UNIT 17  DESIGN OF COMBINED FOOTINGS

Structure

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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 without 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 x 250 mm and 300 x 300 mm are loaded with 300 kN and 500 kN respectively. The c/e distance between the column is 4 m and the bearing capacity of soil is 100 kN/m². Design a rectangular combined footing without beam.

Solution

Loads

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

Size of Footing

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

Hence provided area of footing = 6 m x 1.5 m = 9 m² > 8.8 m²

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.

\[ x = \frac{L}{2} - (4 - 2.5) = 3 - 1.5 = 1.5 \text{ m} \]

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 for Xm Bending Moment consideration

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

\[ x = \frac{233.33}{1.5 \times 88.89} = 1.75 \text{ m} \]
\[ M_{\text{max}} = -300 \times 1.75 + 88.89 \times 1.5 \times \left(\frac{0.5 + 1.72}{2}\right) = -187.5 \text{kNm} \]

\[ d = \sqrt{\frac{M}{R_c b}} = \sqrt{\frac{187.5 \times 16^6}{0.874 \times 1500}} = 378.18 \text{ mm} \]

For \( \phi 16 \) as main reinforcement

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

**Hence provided \( D = 430 \text{ mm} \)**

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)

\[ V = 500 - 88.89 \times (0.3 + 0.382)^2 = 458.655 \text{ kN} \]

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

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

\[ k_s = 0.5 + \frac{\text{Short side of column}}{\text{Long side of column}} = 0.5 + 1 = 1.5 > 1 \]

**Hence \( k_s = 1 \)**

\[ \tau_c = 0.16 \sqrt{f_{c'k}} = 0.16 \sqrt{15} = 0.62 \text{ N/mm}^2 \]

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

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* 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 $M = 187.50 \text{kNm}$

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

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

Hence provided 21ø16 as longitudinal tensile reinforcement

Check for Development Length at point of Inflection

Let $n$ be the number of ø16 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, $M_1 = f_y A_{st} f_y d$

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

$$= 9.298n \text{ kNm}$$

$V = 262.39 \text{ kNm}$

$L_d$ greater of $12 \times ($dia of bar$) = 12 \times 16 = 192 \text{ mm or } d = 382$. i.e. $L_d = 382 \text{ mm}$

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

Substituting above values, the equation for $L_d$:

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

or $n > 15.56$

Hence all the 21 number ø16 may be extended beyond point of inflection for a distance of effective depth (382 mm) and, thereafter, only alternative bars may be extended upto the edge of footing as nominal reinforcement.

Longitudinal Tensile Reinforcement at column $C_1$

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

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

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

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

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

$$= -11.46 \text{ kNm (hogging B.M.)}$$
Longitudinal tensile reinforcement for maximum sagging moment $= 9.38 \text{kNm}$ will only be designed.

$$A_n = \frac{X}{\sigma_{n} J P d} = \frac{9.38 \times 10^6}{140 \times 0.865 \times 382} = 202.77 \text{mm}^2 < A_{n \text{min}} (= 1948 \text{mm}^2)$$

**Hence provided 10$\phi 16$ ($A_n = 2010 \text{mm}^2$)**

**Longitudinal Tensile Reinforcement at column $C_2$**

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

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

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

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

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

$$= 106.502 \text{kNm}$$

Longitudinal tensile reinforcement for maximum sagging moment

$$A_n = \frac{M}{\sigma_{n} J P d} = \frac{121.5 \times 10^6}{140 \times 0.865 \times 382} = 2626.445 \text{mm}^2 > A_{n \text{min}} (= 1948.2 \text{mm}^2)$$

**Hence provided 14 number $\phi 16$ ($A_n = 2814 \text{mm}^2$)**

**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 $= k\tau_c = 1 \times 0.287 \text{N/mm}^2 > \tau_v$

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

\[
p\% = \frac{10 \times 201}{1500 \times 382} \times 100 = 0.351\%
\]

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

Permissible shear stress \( = k\tau_c = 1 \times 0.248 = 0.248 \text{ N/mm}^2 < \tau_v \)

Hence shear reinforcement will be provided for a shear force of

\[
V_j = V - V_c = 227.997 - 0.248 \times 1500 \times 382 \times 10^{-3} = 85.893 \text{ kN}
\]

