Plain & Reinforced Concrete-1

Seventh Term
Civil Engineering

CE-414

Lecture # 1

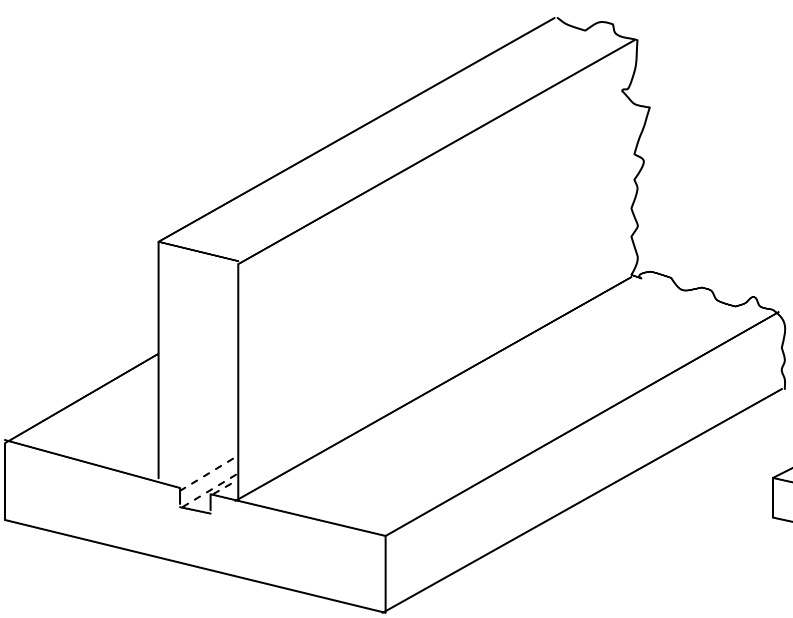
Foundations

INTRODUCTION

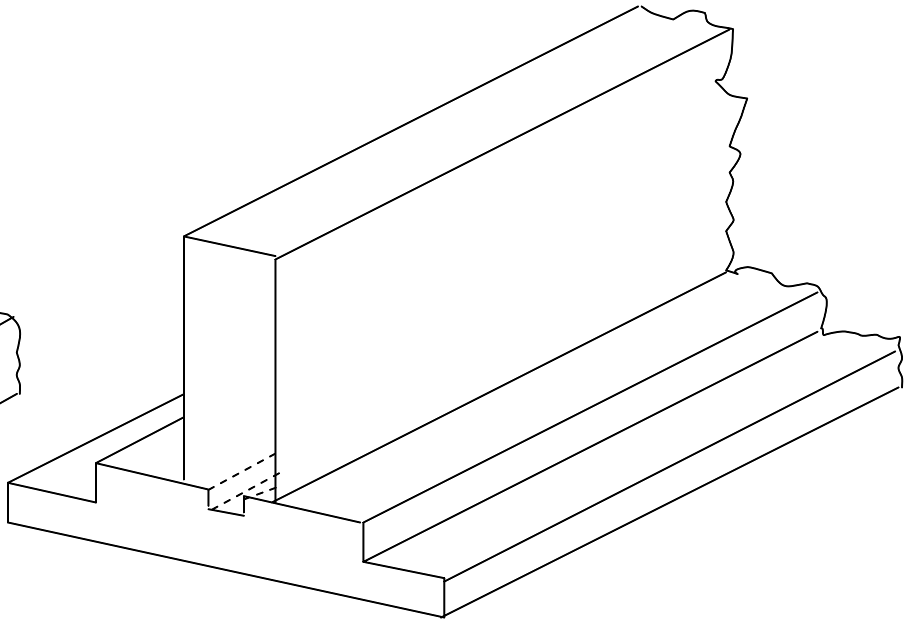
- Foundations are the structural elements that are used to transfer the structural loads to the soil giving sufficient margin of safety against its failure, without excessive settlement and rotation.
- The loads on the columns and walls are concentrated over smaller areas and the function of foundations is to spread this load over larger areas until it becomes equal to the safe bearing capacity of soil.

TYPES OF FOOTINGS

- Footings can be made in various shapes and types depending upon the use and the underlying soil characteristics.
- ***Strip*** or ***wall footings*** are used under the walls and these consist of one-way slabs cantilevered out from the bottom of wall in one or both the lateral directions.
- The wall footing may also be a ***stepped wall footing***, where the slab thickness decreases in steps away from the wall.

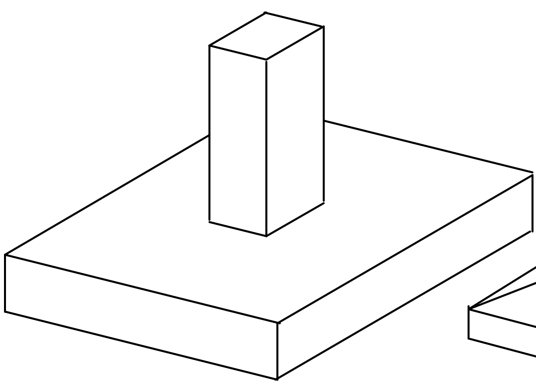


Wall or Strip Footing

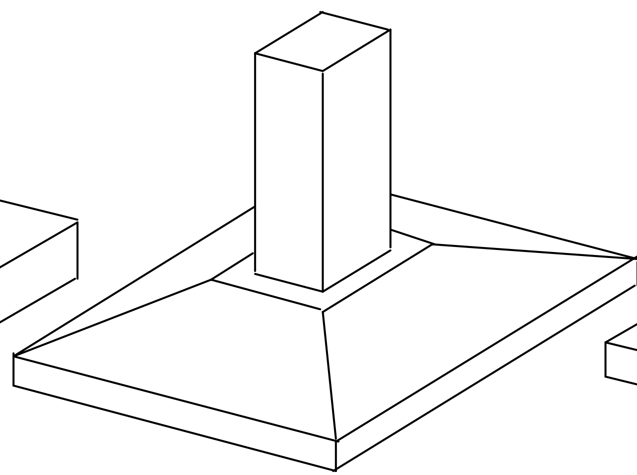


Stepped Wall Footing

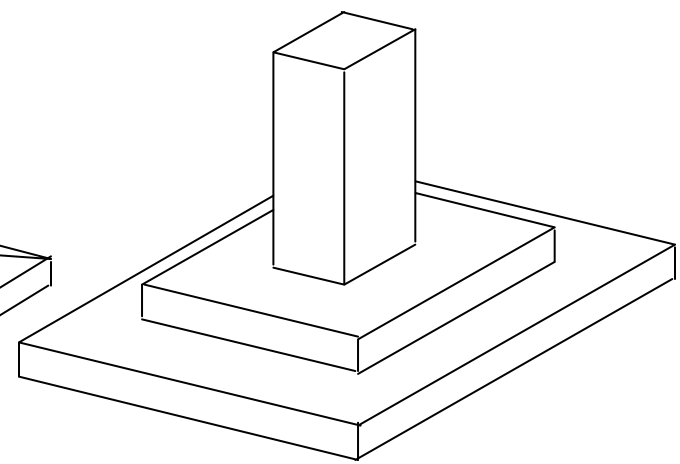
Isolated column footings can be in the form of reinforced concrete pads under the columns, which may be in the form of ***spread footings***, ***stepped footings*** or ***tapered footings***.



