd} = \frac{240.93 \times 10^3}{1165 \times 470} = 0.44 \text{ N/mm}^2$$

$$p\% = \frac{A_{st}}{bd} \times 100 = \frac{14 \times 314}{470 \times 1165} \times 100 = 0.803\%$$

$$\tau_c = 0.34 + \frac{(0.37 - 0.34)}{0.25} \times (0.803 - 0.75) = 0.346 \text{ N/mm}^2$$

$$k = 1$$

$\therefore$ Permissible shear stress

$$= k\tau_c = 1 \times 0.346 = 0.346 \text{ N/mm}^2 < \tau_v$$

Assuming ϕ8-8legged stirrups

$A_{sv} = 50.26 \times 8 = 402.08$ mm^2

$V_c = \tau_c bd = 0.346 \times 1165 \times 470 = 189.45 \times 10^3$ N

$\therefore\ V_s = V - V_c = 240.93 \times 10^3 - 189.45 \times 10^3 = 51.48 \times 10^3$ N

$$s_v = \frac{\sigma_{sv} A_{sv} d}{V_s} = \frac{250 \times 402.08 \times 470}{51.48 \times 10^3} = 917.72 \text{ mm}$$

$$s_{v,min} = \frac{0.87 \times f_y A_{sv}}{0.4b} = \frac{0.87 \times 250 \times 402.08}{0.4 \times 1165} = 187.67 \text{ mm}$$

Spacing is minimum of

i) 917.72

ii) 187.67

iii) $0.75d = 0.75 \times 470 = 352.5$

iv) 450

Hence provided ϕ8-8 legged stirrups @185c/c

ii) **Near Column C_2**

The S.F. at critical section d from inner face of column C_2

$V = 274.13$

Width of footing at this section, $B_x = B_1 + \dfrac{B_2 - B_1}{L} x$

$= 0.7 + 0.656 \times (4.27 - 0.3 - 0.47) = 2.996$m

$$V = p\frac{B_x + B_2}{2}(b_2 + d) - 500$$

$$= 89.22 \times \frac{(2.996 + 3.5)}{2}(0.3 + 0.47) - 500$$

$= 276.864$ kN

$$\tau_v = \frac{V}{bd} = \frac{276.864 \times 10^3}{2996 \times 470} = 0.197 \text{ N/mm}^2$$

$$p\% = \frac{A_{st}}{bd} \times 100 = \frac{21 \times 314}{2996 \times 470} \times 100 = 0.468\%$$

$$\therefore\ \tau_c = 0.22 + \frac{(0.29 - 0.22)}{0.25}(0.468 - 0.25) = 0.281$$

$\therefore$ Permissible shear stress $k\tau_c = 1 \times 0.281 = 0.281$ N/mm$^2 > \tau_v$

Hence no shear reinforcement is required at this end.

Transverse Reinforcement

i) *Under Column C_1*

Width of footing at the centre of column $C_1 = 0.7 + 0.656 \times 0.12 = 0.78$ m

Projection of slab at the centre of column = $\frac{(0.78-0.24)}{2}$ = 0.27 m

Width of bending strip = 0.24 + 0.47 = 0.71 m

Width of footing at 0.71 m

= 0.7 + 0.656 × 0.71 = 1.159 m

Area under column load = $\frac{(0.7+1.17)}{2} \times 0.71 = 0.664\ \text{m}^2$

∴ Upward pressure = $\frac{300}{0.664}$ = 451.81 kN/m²

Maximum B.M. at the face of column

$$M = \frac{451.81 \times 0.27^2}{2} = 16.468\ \text{kNm}$$

$$d = 520 - 40 - 20 - \frac{20}{2} = 450$$

$$\therefore \quad A_{st} = \frac{16.468 \times 10^6}{140 \times 0.865 \times 450} = 302.193\ \text{mm}^2$$

Nominal reinforcement = $\frac{0.15}{100} bD = \frac{0.15 \times 780 \times 520}{100} = 608.4\ \text{mm}^2$