If \(8\)-\(8\) legged stirrups are provided

\[
s_v = \frac{\sigma_{sy} A_{dy}}{V_j} = \frac{140 \times 8 \times 50 \times 382}{85.893 \times 10^3} = 249.05 \text{ mm/c}
\]

Spacing for minimum shear reinforcement is given by

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

The spacing shall also not exceed \(0.75 \times 382 = 286.5 \text{ c/c}\)

**Hence provided \(8\)-\(8\) legged stirrups @\(145 \text{ c/c}\)**

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

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

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

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

Permissible shear stress \( = k\tau_c = 1 \times 0.287 = 0.287 \text{ N/mm}^2 < \tau_v \)

Hence shear reinforcement will be provided for a shear force of

\[
V_j = V - V_c = 262.39 - 0.287 \times 1500 \times 382 \times 10^{-3} = 97.939 \text{ kN}
\]

If \(8\)-\(8\) legged stirrups are provided

\[
s_v = \frac{\sigma_{sy} A_{dy}}{V_j} = \frac{140 \times 8 \times 50 \times 382}{97.939 \times 10^3} = 218.42 \text{ mm/c}
\]

**Hence provided \(8\)-\(8\) legged stirrups @\(145 \text{ c/c}\)**

(as this is according to minimum shear reinforcement)

*Transverse Reinforcement*

i) **Under Column \(C_1\)**

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

\[ b' = 0.25 + 2 \times 0.382 = 1.014m \]

\[ b' = 0.25 + 2 \times 0.382 = 1.014m \]

\[ \therefore \text{Net upward pressure} = \frac{300}{1.014 \times 1.5} = 197.239 \text{ kN/m}^2 \]

Considering 1m wide strip

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

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

\[ A_{st} = \frac{M}{\sigma_{st} f_y d} = \frac{38.52 \times 10^6}{140 \times 0.865 \times 368} = 864.36 \text{ mm}^2 \]

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

Hence \[ A_{st} = 967.5 \text{ mm}^2/m \]

spacing: \[ \frac{1000 \times 113}{967.5} = 116.80 \text{ mm c/c} \]

Hence provided \#12 @115 c/c in a width of 1.014 m.

ii) Under column \( C_2 \)

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

\[ b' = 0.3 + 2 \times 0.382 = 1.064 \text{ m} \]

Net upward pressure = \[ \frac{500}{1.064 \times 1.5} = 313.28 \text{ kN/m}^2 \]

Considering 1m wide strip

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

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

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

spacing: \[ \frac{1000 \times 113}{1265.35} = 89.30 \]

Hence provided \#12 @85 for a width of 1.064 m under column \( C_2 \)

The detailing of reinforcement have been shown in Figure 17.3.
Example 17.2

Two columns having cross-section of 500 x 500 mm and 600 x 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/m². Design a combined rectangular footing with beam joining the columns.

Solution

Loads

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

Total load = 3300 kN

Size of Footing

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

Hence provided size of footing = 6.5 m x 1.7 m = 11.05 m² > 11 m²

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 \(\bar{x} = 2.916 \text{ m}\)
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

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

\[ \therefore \text{Projection of slab beyond the longitudinal face of beam} = \frac{17 - 0.6}{2} = 0.55 \text{ m} \]

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

\[ \therefore d = \sqrt{\frac{M}{R_kb}} = \sqrt{\frac{41.06 \times 10^6}{0.874 \times 1000}} = 216.75 \text{ mm} \]

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

Provided \( D = 350 \text{ mm} \)

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

\[ A_{n} = \frac{M}{\sigma_n j_b d} = \frac{41.06 \times 10^6}{140 \times 0.865 \times 288} = 1177.29 \text{ mm}^2 \]

\[ \therefore \text{Provided } \phi 12@95 \text{ c/c } (= 1189.47 \text{ mm}^2/\text{m}) \]

**Check for shear**

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

\[ = 71.13 \text{ kN/m width} \]

\[ \tau_v = \frac{7113 \times 10^3}{1000 \times 288} = 0.247 \text{ N/mm}^2 \]

\[ p\% = \frac{1189.47}{1000 \times 288} \times 100 = 0.41\% \]

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

Permissible shear stress = \( k \tau_c = 1 \times 0.265 = 0.265 \text{ N/mm}^2 > \tau_v \)

Hence no shear reinforcement is necessary.