Isolated or Spread Footing



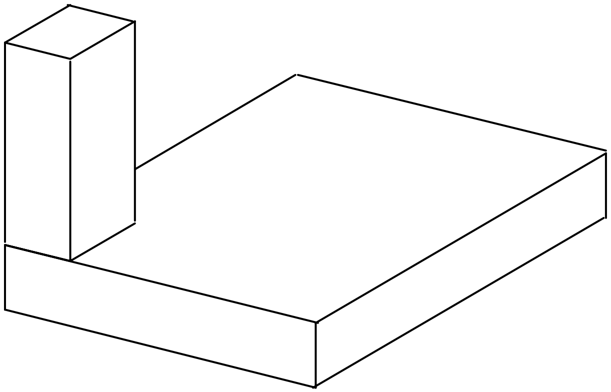
Tapered Spread Footing



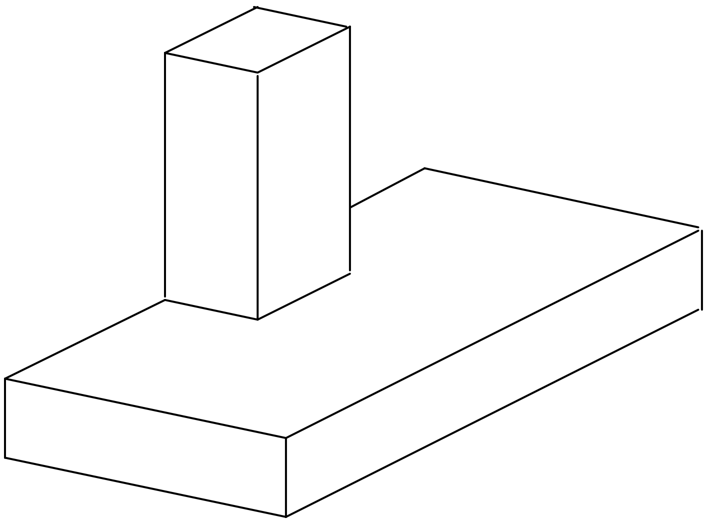
Stepped Column Footing

The isolated footing for corner column of a building, where there is site restriction on two sides, may be in the form of a slab extending as a cantilever only in two perpendicular directions.

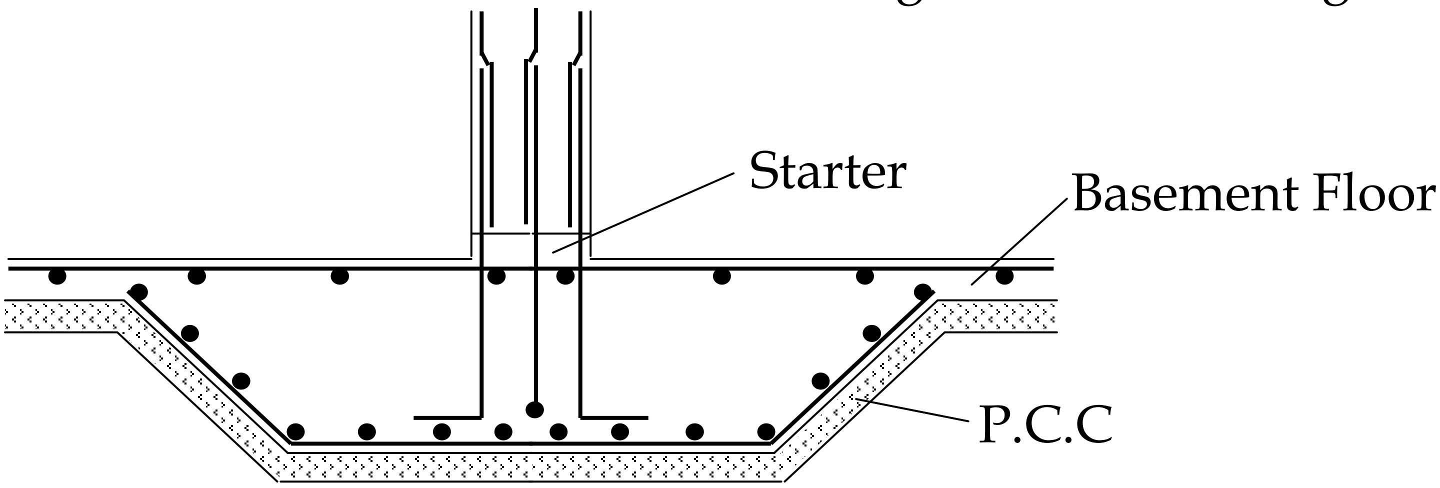
Similarly, for the edge column, the isolated footing extends on both sides of the column parallel to edge of the building but is cantilevered only on inner side in the perpendicular direction.



Corner Column Footing



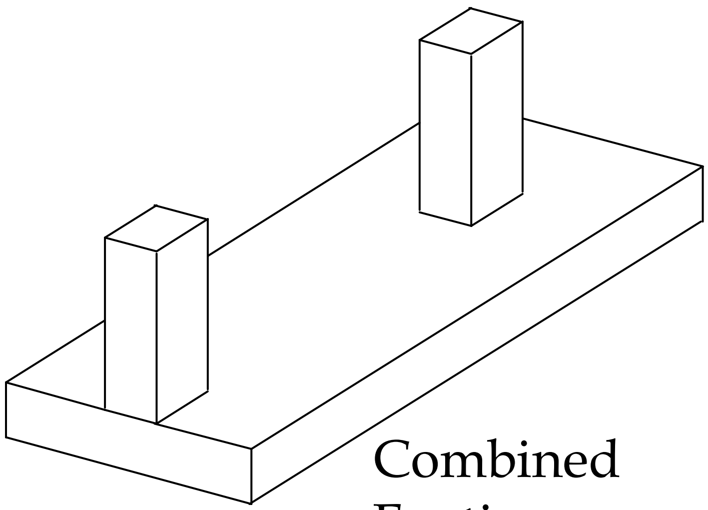
Edge Column Footing



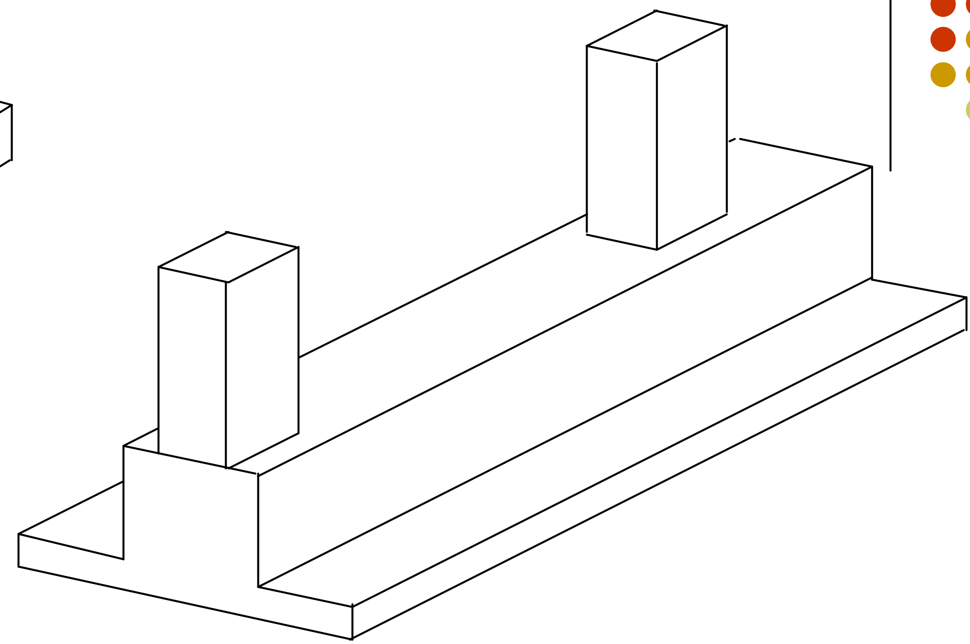
Isolated Column Footing For Basement

Two or more columns can have ***combined footing*** in the form of either a rectangular base or two individual bases connected by a narrow strip (The latter type is called ***strap footing***).