Spacing for ϕ12 bars = $\frac{710 \times 113}{608.4}$ = 131.87

Hence provided ϕ12@130 c/c

i) *Under column C_2*

Width of footing at the centre of column C_2 = 0.7 + 0.656 × 4.12 = 3.4 m

Projection of slab = $\frac{(3.4 - 0.3)}{2}$ = 1.55m

Width of bending strip = 0.3 + 0.47 = 0.77 m

Width of footing at 0.77 m from right edge

= 0.7 + 0.656 × 3.5 = 2.996 m

Area of loaded strip = $\frac{(2.996+3.5)}{2} \times 0.77 = 2.5\ \text{m}^2$

Net upward pressure = $\frac{500}{2.5}$ = 200 kN/m²

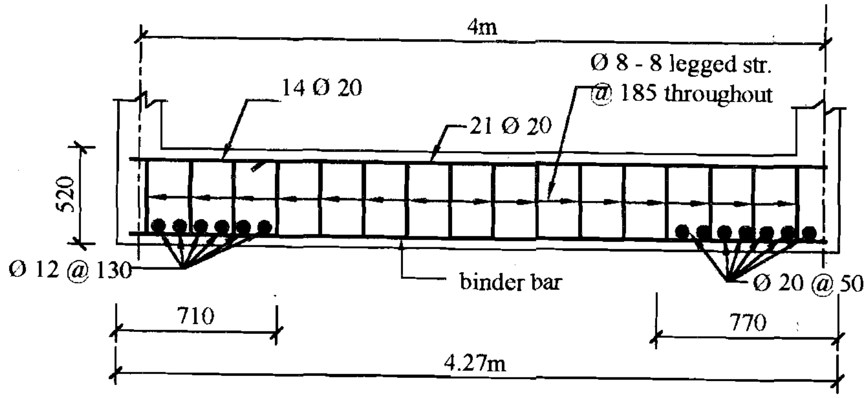
B.M. at the face of column

$$= 200 \times \frac{1.55^2}{2} = 240.25\ \text{kNm}$$

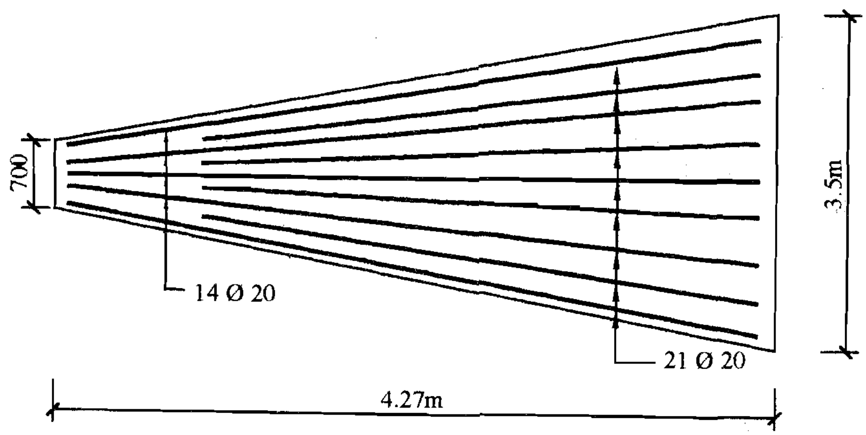
$$A_{st} = \frac{240.25 \times 10^6}{140 \times 0.865 \times 450} = 4408.66\ \text{mm}^2$$

Hence provided ϕ20@50 c/c

The detailings of reinforcement have been shown in Figure 17.9.



(a) Detailing of the Trapezoidal Footing



(b) Plan showing Longitudinal Reiforcement

Figure 17.9: Detailing of the Designed Footing

SAQ 2

Design and detail a trapezoidal combined footing *with* beam for the data given in Example 17.3.

17.4 DESIGN OF STRAP FOOTING

A strap footing is a combination of two or more isolated footings joined by a beam called a *strap* beam. The size of isolated footings and their locations with respect to axes of respective columns are such that the C.G. of applied column loads coincides with the C.G. of all isolated footings taken together.