**Check for development length**

\[ L_d = \frac{\phi \sigma_s}{4 \tau_{pd}} = \frac{12 \times 140}{4 \times 0.6} = 700 \text{ mm} \]

Straight length available beyond the face of the beam = 550 - 40 = 510 \(< L_d \)

With standard U-hook total length = 510 + 16 \times 12 = 702 > 700 Hence O.K.

**Design of Beam**

The beam will act as a T-beam in the span between points of contraflexure and as rectangular beam in the projected portion.
Design of beam between $C_1$ & $C_2$

\[ l_o \text{ for effective width calculation of isolated T-beam} = 5 - (0.024 + 0.273) = 4.703 \text{ m} \]

\[ b_j = l_o - b_n = \frac{4.703}{4.703 + 4} = 1.295 \text{ m} < 1.7 \text{ m} \]

For balanced section

assuming $j_B = 0.9$

\[ M = 0.45 D_f d_f \sigma_{c_e} \left( \frac{2k_B d - D_f}{k_B} \right) \]

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 $d = 1150 \text{ mm}$

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

Areas of steel ($A_u$)

i) For critical B.M. in the span

\[ M = \sigma_u \times A_u \times l_a \]

\[ l_a = 0.9d = 0.9 \times 1062 = 955.8 \text{ mm} \]

\[ \therefore \quad A_u = \frac{M}{\sigma_u l_a} = \frac{1275.24 \times 10^6}{130 \times 955.8} = 10263.17 \text{ mm}^2 \]

**Hence provided 13$\phi$32 in two layers ($A_u = 10452 \text{ mm}^2$)**

Check for Development Length

\[ L_d \leq \frac{M_1}{V} + L_o \]

where \[ M_1 = \sigma_u A_u j_B d \]

\[ = 140 \times n \times 804 \times 0.9 \times 1062 \]

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

\[ V = 1085.86 \text{ kN} \]

\[ L_o = \text{ greater of } 12\phi (=12 \times 32 = 384) \text{ or } d = 1062 \]

Substituting all values in the above equation

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

or \[ n \geq 8.11 \]

Hence all 13$\phi$32 have been extended up to the edge of footing on R.H.S. and on L.H.S. all the bars may be bent at 90° to make up $L_o$ at the edge.
ii) For B.M. at the face of column on R.H.S. projection

Load per m run = 271.49 \times 1.7 = 461.53 \text{ kN/m}

\[ M = 461.53 \times \frac{(1.166 - 0.3)^2}{2} = 173.064 \text{ kNm} \]

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

\[ A_{st} = \frac{173.064 \times 10^6}{130 \times 0.865 \times 1094} = 1406.792 \text{ mm}^2 \]

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

Hence provided 8\#20 (\( A_{st} = 2513.27 \text{ mm}^2 \))

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 8\#20 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

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

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

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

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

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

\[ s_v = \frac{\sigma_{fy}A_{sy}d}{V_s} = \frac{140 \times 4 \times 78 \times 1062}{447.124 \times 10^3} = 103.75 \text{ mm} \]

Hence provided 10\#4 legged stirrups @ 100 c/c

The details of reinforcements have been shown in Figure 17.5.
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 x 400 mm and 500 x 500 mm respectively. The distance between the columns is 3.5 m and the bearing capacity of soil is 250 kN/m².

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 x 240 mm and 300 x 300 mm are loaded with 300 kN and 500 kN respectively. The c/c distance between the column is 4 m. The bearing capacity of soil is 100 kN/m². 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 = 80 kN
(assuming 10% of superimposed)
Total load = 880 kN

Size of Footing

Required area of footing = \( \frac{880}{100} \) = 8.8 m²

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

or \( B_1 + B_2 = 4.12 \) m

(d) Bending Moment Diagram

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 \( \bar{x} = \frac{500 \times 4}{(300 + 500)} = 2.5 \) 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 \( B_1 + 2B_2 = 1.84 (B_1 + B_2) \)

or \( -0.84 B_1 + 0.16 B_2 = 0 \) \( \ldots(17.2) \)

Solving simultaneous equations 17.1 and 17.2

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

\[ \therefore \] Net upward pressure on footing

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

**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 \( C_1 \)

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

\[ = 0.779 \text{m} \]

\[ \therefore \] S.F. at centroid of \( C_1 \)

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

\[ = -7.917 \text{kN} \]

**S.F. at R.H.S. of centroid of \( C_1 \)**

\[
= 300 - 7.917 = 292.083 \text{kN}
\]