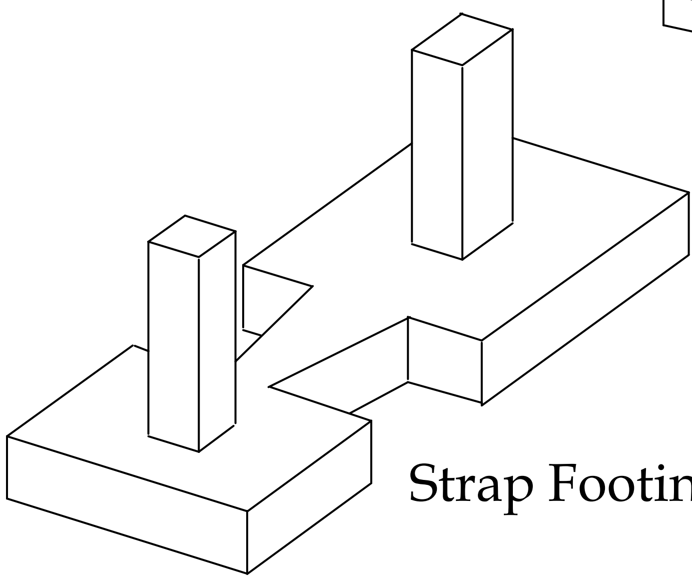
In some cases, the combined footing can have a foundation beam running along the column centerline and is called ***foundation beam footing***.



Combined Footing

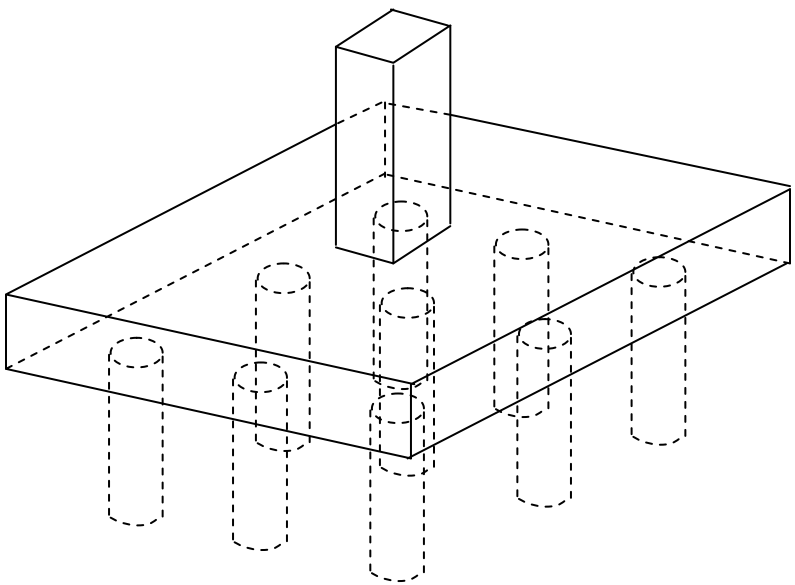


Foundation Beam Footing



Strap Footing

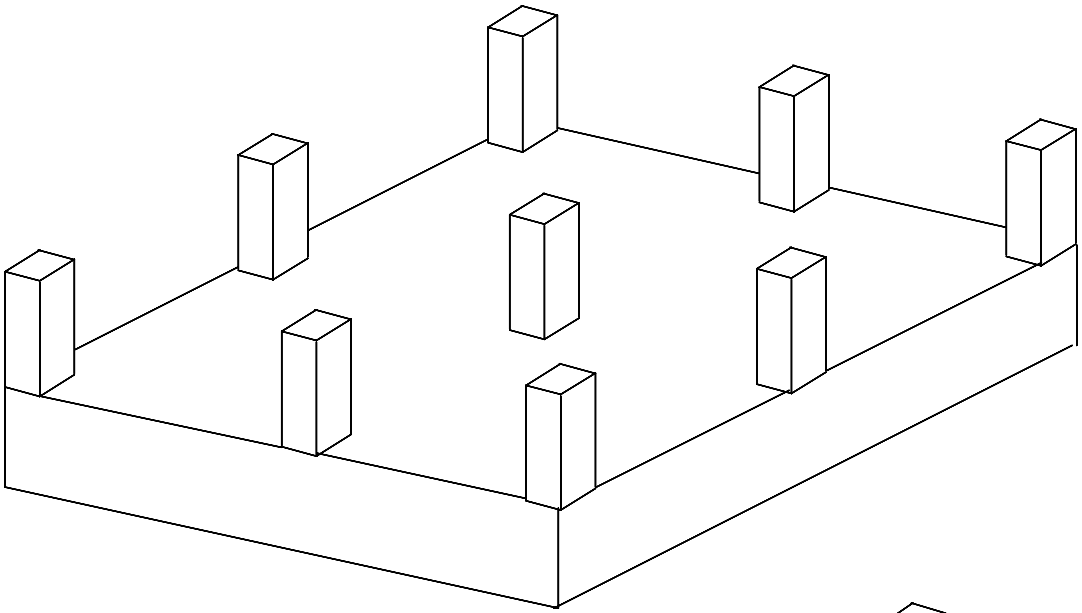
If the bearing capacity under the footing is too low compared with the applied load, the base may be supported by piles driven into the ground. The base slab in this type of foundation is designed as a pile cap and the resulting footing is called ***pile cap footing***.



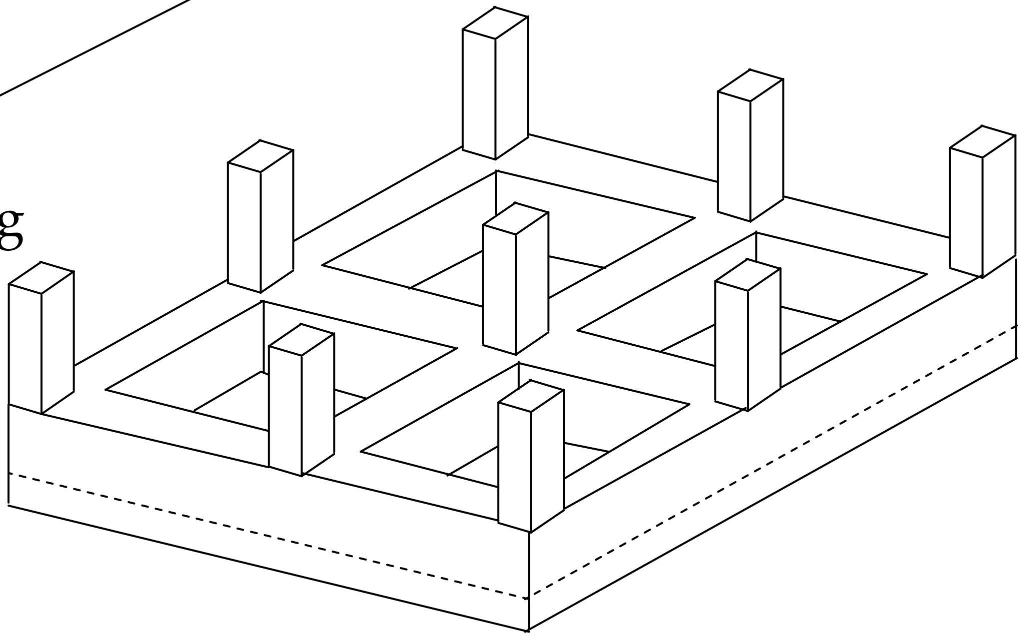
Pile Cap Footing

In case the isolated column footings for a building come closer to each other and reinforcement is required on both top and bottom of the pad, it is advantageous to combine all the footings in the form of a ***raft***, ***mat*** or ***floating foundation***.