Example 17.4

Two columns having cross-sections of 240 × 240 mm and 300 × 300 mm are loaded with 300 kN and 500 kN, respectively. The c/c distance between the columns is 4 m. The bearing capacity of soil is 100 kN/m^2. Design a combined footing *strap* beam.

Solution

Loads

Super imposed load	= 300 + 500	= 800 kN
Self-weight of footing (assuming 10% of superimposed)		= 80 kN
Total load		= 880 kN

Size of Footing

Required area of footing = $\frac{880}{100} = 8.8 \text{ m}^2$

Keeping width of footings under both columns, $B = 2$ m

Sum of Lengths of footings, $(L_1 + L_2) = \frac{8.8}{2} = 4.4$ m

C.G. of loads from column C_2

$$\bar{x}_2 = \frac{300 \times 4}{(300 + 500)} = 1.5 \text{ m}$$

The C.G. of the two footings (Figure 17.10) should coincide with the C.G. of loads for uniform pressure; therefore, equating moments of superimposed and reactive forces about centre of C_2

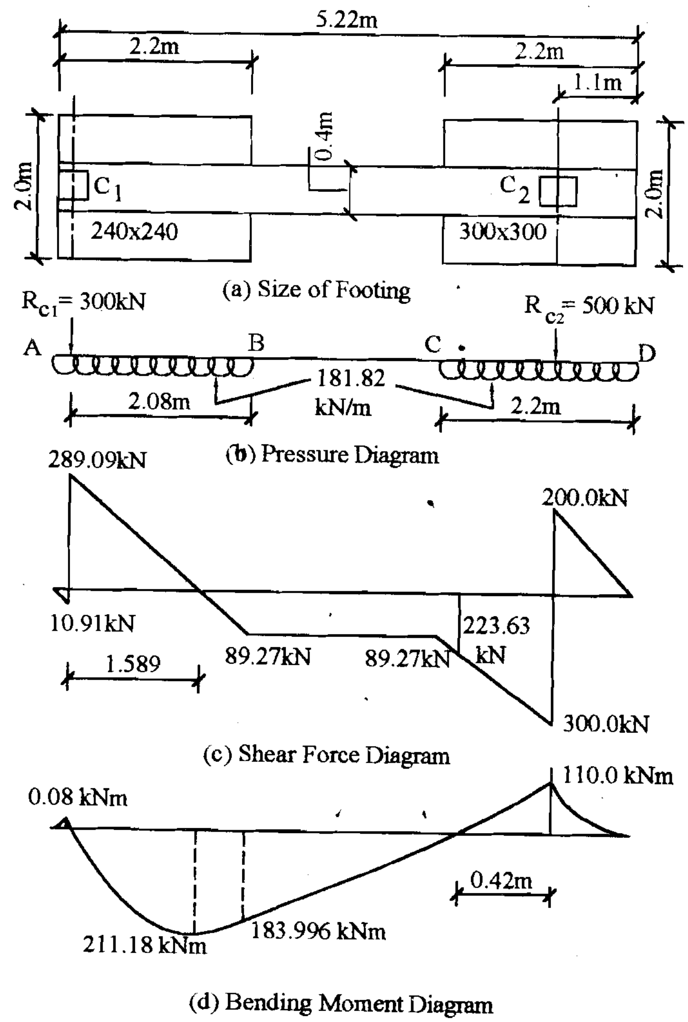


Figure 17.10: S. D. & B. M. D. of the Beam

$$\bar{x}_2 B(L_1 + L_2) = BL_1 \left(4 + 0.12 - \frac{L_1}{2}\right)$$

or $\quad 1.5 \times 2 \times 4.4 = 2L_1 \left(4.12 - \frac{L_1}{2}\right)$

or $\quad L_1^2 - 8.24L_1 + 13.2 = 0$

or $\quad L_1 = 2.177 \approx 2.2\text{m}$

$\therefore \quad L_2 = 4.4 - 2.2 = 2.2\text{m}$

$\therefore$ Net upward pressure on footing $= \frac{800}{2\,(2.2 + 2.2)} = 90.91$ **kN/m²**