**S.F. at R.H.S. of centroid of \( C_2 \); Breadth at centroid of \( C_2 \)**

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

\[ = 3.402 \text{m} \]

\[ \therefore \] S.F. at R.H. of centroid of \( C_2 \)

\[
= 89.22 \times \frac{3.402 + 3.5}{2} \times 0.15
\]

\[ = 46.185 \text{kN} \]

\[ \therefore \] S.F. at L.H. of centroid of \( C_2 \)

\[
= -500 + 46.185 = -453.815 \text{kN} \]
Let S.F. = 0 at \( x \), then

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

or

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

or

\[
62.454 x + 29.264 x^2 - 300 = 0
\]

or

\[
x^2 + 2.134 x - 10.252 = 0
\]

or

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

**B.M. at centroid of** \( C_1 \)

\[
+7.917 \times \frac{(2 \times 0.7 + 0.779)}{(0.7 + 0.779) \times \frac{0.12}{3}} = 0.467 \text{ kNm}
\]

**B.M. at centroid of** \( C_2 \)

\[
+46.185 \times \frac{(3.402 + 2 \times 3.5)}{2} \times \frac{0.15}{3} = 12.010 \text{ kNm}
\]

Max. B.M. at \( x = 2.308 \)

\[
B_x = 0.7 + 0.656 \times 2.308
\]

\[
= 2.214 \text{ m}
\]

\[
M_{\text{max}} = 300 \times \frac{(2 \times 0.7 + 2.214)}{(0.7 + 2.214)} \times \frac{2.308}{3} - 300 \times (2.308 - 0.12)
\]

\[
= 286.243 - 656.4 = -370.157 \text{ kNm}
\]

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

---

**Depth of Footing from Bending Moment Consideration**

\[
d = \sqrt{\frac{M}{R_{p,b}}} = \sqrt{\frac{370.157 \times 10^6}{0.865 \times 2214}} = 439.639
\]

**Depth of Footing from Two-way Shear Consideration**

i) Under column \( C_1 \) (Figure 17.7)

\[
b_0 = (b_1 + 2 \times \frac{d}{2}) + 2 \times (b_1 + \frac{d}{2})
\]
RCC Structures-I

\[ = (0.240 + d) + (0.48 + d) \]
\[ = (0.72 + 2d) \]

Shear force on critical section

\[ = P_1 - p (b_1 + d) \left( b_1 + \frac{d}{2} \right) \]
\[ = 300 - 89.22 \times (0.24 + d) \left( 0.24 + \frac{d}{2} \right) \]
\[ = 300 - 89.22 \times (0.24 + d) \left( 0.24 + \frac{d}{2} \right) \]
\[ = 294.861 - 32.119d - 44.61d^2 \]
\[ \therefore \tau = \frac{294.861 - 32.119d - 44.61d^2}{b_0d} \]
\[ = \left( \frac{294.861 - 32.119d - 44.61d^2}{0.72 + 2d} \right) \times 10^3 \]

\[ k_c = (0.5 + 1) = 1.5 > 1 \]

Hence \( k_c = 1 \)

\[ \therefore \tau_c = k_c \times \frac{0.16 \sqrt{15}}{1} = 1 \times 0.619 = 0.619 \text{ N/mm}^2 \]

Equating \( \tau \) and \( \tau_c \)

\[ \frac{294.861 - 32.119d - 44.61d^2}{0.72 + 2d} \times 10^3 = 0.619 \]

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 \[ d = \frac{-0.372 \pm \sqrt{0.138 + 0.916}}{2} = 0.3206 \text{ m} = 320.6 \text{ mm} \]

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

\[ = P_2 - p (b_2 + d) \left( b_2 + \frac{d}{2} \right) \]
\[ = 500 - 89.22 \times (0.3 + d) \left( 0.3 + \frac{d}{2} \right) \]
Hence $k_c = 1$

Therefore maximum value of $d$ from above considerations = 439.639

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

Hence provided $D = 520$

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

\[ A_m = \frac{M}{\sigma_s J_g d} = \frac{370.157 \times 10^6}{140 \times 0.865 \times 470} = 653.45 \text{ mm}^2 \]

Hence provided 21\#20

Check for development length near column $C_1$

Let 14\#20 are only extended upto the column $C_1$

\[ M_1 = \sigma_s A_m J_g d = 140 \times 14 \times 314 \times 0.865 \times 470 = 250.21 \times 10^6 \text{ Nmm} \]

\[ V = 292.083 \text{ kN} \]