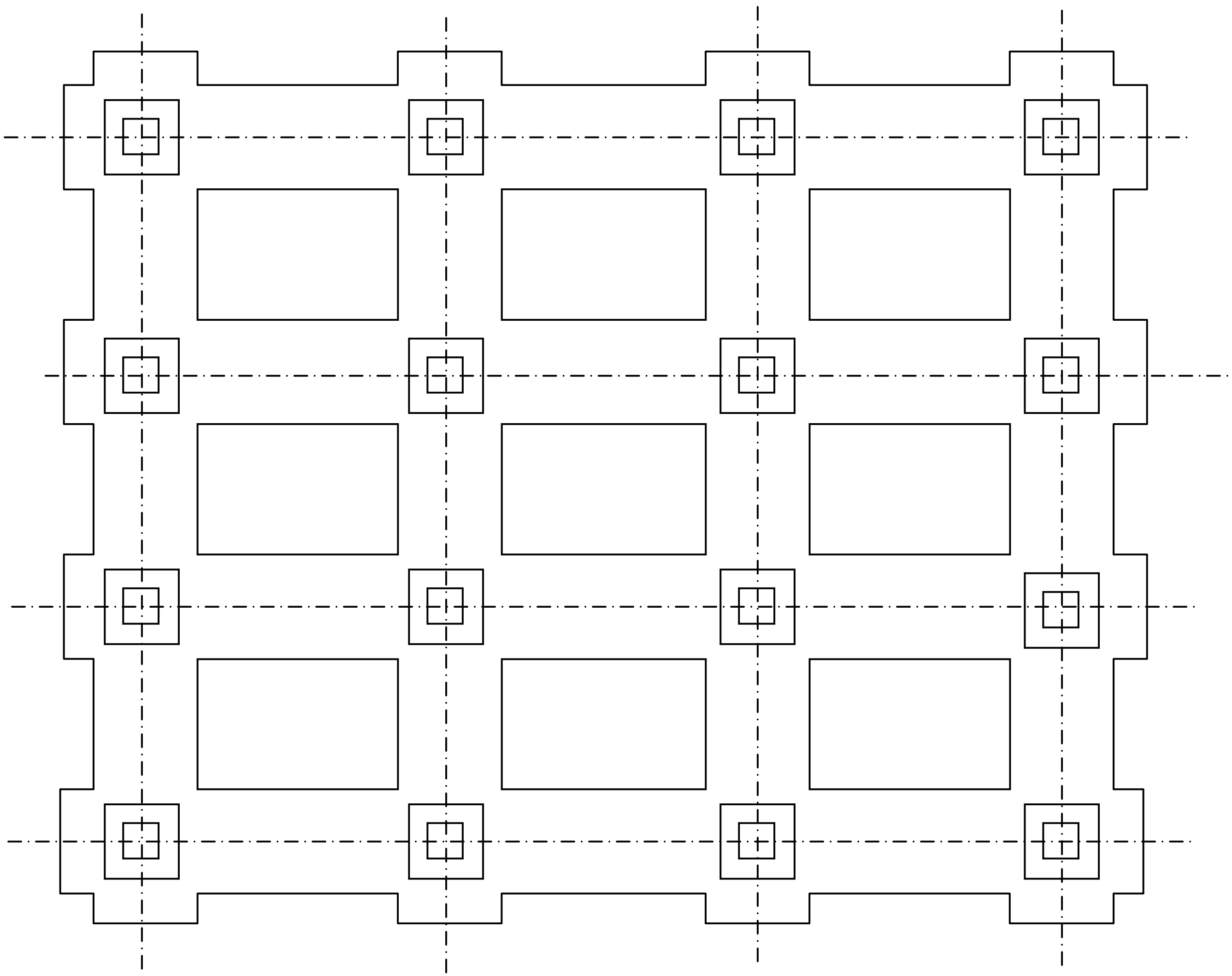
The raft may be a base slab of constant thickness or a slab supported by foundation beams at each column centerline. Raft is also effective in reducing the chances of differential settlement of different parts of the building.



Mat or Raft Footing

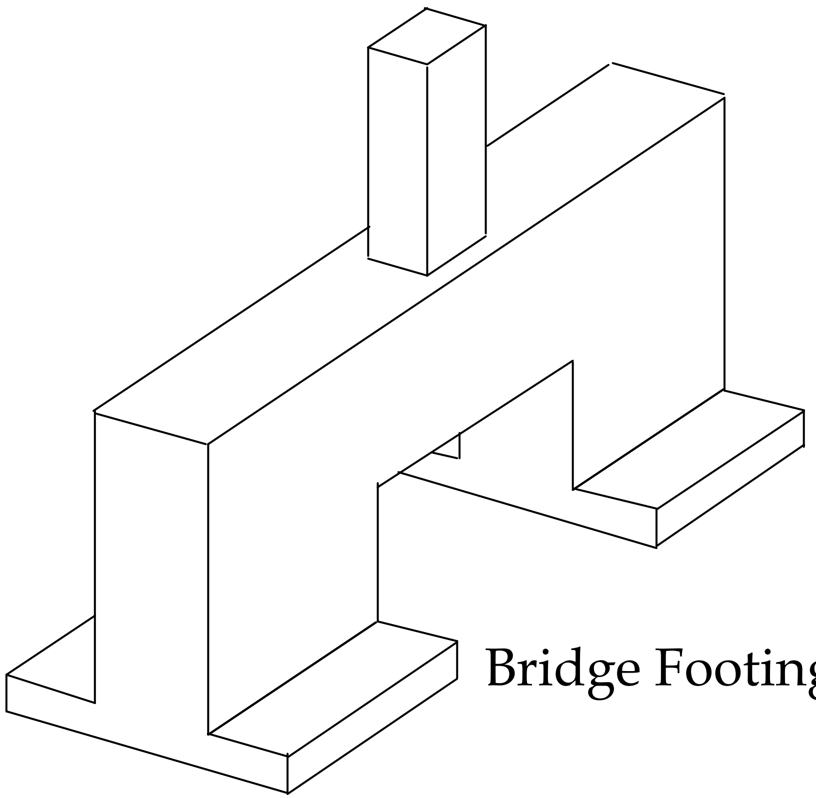


Raft Footing with Beams



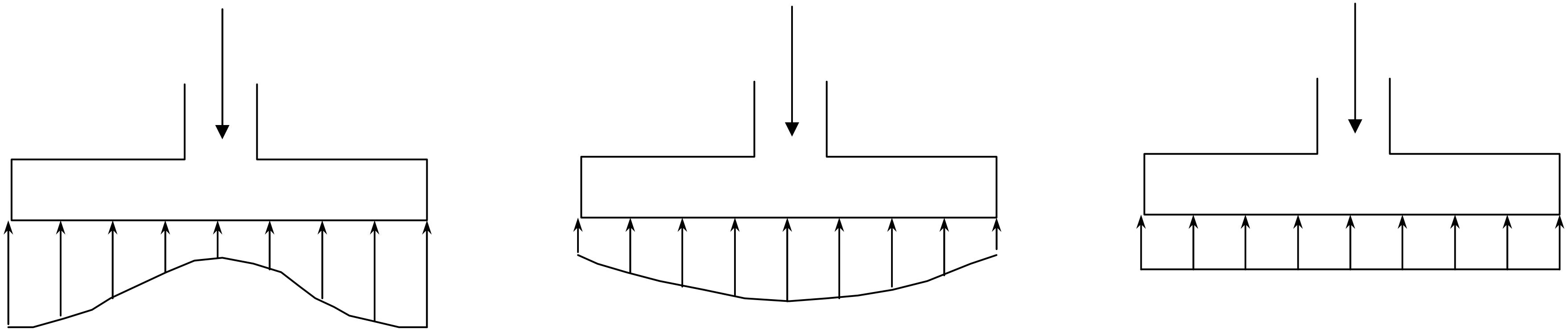
Grid or Grillage Foundation in Plan

A ***bridge footing*** is used to erect a column over a place that is already occupied.