Design of Footing Slab

Let the width of strap beam = 400 mm

The projection of slab beyond the longitudinal face of beam $= \frac{2 - 0.4}{2} = 0.8$ **m**

$$\therefore \quad M = \frac{90.91 \times 0.8^2}{2} = 29.09 \text{ kNm/m}$$

$$\therefore \quad d = \sqrt{\frac{M}{R_b b}} = \sqrt{\frac{29.09 \times 10^6}{0.865 \times 1000}} = 183.38 \text{ mm}$$

Adopting $\phi 12$ bars

$$D = 183.38 + 40 + \frac{12}{2} = 229.38 \text{ mm}$$

Hence provided D = 230 mm

$$\therefore \quad D = 230 - 40 - \frac{12}{2} = 184$$

Check for one way-shear

The shear force at a distance d from face of the beam,

$$V = p\,(l - d) = 90.91\,(0.8 - 0.184) = 56 \text{ kN}$$

$$\therefore \quad \tau_v = \frac{V}{bd} = \frac{56 \times 10^3}{1000 \times 184} = 0.304 \text{ N/mm}^2$$

for $p_B\% = 0.72\%$

$$\tau_c = 0.29 + \frac{(0.34 - 0.29)}{0.25} \times (0.72 - 0.5) = 0.334 \text{ N/mm}^2$$

For $\quad D = 230$

$$k = 1.10 + \frac{(1.15 - 1.10)}{(250 - 225)} \times (250 - 230) = 1.14$$

$\therefore$ Permissible shear stress $= 1.14 \times 0.334 = 0.381 \text{ N/mm}^2 > \tau_v$

Hence no shear reinforcement is necessary in the slab

$$A_{st} = \frac{M}{\sigma_{st} j_B d} = \frac{29.09 \times 10^6}{140 \times 0.865 \times 126} = 1906.47 \text{ mm}^2$$

Hence provided ϕ12@85

Distribution steel

$$= \frac{0.15}{100} \times b \times D = \frac{0.15}{100} \times 1000 \times 230 = 345 \text{ mm}^2$$

Hence provided ϕ10@225

Check for development length

$$L_d = 58\ 3\phi = 58.3 \times 12 = 699.6 \text{ mm}$$

Length available = 800 – 40 = 760 > L_d Hence O.K.

Design of Strap Beam

The S.F.D. and B.M.D. for the beam have been drawn in Figure 17.8 (c&d).

The beam just right of B (Figure 17.8(b)) will act as rectangular beam

$$\therefore\ d = \sqrt{\frac{M}{R_b b}} = \sqrt{\frac{183.996 \times 10^6}{0.874 \times 400}} = 725.47 \text{ mm}$$

Let dia. of bar be 25mm

$$\therefore D = 725.47 + 40 + \frac{25}{2} = 777.97 \text{ mm}$$

Hence provided D = 850

$$\therefore d = 850 - 40 - \frac{25}{2} = 797.5$$

$\boldsymbol{A_{st}}$

i) **Main reinforce in the span**

Since the beam acts as T-beam at the point of maximum B.M.

$\therefore$ taking $j \approx 0.9$

$$A_{st} = \frac{211.18 \times 10^6}{130 \times 0.9 \times 797.5} = 2263.27 \text{ mm}^2$$

Hence provided 5ϕ25 (A_{st} = 2454.37 mm²)

Check for development length at point of contra-flexture

$$M_1 = \sigma_{st} \times n \times \frac{\pi}{4} \times 25^2 \times 0.9 \times d$$

$$= 130 \times n \times \frac{\pi}{4} \times 25^2 \times 0.9 \times 797.5$$

$$= 45.802\ n \text{ kNm}$$

$$V = 223.63 \text{ kN}$$

$$l_0 = \text{greater of } 12\phi \text{ or } d = 797.5$$

$$L_d = 58.3 \times 25 = 1457.25$$

$$L_d \le \frac{M_1}{V} + l_0$$

$$1457.25 \leq \frac{45.802n \times 10^6}{223.63 \times 10^3} + 797.5$$

or $n \geq 3.22$

Hence all 5ϕ25 are extended upto the right edge.