Providing 90° bend,
\[ L_0 = 120 - 40 + 8\phi = 120 - 40 + 8 \times 20 = 240 \]
\[ L_d \leq \frac{1.3M_1}{V} + L_0 \]
\[ \text{or } 58.3\phi \leq \frac{1.3 \times 250.21 \times 10^6}{292.083 \times 10^3} + 240 \]
\[ \text{or } \phi \leq 23.22 \text{ Hence O.K.} \]

**Check for development length under column } C_2 \**

Let all 21\#20 extend up to the R.H. edge of footing

\[ M_1 = \sigma_{st} A_{pl} f_y d = 140 \times 21 \times 314 \times 0.865 \times 470 = 375.31 \text{ kNm} \]
\[ V = 453.815 \text{ kN} \]
Providing 90\(^\circ\) bend at the edge,
\[ L_0 = 150 - 40 + 8 \times 20 = 270 \]
\[ L_d = 58.3 \phi \]
\[ L_d \leq \frac{1.3M_1}{V} + L_0 \]
\[ \text{or } 58.3\phi \leq \frac{1.3 \times 375.31 \times 10^6}{453.815 \times 10^3} + 270 \]
\[ \text{or } \phi \leq 23.07 \text{ Hence O.K.} \]

**Check for one-way shear**

i) **Near column } C_1 \**

The shear force at critical section } d \text{ from inner face of column } C_1 \text{ is given by}

\[ \dot{B}_x = \dot{B}_1 + \frac{B_2 - B_1}{L} x = 0.7 + 0.656 \times (0.24 + 0.47) = 1.165 \text{ m} \]
\[ V = \frac{B_1 + B_x}{2} \left( b_1 + d \right) + 300 = 89.22 \times \frac{(0.7 + 1.165)}{2}(0.24 + 0.47) + 300 \]
\[ = 240.93 \text{ kN} \]
\[ \tau_v = \frac{V}{bd} = \frac{240.93 \times 10^3}{1165 \times 470} = 0.44 \text{ N/mm}^2 \]
\[ p\% = \frac{A_{nt}}{bd} \times 100 = \frac{14 \times 314}{470 \times 1165} \times 100 = 0.803\% \]
\[ \tau_\phi = 0.34 + \frac{(0.37 - 0.34)}{0.25} \times (0.803 - 0.75) = 0.346 \text{ N/mm}^2 \]
\[ k = 1 \]
\[ \therefore \text{ Permissible shear stress } \]
\[ = k\tau_v = 1 \times 0.346 = 0.346 \text{ N/mm}^2 < \tau_v \]
Assuming $8-8$ legged stirrups

\[
A_{sv} = 50.26 \times 8 = 402.08 \text{ mm}^2
\]

\[
V_r = \tau_c bd = 0.346 \times 1165 \times 470 = 189.45 \times 10^3 \text{ N}
\]

\[
\therefore V_e = V - V_r = 240.93 \times 10^3 - 189.45 \times 10^3 = 51.48 \times 10^3 \text{ N}
\]

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

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

Spacing is minimum of

i) 917.72
ii) 187.67
iii) 0.75d = 0.75 \times 470 = 352.5
iv) 450

Hence provided $8-8$ 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,

\[
B_x = B_1 + \frac{B_2 - B_1}{L} x
\]

\[
= 0.7 + 0.656 \times (4.27 - 0.3 - 0.47) = 2.996 \text{ m}
\]

\[
V = \rho \frac{B_x + B_2}{2} (b_2 + d) - 500
\]

\[
= 89.22 \times \frac{(2996 + 3.5)}{2} (0.3 + 0.47) - 500
\]

\[
= 276.864 \text{ kN}
\]

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

\[
p\% = \frac{A_{sv}}{bd} \times 100 = \frac{21 \times 314}{2996 \times 470} \times 100 = 0.468\%
\]

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

\[
\therefore \text{Permissible shear stress } k\tau_c = 1 \times 0.281 = 0.281 \text{ N/mm}^2 > \tau_v
\]

Hence no shear reinforcement is required at this end.