BEARING PRESSURE

* When structure loads are concentrically applied on isolated footings, the bearing stresses are not actually uniformly distributed over the area of the footing due to deflected shape of the foundation and uneven behavior of the soil underneath (Fig. 9.2).
* Cohesive soils produce shear resistance at the edges of the footing and hence more bearing pressure is developed near the perimeter as compared with the bearing pressure inside the footing, as shown in Fig. 9.2a.
* In case of granular soils, the soil particles near the end of the foundation tend to slide towards the less stress area (sideward, outside the footing) and hence a distribution shown in Fig. 9.2b is obtained with more pressure near the center and less pressure near the ends of the footing.



a) Actual Distribution for Cohesive Soils

b) Actual Distribution for Granular Soils

c) Assumed Distribution

* The determination of actual pressure distributions and the resulting moments and shears for practical designs becomes quite cumbersome and hence simplified pressure distribution shown in Fig. 9.2c is usually utilized for design of isolated footings and other simple types of foundations.
* The assumption of uniform pressure under the footings produces insignificant difference in shears and moments within the acceptable limits of tolerance.
* For mats and other sophisticated foundations, it is better to use more refined estimates of bearing pressures in place of the common practice of using the uniform pressure distribution.

ALLOWABLE BEARING CAPACITY

* The allowable bearing capacity of a soil, denoted by q_a, at a certain level is defined as the amount of maximum external uniform pressure that may be applied with sufficient margin of safety against failure of soil and against excessive settlement.
* The bearing capacity of a shallow foundation depends on the depth, shape, type and width of foundation along with the type of soil, the natural level of compaction and consolidation, the ground water table level, the depth of frost penetration and the drainage properties of the area.

- The allowable settlement, the chances of differential settlement and the required factor of safety are also important factors in the final establishment of the bearing capacity.
- The factor of safety against the rupture of soil is taken from 2 to 3, the most common value is usually equal to 3.

- It is estimated that architectural damage to buildings occurs when the amount of differential settlement divided by the distance between the two points where the settlement is noted (called differential settlement per unit length or distortion) exceeds 1/300, and structural damage occurs when this value reaches 1/150.
- For isolated footings, the maximum allowed settlement is taken equal to 1200 times the maximum allowed distortion for clays and 600 times the distortion for sands.

- For a maximum allowed distortion of 1/300, the maximum settlement comes equal to 100 mm for clays and 50 mm for sands.
- However, a good isolated foundation is that which does not settle more than 25mm under the application of allowable bearing pressure.

- The allowable bearing capacity based on settlement is usually taken as the applied pressure producing a maximum settlement not to exceed 25mm and differential settlement not to exceed 20mm.
- For sandy soils, the settlement may be predicted by the blow counts of the standard penetration test, the larger the blow counts; the less will be the settlement.

- When backfill load is applied on any soil layer at certain depth, we need not to provide full factor of safety of 3.0, but a reduced value of 1.5 may be used.
- This means that half of the backfill load is assumed to consume part of the bearing capacity.
- For example, 1.2m of backfill with a soil of density 1600 kg/m^3 may be considered to consume $0.5 \times 1600 \times 1.2 \times 9.81/1000 = 9.4$ kPa of bearing capacity.
- Most building codes permit a 33 percent increase in the allowable bearing pressure for the load combinations involving wind or earthquake.

NET CONTACT PRESSURE

- The pressure at the contact surface of the foundation with the soil is obtained by dividing the total foundation load with the area of the footing. However, the load that directly acts over the foundation slab counteracts the contact pressure up to certain extent.
- This reduced pressure to be used for the structure design of the foundation components is called ***net contact pressure***, whose unfactored value is denoted by q_n and factored value is denoted by q_u.

Net contact pressure

= total foundation load per unit area of footing

– load directly acting on the foundation slab

- Unfactored loads are generally used for sizing of the foundations. Accordingly, unfactored load combinations are to be utilized for this purpose.

  Where, D = dead load, L = live load, L_r = roof live load, S = snow load, R = rain load, F = fluid load, H = weight and pressure of soil, W = wind load and E = earthquake load.

- Considering 33 percent increase in allowable bearing pressure in case wind or earthquake loads are included, the combinations for the determination of sizes of the footings may be written as follows:

  $S = (D + F) + (L + H) + (L_r \text{ or } S \text{ or } R)$

  $S = 0.75(D + F) + 0.75(L + H) + (0.75W \text{ or } 0.525E) + 0.75(L_r \text{ or } S \text{ or } R)$

  $S = 0.45(D + F) + (0.75W \text{ or } 0.525E)$

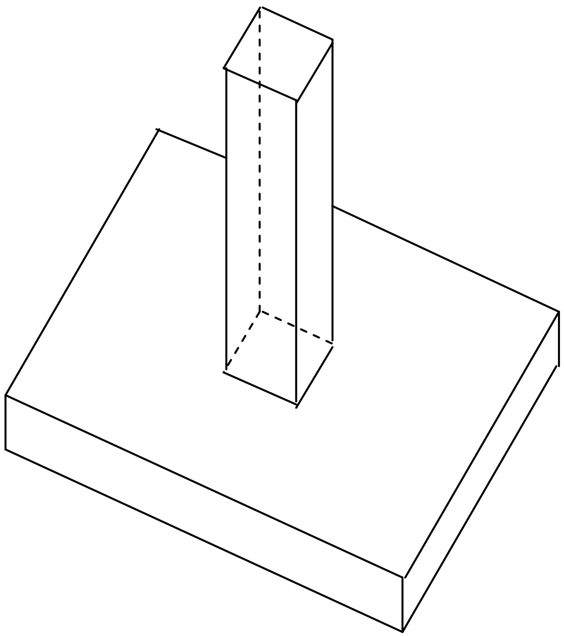
- For concentrically loaded footings, the required area is determined as follows:

$$A_{req} = \frac{S}{q_a}$$

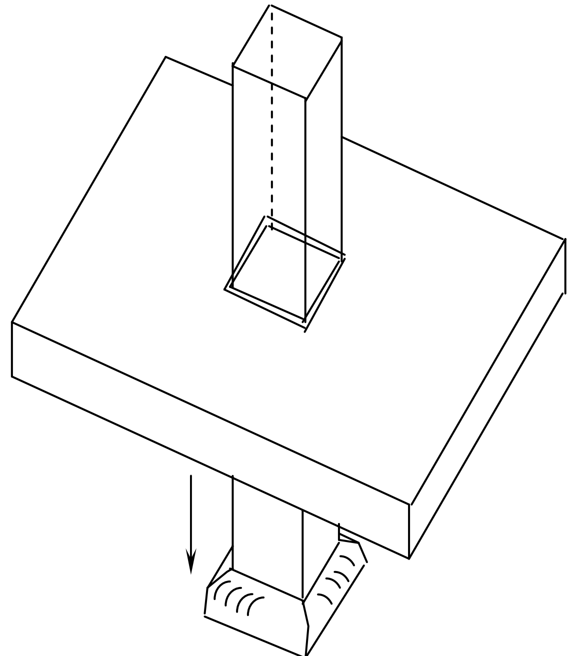
- After sizing of the foundation, the factored contact pressure is calculated by the usual factored load combinations and design is carried out according to these factored actions.
- To check the overturning of a foundation, only those parts of the loads causing overturning should be included and these must be balanced by dead loads multiplied with 0.9, maintaining a minimum factor of safety against overturning of 1.5.