ii) **Main reinforcement at support C_2**

B.M. at exterior face of column C_2

$$= 90.91 \times \frac{(1.1 - 0.15)^2}{2} = 41.023 \text{ kNm}$$

B.M. at interior face of column C_2

$$= 90.91 \times \frac{(1.1 + 0.15)^2}{2} - 500 \times 0.15 = -3.976 \text{ kNm}$$

Hence design moment, $M = 41.023$ kNm

$$A_{st} = \frac{41.023 \times 10^6}{140 \times 0.865 \times 797.5} = 424.769 \text{ mm}^2$$

Hence provided 3ϕ16 (A_{st} = 603 mm^2)

Check for Development Length

At point of contra-flexture

$$M_1 = \sigma_{st} \times n \times \frac{\pi}{4} \times \phi^2 \times jd$$

$$= 140 \times n \times \frac{\pi}{4} \times 16^2 \times 0.865 \times 797.5$$

$$= 19.418n \text{ kNm}$$

$$V = 223.63 \text{ kN}$$

$$l_0 = 797.5$$

$$L_d = 932.8 \text{ mm}$$

$$L_d \leq \frac{1.3M_1}{V} + L_0$$

$$932.8 \leq \frac{1.3 \times 19.418n \times 10^6}{223.63 \times 10^3} + 797.5$$

or $n \geq 1.19 < 3$ Hence O.K.

Hence all 3ϕ16 will be extended upto a distance d beyond point of contra-flexture in the span.

Check for shear

i) **At column C_1**

S.F. at distance *d* from interior face of column

$V = 300 - 90.91 \times 2 \times (0.24 + 0.7975)$

$= 111.36$ kN

$$\tau_v = \frac{111.36 \times 10^3}{400 \times 797.5} = 0.349 \text{ N/mm}^2$$

$$p\% = \frac{5 \times 490}{400 \times 797.5} \times 100 = 0.77\%$$

$$\tau_c = 0.34 + \frac{(0.37 - 0.34)}{(1 - 0.75)} \times (0.77 - 0.75) = 0.3424 \text{ N/mm}^2 < \tau_v$$

$\therefore V_c = \tau_c bd = 0.3424 \times 400 \times 797.5 = 109.23$ kN

$V_s = V - V_c = 111.36 - 109.23 = 2.13$ kN

Using ϕ8-2 legged stirrups

$$A_{sv} = 2 \times \frac{\pi}{4} 8^2 = 100 \text{ mm}^2$$

$$s_v = \frac{\sigma_{sv} \times A_{sv} \times d}{V_s} = \frac{140 \times 100 \times 797.5}{2.13 \times 10^3} = 5241.7$$

$$\therefore \quad s_{v,min} = \frac{A_{sv} \times 0.87 f_y}{0.4b} = \frac{100 \times 0.87 \times 250}{0.4 \times 400} = 135.94$$

The spacing shall be minimum of

i) 5241.7

ii) 135.94

iii) 0.75 d = 0.75 × 797.5 = 598.125

iv) 450

Hence provided ϕ8-2 legged stirrups @135 c/c

ii) **At column C_2**

S.F. at distance *d* from interior face of column,

$V = 90.91 \times 2\,(1.1 + 0.15 + 0.7975) - 500$

$= -127.72$ kN

$$\tau_v = \frac{V}{bd} = \frac{127.72 \times 10^3}{400 \times 797.5} = 0.4 \text{ N/mm}^2$$

$$p\% = \frac{5 \times 490}{400 \times 797.5} \times 100 = 0.77\%$$

$$\tau_c = 0.22 + \frac{(0.37 - 0.34)}{0.25} \times (0.77 - 0.75) = 0.3424 \text{ N/mm}^2 < \tau_v$$