Transverse Reinforcement

i) Under Column $C_1$

Width of footing at the centre of column $C_1 = 0.7 + 0.656 \times 0.12 = 0.78 \text{ m}$
Projection of slab at the centre of column = \( \frac{(0.78 - 0.24)}{2} = 0.27 \text{ m} \)

Width of bending strip = 0.24 + 0.47 = 0.71 m

Width of footing at 0.71 m

\[ = 0.7 + 0.656 \times 0.71 = 1.159 \text{ m} \]

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

\[ \therefore \text{ Upward pressure} = \frac{300}{0.664} = 451.81 \text{ kN/m}^2 \]

Maximum B.M. at the face of column

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

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

\[ \therefore A_{sl} = \frac{16.468 \times 10^6}{140 \times 0.865 \times 450} = 302.193 \text{ mm}^2 \]

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

Spacing for \#12 bars = \( \frac{710 \times 113}{608.4} = 131.87 \)

Hence provided \#12@130 c/c

i) Under column \( C_2 \)

Width of footing at the centre of column \( C_2 = 0.7 + 0.656 \times 4.12 = 3.4 \text{ m} \)

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

Width of bending strip = 0.3 + 0.47 = 0.77 m

Width of footing at 0.77 m from right edge

\[ = 0.7 + 0.656 \times 3.5 = 2.996 \text{ m} \]

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

Net upward pressure = \( \frac{500}{2.5} = 200 \text{ kN/m}^2 \)

B.M. at the face of column

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

\[ A_{sl} = \frac{240.25 \times 10^6}{140 \times 0.865 \times 450} = 4408.66 \text{ mm}^2 \]

Hence provided \#20@50 c/c
The detailing of reinforcement have been shown in Figure 17.9.

![Diagram of Trapezoidal Footing](image)

(a) Detailing of the Trapezoidal Footing

![Diagram of Plan](image)

(b) Plan showing Longitudinal Reinforcement

**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 $240 \times 240$ mm and $300 \times 300$ 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**

- Superimposed 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 = \( \frac{880}{80} = 11 \text{ m}^2 \)
- Keeping width of footings under both columns, \( B = 2 \text{ m} \)
- Sum of lengths of footings, \( (L_1 + L_2) = 8.8 \text{ m} \)
- C.G. of loads from column \( C_2 \)

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 \( C_2 \)
\[ \bar{x}_2B(L_1 + L_2) = 2L_1 \left( 4 + 0.12 \frac{L_1}{2} \right) \]

or \[ 1.5 \times 2 \times 4.4 = 2L_1 \left( 4.12 \frac{L_1}{2} \right) \]

or \[ L_1^2 - 8.24L_1 + 13.2 = 0 \]

or \[ L_1 = 2.177 \approx 2.2 \text{ m} \]

\[ \therefore L_2 = 4.4 - 2.2 = 2.2 \text{ m} \]

\[ \therefore \text{Net upward pressure on footing} = \frac{800}{2(2.2+2.2)} = 90.91 \text{ kN/m}^2 \]

Design of Footing Slab

Let the width of strap beam = 400 mm

The projection of slab beyond the longitudinal face of beam = \[ \frac{2 - 0.4}{2} = 0.8 \text{ m} \]

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

\[ \therefore d = \sqrt{\frac{M}{K_p b}} = \sqrt{\frac{29.09 \times 10^6}{0.865 \times 1000}} = 183.38 \text{ mm} \]

Adopting \#12 bars

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

**Hence provided** \[ D = 230 \text{ mm} \]

\[ \therefore D = 230 - 40 - \frac{12}{2} = 184 \]

Check for one way-shear

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

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

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

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 \( D = 230 \)

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

\[ \therefore \text{Permissible shear stress} = 1.14 \times 0.334 = 0.381 \text{ N/mm}^2 > \tau_v \]
Hence no shear reinforcement is necessary in the slab

\[ A_{sh} = \frac{M}{\sigma_{sh} J_{bd}} = \frac{29.09 \times 10^6}{140 \times 0.865 \times 126} = 1906.47 \text{ mm}^2 \]

Hence provided \( \phi 12 \@ 85 \)

Distribution steel

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

Hence provided \( \phi 10 \@ 225 \)

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

\[ \therefore d = \sqrt{\frac{M}{R_{sh} b}} = \sqrt{\frac{183996 \times 10^6}{0.874 \times 400}} = 725.47 \text{ mm} \]

Let dia of bar be 25mm

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

Hence provided \( D = 850 \)

\[ \therefore d = 850 - 40 - \frac{25}{2} = 797.5 \text{ mm} \]