FAILURE BEHAVIOR OF ISOLATED FOOTINGS

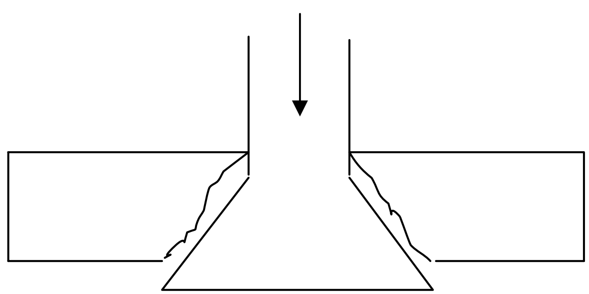
- A sufficiently thick isolated rectangular column footing may failure by flexural cracks parallel to each side close to the column or by one-way shear cracks also at the same location.
- However, the most common failure of such a column is by punching through the relatively thin slab.
- The failure occurs around the perimeter of the column by a special type of shear called the ***2-way*** or ***punching shear***, as shown in Fig. 9.3.



a) Before Failure

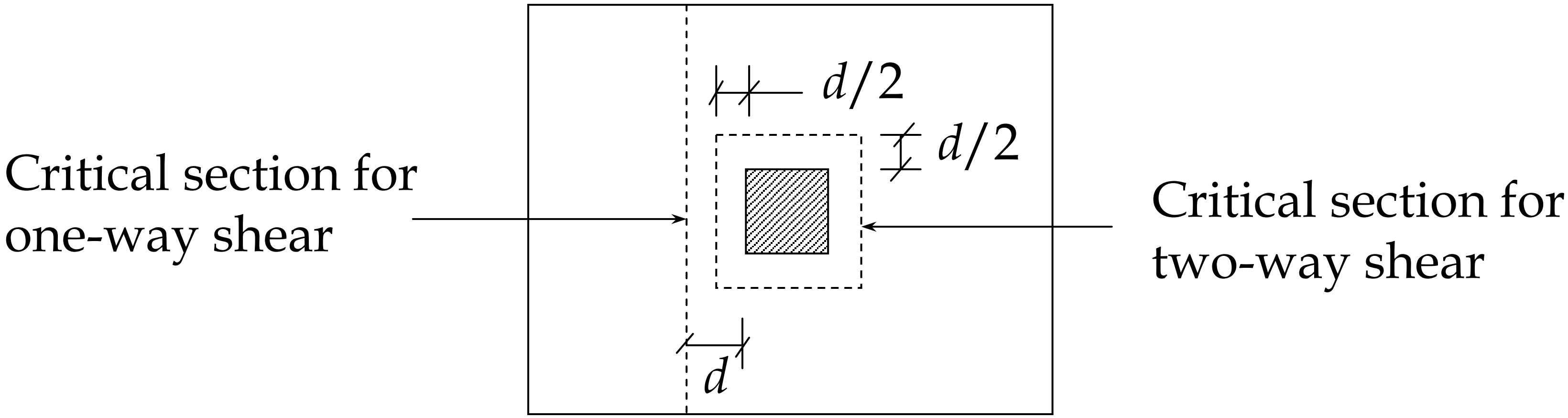


b) After Failure



c) Failure Mechanism in Sectional Elevation

* Due to the presence of simultaneous high vertical compressive stresses, lateral compressive stresses at the top of footing and lateral tensile stresses at the bottom of footing, the critical section for calculation of strength against this type of shear failure is to be considered at $d/2$ distance from the face of the column compared with d distance taken in case of one-way shear.
* The critical sections for one-way and two-way shears are shown in Fig. 9.4.


Critical section for one-way shear
$d/2$
$d/2$
d
Critical section for two-way shear

* It is not a common practice to provide shear reinforcement in the footings and hence these are usually designed by sufficiently increasing the depth of the footing such that the concrete shear strength alone is greater than the applied punching shear.
* In single footings, the total and effective depth (d) of the footings is mostly governed by shear and is decided in the start of the design process.

- The concrete strength against two-way shear is estimated as follows:

ϕV_c = lesser of the following three expressions

1. $0.75 \times 0.33\lambda\sqrt{f_c'}\,b_o d$

2. $0.75 \times 0.17\left(1+\dfrac{2}{\beta}\right)\lambda\sqrt{f_c'}\,b_o d$

3. $0.75 \times 0.083\left(\dfrac{\alpha_s d}{b_o}+2\right)\lambda\sqrt{f_c'}\,b_o d$

Where,

b_o = critical section perimeter, at a distance $d/2$ from the column perimeter

d = effective depth of the footing

β = ratio of the long to short sides of the column

α_s = a constant, 40 for interior loading, 30 for edge loading and 20 for corner loading of a footing.

and

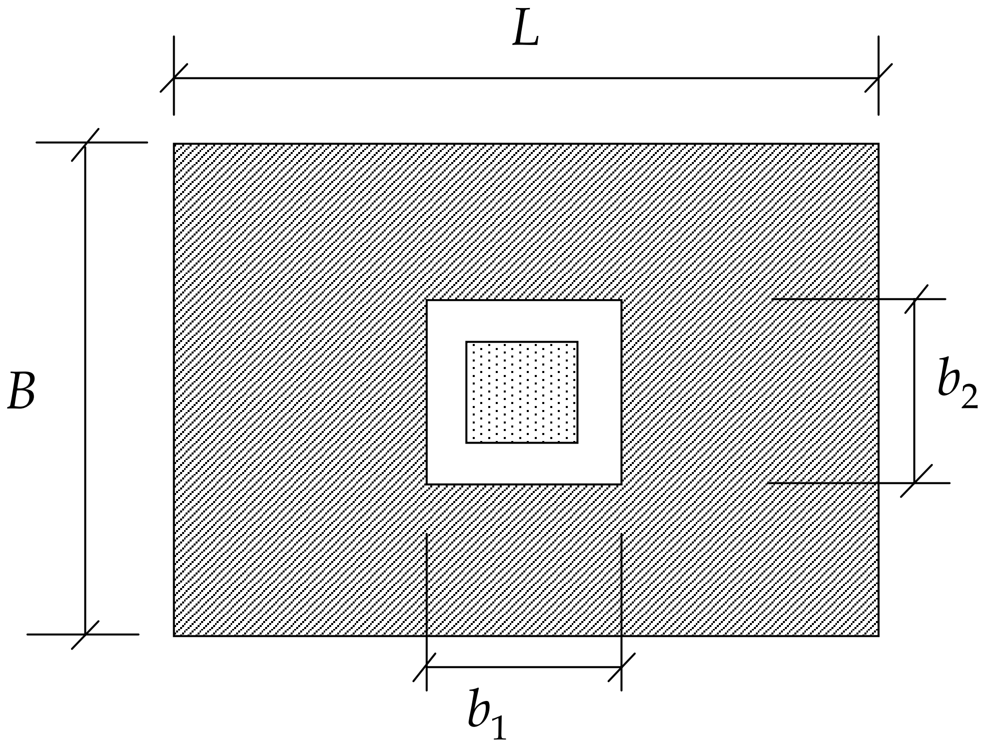
λ = 1.0 for normal weight concrete.

- According to ACI 15.7, ***the depth of footing above bottom reinforcement should not be less than 150mm for footings on soil and not less than 300mm for footings on piles***.
- The applied load causing punching shear is calculated by taking sum of all the contact pressure acting outside the critical perimeter of the column, as shown shaded in Fig. 9.5.