$V_s = V - V_c$

$= 127.72 - 0.3424 \times 400 \times 797.5 \times 10^{-3} = 18.494$ kN

Using ϕ8-2 legged stirrups

$$s_v = \frac{\sigma_{sv} \times A_{sv} \times d}{V_s} = \frac{140 \times 100.26 \times 797.5}{18.494 \times 10^3} = 605.279$$

Similarly, S.F. at distance d on R.H. of exterior face of column C_2

$V = 90.91 \times 2\ (1.1 - 0.15 - 797.5) = 27.72$ kN

$$\tau_v = \frac{V}{bd} = \frac{27.72 \times 10^3}{400 \times 797.5} = 0.087 \text{ N/mm}^2 \text{ (very much less)}$$

Hence only nominal reinforcement will be provided

Spacing will be minimum of

i) 605.279

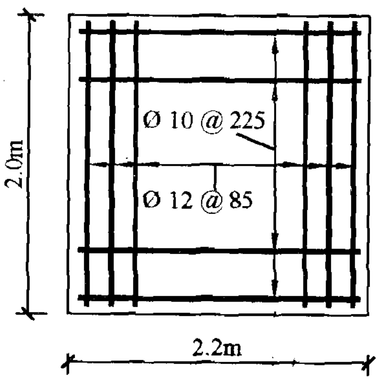
ii) 135.94

iii) 0.75 d = 598.125

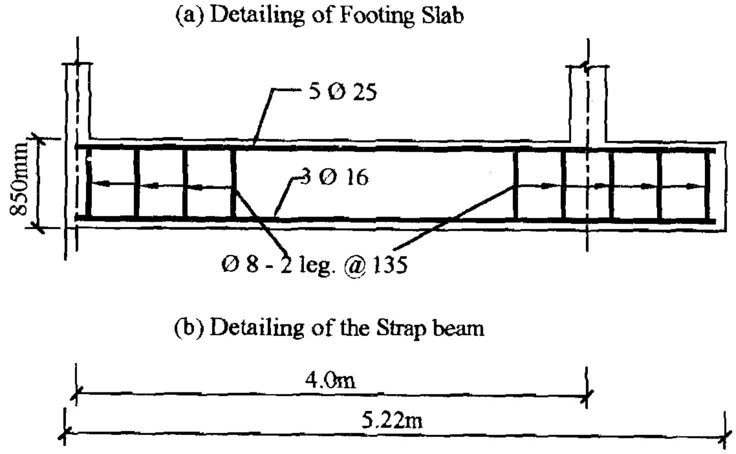
iv) 450

Hence provided ϕ8-2 legged stirrups @135 c/c throughout the beam.

The reinforcement detailings have been shown in Figure 17.11.



(a) Detailing of Footing Slab



(b) Detailing of the Strap beam

Figure 17.11: Detailing of the Designed Strap Footing

SAQ 3

Design a strap footing for the data given in SAQ 2.

17.5 SUMMARY

A combined footing for two or more columns in a line is provided if single or isolated footing for them would have overlapped *or* if uniform pressure below the footing is desired. A trapezoidal combined footing is provided where there is restriction on length of footing for obtaining uniform soil pressure. The footing slab *without* beam is designed essentially as a *wide* beam but a portion of slab under a column bends in transverse direction as well. If a beam is provided for joining the columns, it is designed in an usual way; whereas the projections of slab beyond the longitudinal faces of beam are designed as cantilevers.

17.6 ANSWERS TO SAQs

SAQ 1

i) Refer section 17.1

ii) Refer section 17.1

iii) Refer section 17.2

iv) Refer Example 17.1

v) Refer Example 17.2

SAQ 2

Hint : Determination of size of footing is the same as that for Example 17.3. The design of beam and projecting transverse slab may be done in the same way as done for them in Example 17.2

SAQ 3

Refer Example 17.4