\( A_{sh} \)

i) Main reinforce in the span

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

\[ \therefore \text{ taking } j \approx 0.9 \]

\[ A_{sh} = \frac{21118 \times 10^6}{130 \times 0.9 \times 797.5} = 2263.27 \text{ mm}^2 \]

Hence provided \( 5\phi 25 \) (\( A_{sh} = 2454.37 \text{ mm}^2 \))

Check for development length at point of contra-flexure

\[ M_i = \sigma_{sh} \times n \times \frac{\pi}{4} \times 25^2 \times 0.9 \times d \]

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

\[ = 45.802 n \text{ kNm} \]

\[ V = 223.63 \text{ kN} \]

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

\[ L_d = 58.3 \times 25 = 1457.25 \]

\[ L_d \leq \frac{M_i}{V} + l_0 \]
or \( n \geq 3.22 \)

Hence all 5\( \phi25 \) are extended up to the right edge.

ii) **Main reinforcement at support \( C_2 \)**

B.M. at exterior face of column \( C_2 \)

\[
= 90.91 \times \frac{(11 - 0.15)^2}{2} = 41.023 \text{ kNm}
\]

B.M. at interior face of column \( C_2 \)

\[
= 90.91 \times \frac{(11 + 0.15)^2}{2} = 500 \times 0.15 = -3.976 \text{ kNm}
\]

Hence design moment, \( M = 41.023 \text{ kNm} \)

\[
A_{\text{u}} = \frac{41023 \times 10^6}{140 \times 0.865 \times 797.5} = 424.769 \text{ mm}^2
\]

**Hence provided 3\( \phi16 \) \((A_{\text{u}} = 603 \text{ mm}^2)\)**

**Check for Development Length**

At point of contra-flexure

\[
M_1 = \sigma_u \times n \times \frac{\pi}{4} \times \phi^2 \times jd
\]

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

\[
= 19.418n \text{ kNm}
\]

\( V = 223.63 \text{ kN} \)

\( l_0 = 797.5 \)

\( L_d = 932.8 \text{ mm} \)

\[
L_d \leq \frac{1.3M_1}{V} + l_0
\]

\[
932.8 \leq \frac{1.3 \times 19.418n \times 10^6}{22363 \times 10^3} + 797.5
\]

or \( n \geq 1.19 < 3 \) Hence O.K.

**Hence all 3\( \phi16 \) will be extended up to a distance \( d \) beyond point of contra-flexure in the span.**
Check for shear

i) At column $C_1$

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

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

$= 111.36 \text{ kN}$

$\tau_v = \frac{11136 \times 10^3}{400 \times 797.5} = 0.349 \text{ N/mm}^2$

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

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

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

$V_s = V - V_c = 111.36 - 109.23 = 2.13 \text{ kN}$

Using 8-2 legged stirrups

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

$s_v = \frac{\sigma_{sv} \times A_{sv} \times d}{V_s} = \frac{140 \times 100 \times 797.5}{2.13 \times 10^3} = 52417$

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

The spacing shall be minimum of:

i) 5241.7

ii) 135.94

iii) 0.75 $d = 0.75 \times 797.5 = 598.125$

iv) 450

Hence provided 8-2 legged stirrups @135 c/c

ii) At column $C_2$

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

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

$= -127.72 \text{ kN}$

$\tau_v = \frac{V}{bd} = \frac{12772 \times 10^3}{400 \times 797.5} = 0.4 \text{ N/mm}^2$

$p\% = \frac{5 \times 490}{400 \times 797.5} \times 100 = 0.77\%$
\[ \tau_v = 0.22 + \frac{(0.37 - 0.34)}{0.25} \times (0.77 - 0.75) = 0.3424 \text{ N/mm}^2 < \tau_y \]

\[ V_f = V - V_c \]

\[ = 127.72 - 0.3424 \times 400 \times 797.5 \times 10^{-3} = 18.494 \text{ kN} \]

Using \$8-2$ legged stirrups

\[ s_v = \frac{\sigma_y \times A_y \times d}{V_f} = \frac{140 \times 100.26 \times 797.5}{18.494 \times 10^3} = 605.279 \]

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

\[ V = 90.91 \times 2 \times (1.1 - 0.15 - 797.5) = 27.72 \text{ kN} \]

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

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 \$8-2$ legged stirrups @135 c/c throughout the beam.**

The reinforcement detailings have been shown in Figure 17.11.
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