Punching shear, V_u

$= q_u \times$ area of footing outside critical perimeter

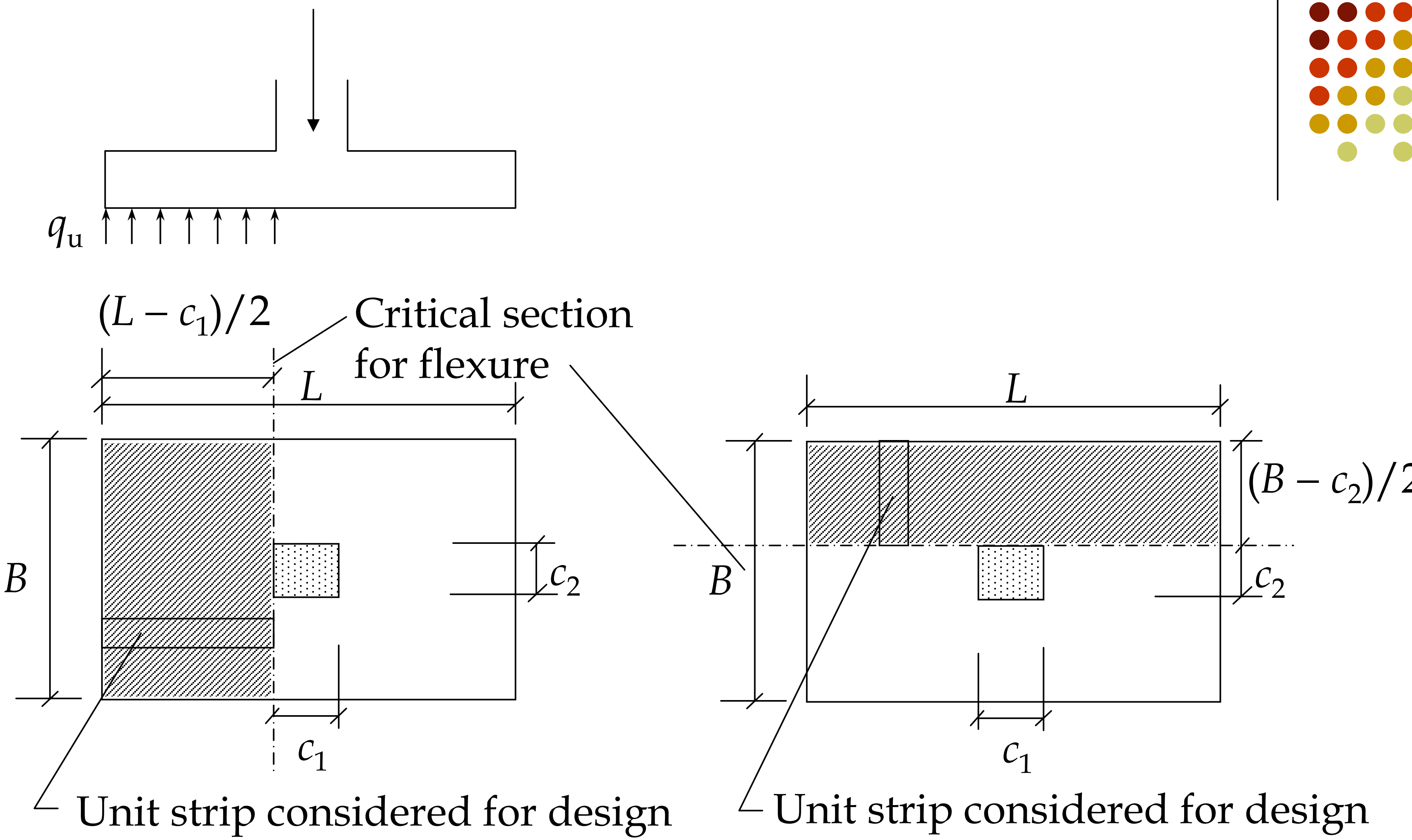
$= q_u \times (B \times L - b_1 \times b_2)$



Area Affective in Producing Punching Shear.

DESIGN BENDING MOMENT AND ONE-WAY SHEAR FOR ISOLATED FOOTINGS

* The critical section for calculating the bending moment is considered at the face of the column, pedestal or wall.
* When the footing supports a masonry wall, the critical section is taken midway between the center of the footing and the edge of wall.
* Referring to Fig. 9.6, it can be seen that a unit strip acts just like a cantilever beam subjected to a uniformly distributed load, q_u, equal to the net contact pressure.



a) Critical Section For Bending Along Longer Dimension

b) Critical Section For Bending Along Shorter Dimension

- Bending moment for longer steel
  $= q_u \times (L - c_1)2 / 8$
- Bending moment for shorter steel
  $= q_u \times (B - c_2)2 / 8$
- Maximum one-way shear for flexure
  $= q_u \times (L / 2 - c_1 / 2 - d)$

TRANSFER OF LOAD FROM COLUMN TO FOOTING

The concentrated load in the columns is transferred to the footing by direct bearing of the column over the footing (producing bearing stresses at the interface) and by forces in the dowels or column main steel bars that cross the joint.

- The loaded area or the area of cross-section of column is denoted by A_1, whereas, a larger area of footing may be considered effective in resisting the bearing stresses and is denoted by A_2.
- The area A_2 at a certain depth inside the footing is found by spreading the area A_1 at a slope of lesser of 2 horizontal to 1 vertical for solid footing and actually available steeper slope on each side.
- The concrete present in the larger area A_2 around the loaded area A_1 provides lateral confinement to the concrete and causes increase in the bearing strength.

The increase in the bearing strength is considered equal to $\sqrt{A_2 / A_1}$ with a maximum value of 2.0. The bearing strength (ϕP_n) is thus estimated by using the following expression with the value of ϕ equal to 0.65:

$$\phi P_n = \phi\, 0.85 f_c' A_1 \sqrt{A_2 / A_1} \leq \phi\, 0.85 f_c' A_1 \times 2.0$$

Where, f_c' is the cylinder strength of the footing concrete.

The above expression is used to check the concrete in the footing just below the column, without dowels having development length beyond this region.

Excess force to be resisted by dowels, having development length beyond the bottom of the footing $= P_u - \phi P_n$

- Area of steel required for dowels within the footing $= \dfrac{P_u - \phi P_n}{\phi f_y}$, where $\phi = 0.65$
- ACI minimum area of steel required for dowels $= 0.005\, A_g$, where, A_g = gross area of the supporting member.
- The above check and amount of dowels is required to make sure that that the footing concrete is capable of taking the full load of the column.

- However, if some of the column steel is discontinued at the top of footing, the portion of column just above the footing will be lacking fully developed bars.
- The bearing strength of this bottom portion of the column is to be checked. The continuing steel must provide a dowel steel area to develop a force equal to $P_u - \phi\, 0.85 f_c' A_g$.
- Area of steel required for dowels within the

column $= \dfrac{P_u - \phi 0.85 f_c' A_g}{\phi f_y}$, ϕ = 0.65

- ACI 15.8.2.1 requires that for cast-in-place columns and pedestals, area of reinforcement across interface must not be less than $0.005A_g$.
- Dowels must have diameter less than or equal to No. 35 and must extend into the supported member by a distance equal to larger of development length in compression of the column main steel bars and the splice length of the dowels.
- Similarly, these dowels must extend into the footing by a minimum distance equal to the development length of the dowels.
- ACI 15.8.2.3 permits the use of No. 44 and No. 57 bars in columns and their splicing with the lesser diameter dowels, if the bars are in compression.

DISTRIBUTION OF FOOTING STEEL

- The steel bars required along the longer side of the footing are uniformly distributed across the entire width of the footing.
- However, the reinforcement in the short direction is concentrated more in the central band of dimension equal to width of the footing.
- The ratio of reinforcement to be placed in the central band with respect to the total required reinforcement is taken equal to , $2/(\beta+1)$ where β is the ratio of long to short side of footing.
- The remainder of the reinforcement is to be uniformly distributed outside the central band of reinforcement.

PROCEDURE FOR DESIGN OF ISOLATED FOOTINGS (CONCENTRICALLY LOADED)

- Gather all input information, such as the allowable bearing capacity of soil, foundation depth, type of foundation, support reactions for applicable load combinations and soil refill, etc.
- Using service loads, calculate the required area of footing. Select the trial size and shape of the footing depending on the area and other site-specific requirements.
- Estimate the depth of footing based on punching shear requirements in an approximate way, which must not be lesser than 250mm according to the ACI code. The approximate depth may be found by using the following expression:

$$45\,\frac{P + M_x + M_y}{\alpha_s \sqrt{A_c f_c'}} + 60\,\text{mm} \geq 250\,\text{mm}$$

$d \;=\; H - 60$

Where A_c is the gross area of column section, moments are in meter units and α_s is already defined.

- Using the factored loads, estimated self-weight of the footing and the weight of the backfill, calculate the design contact pressure, q_u.
- Calculate the factored bending moment, one-way shears and two-way shear at the critical sections.

- Check the effective depth of footing for punching shear. Similarly, check the slab for one-way shear. One-way shear may be critical in case the footing is made slopping towards the ends.
- Calculate the amount of steel required in both the perpendicular directions. The minimum steel must be 0.002*bh* for Grade 300 and 0.0018*bh* for Grade 420 steel.
- In case of rectangular footing, determine the amount of steel required in the central band. Decide the diameter and spacing of bars satisfying the maximum spacing requirements.
- Check bearing strength of the footing and decide the required number and size of dowel bars.
- Check and satisfy all development length and lap splice requirements.

Example 9.1:

A 450 mm square interior column, reinforced with eight No. 25 bars of Grade 420, supports a dead load of 700 kN and a live load of 400 kN. Assume that a live load reduction of 30% may be allowed at the foundation level.

The foundation is to be placed at a depth of 1m where the allowable bearing capacity is 110 kPa.

The average density of the backfill material and the footing concrete together may be taken as 2100 kg/m^3, while the filling from ground to plinth level is to be 0.5m of material with average density of 1800 kg/m^3.

Design a square footing using f_c' = 20 MPa, f_y for foundation steel = 300 MPa and f_y for dowel steel = 420 MPa.

Solution:

- q_a = 110 kPa
- P_D = 700 kN
- P_L = 400 kN
- Live load reduction = 30 %
- f_c' = 20 MPa
- f_y = 300 MPa
- f_y for dowels = 420 MPa
- Depth of footing = 1.0 m
- Average density of fill and foundation = 2100 kg/m^3
- Density of surcharge = 1800 kg/m^3

- $P = P_D + P_L = 700 + (1 - 0.3) \times 400 = 980$ kN
- $q_{net} = 110 - (1.0 \times 0.5 \times 2100 + 0.5 \times 1800) \times 9.81/1000 = 90.87$ kPa
- $A_{f,\,req} = P / q_{net}$

  $= 980 / 90.87 = 10.79$ m^2

  (say 3.3 m $\times$ 3.3 m foundation)
- $P_u = 1.2PD + 1.6PL = 1.2 \times 700 + 1.6 \times 0.7 \times 400$

  $= 1288$ kN
- $q_u = P_u / A_f$

  $= 1288 / 3.32 = 118.3$ kPa

- Approximate depth of footing,

$$H = \sqrt{\frac{P + 1.5M_x + 1.5M_y}{0.01\alpha_s f_c'}} + 60 \text{ mm} \geq 250 \text{ mm}$$

$$= \sqrt{\frac{980 \times 1000}{0.01 \times 40 \times 20}} + 60 \geq 250 \text{ mm}$$

$$= 410 \text{ mm (say 450 mm)}$$

- $d = H - 60 = 390$ mm
- $b_o = 2 \times (c_1 + c_2 + 2d)$
  $= 2 \times (450 + 450 + 2 \times 390) = 3360$ mm
- $\beta = 450 / 450 = 1.0$

- ϕV_c = lesser of $0.75 \times 0.33 \lambda \sqrt{f_c'}\, b_o d$,

$$0.75 \times 0.17\left(1+\frac{2}{\beta}\right)\lambda \sqrt{f_c'}\, b_o d \quad \text{and}$$

$$0.75 \times 0.083\left(\frac{\alpha_s d}{b_o}+2\right)\lambda \sqrt{f_c'}\, b_o d$$

= lesser of the following:

1. $0.75 \times 0.33 \dfrac{\sqrt{20} \times 3360 \times 390}{1000}$ = 1450 kN
2. $0.75 \times 0.17\left(1+\dfrac{2}{1}\right)\dfrac{\sqrt{20} \times 3360 \times 390}{1000}$ = 2241 kN
3. $0.75 \times 0.083\left(\dfrac{40 \times 390}{3360}+2\right)\dfrac{\sqrt{20} \times 3360 \times 390}{1000}$ = 2423 kN

= 1450 kN

- V_u for punching shear

  $= q_u \times \{A_f - (c_1 + d)(c_2 + d)\}$

  $= 118.3 \times \{3.32 - (0.84)(0.84)\}$

  $= 1204.8 \text{ kN} \leq \phi V_c$ (***OK***)

- For 1 m wide strip:

  Cantilever length in short direction,

  $L_s = (B - c_2) / 2$

  $= (3.3 - 0.45) / 2 = 1.425 \text{ m}$

  Cantilever length in short direction,

  $L_l = (L - c_1) / 2$

  $= (3.3 - 0.45) / 2 = 1.425 \text{ m}$

- Bending moment for longer steel, M_{u1}

  $= q_u \times L_l^{\,2} / 2$

  $= \dfrac{118.3 \times 1.425^2}{2} \quad = 120.11$ kN-m

- Bending moment for shorter steel = same as above

- Maximum one-way shear for flexure

  $= q_u \times (L / 2 - c_1 / 2 - d)$

  $= 118.3 \times (3.3/2 - 0.45/2 - 0.39)$

  $= 122.44$ kN

- $\phi V_c = 0.75 \times 0.17\sqrt{f_c'}\ bd$

  $= 0.75 \times 0.17 \dfrac{\sqrt{20} \times 1000 \times 390}{1000}$

  $=$ 222.4 kN > applied shear (***OK***)
- $\dfrac{M_{u1}}{bd^2} = \dfrac{120.11 \times 10^6}{1000 \times 390^2} = 0.790 \Rightarrow \rho = 0.0030$
- $A_s = \rho \times bd = 0.0030 \times 1000 \times 390$

  $= 1170$ mm^2 / meter width
- $A_{s,\ min} = 0.002 \times bh = 0.002 \times 1000 \times 450$

  $= 900$ mm^2 / meter width
- Selected steel: #13 @ 110 mm c/c

  or #16 @ 170 mm c/c

- $A_1 \;=\; A_c \;=\; 202{,}500 \text{ mm}^2$
- $A_2 \;=\;$ lesser of 3300^2 (actual size at base) and $(c_1 + 4d) \times (c_2 + 4d)$

  = lesser of 10,890,000 and $(450 + 4{\times}390) \times (450 + 4{\times}390) \;=\; 4{,}040{,}100 \text{ mm}^2$

- $\sqrt{A_2 / A_1} \;=\; 4.47$ (consider maximum value of 2.0)

- $\phi P_n = \;\phi \times 2 \times 0.85\, f_c'\, A_1$

  $= \;0.65 \times 2 \times 0.85 \times 20 \times 202{,}500 / 1000$

  $= \;4475 \text{ kN} \;>\; P_u$ (***OK***)
- Foundation concrete is capable of resisting the column load even without the dowels.

- Now we check the dowels required to provide development length for the column steel.
- Area of steel required for dowels within the column

$$= \frac{P_u - \phi 0.85 f_c' A_g}{\phi f_y}$$

$$= \frac{1288 \times 1000 - 0.65 \times 0.85 \times 20 \times 450^2}{0.65 \times 420} = \text{zero}$$

- Minimum area of steel required for dowels

$= 0.005\ A_c$

$= 0.005 \times 450^2 = 1013\ mm^2$

- 4 – #19 bars are sufficient as dowel reinforcement. However, 8 – #19 bars may also be used for a conservative design.

- Development length in tension for #16 bars, f_y = 300 MPa, is:

  $l_d = 0.485 \dfrac{f_y}{\sqrt{f_c'}} d_b \geq 300\text{mm}$ for $d_b \leq$ No. 20 bottom bars

  $= 0.485 \dfrac{300}{\sqrt{20}} \times 16 = 521$ mm

- Development length in compression for #25 bars, f_y = 420 MPa, is:

  $l_{dc} = 0.24 \dfrac{f_y}{\sqrt{f_c'}} d_b \geq 200$ mm

  $= 0.24 \times \dfrac{420}{\sqrt{20}} \times 25 = 564$ mm

- Development length in compression for #19 bars, f_y = 420 MPa, is:

$$l_{dc} = 0.24 \frac{f_y}{\sqrt{f_c'}} d_b \geq 200 \text{ mm}$$

$$= 0.24 \times \frac{420}{\sqrt{20}} \times 19 = 429 \text{ mm}$$

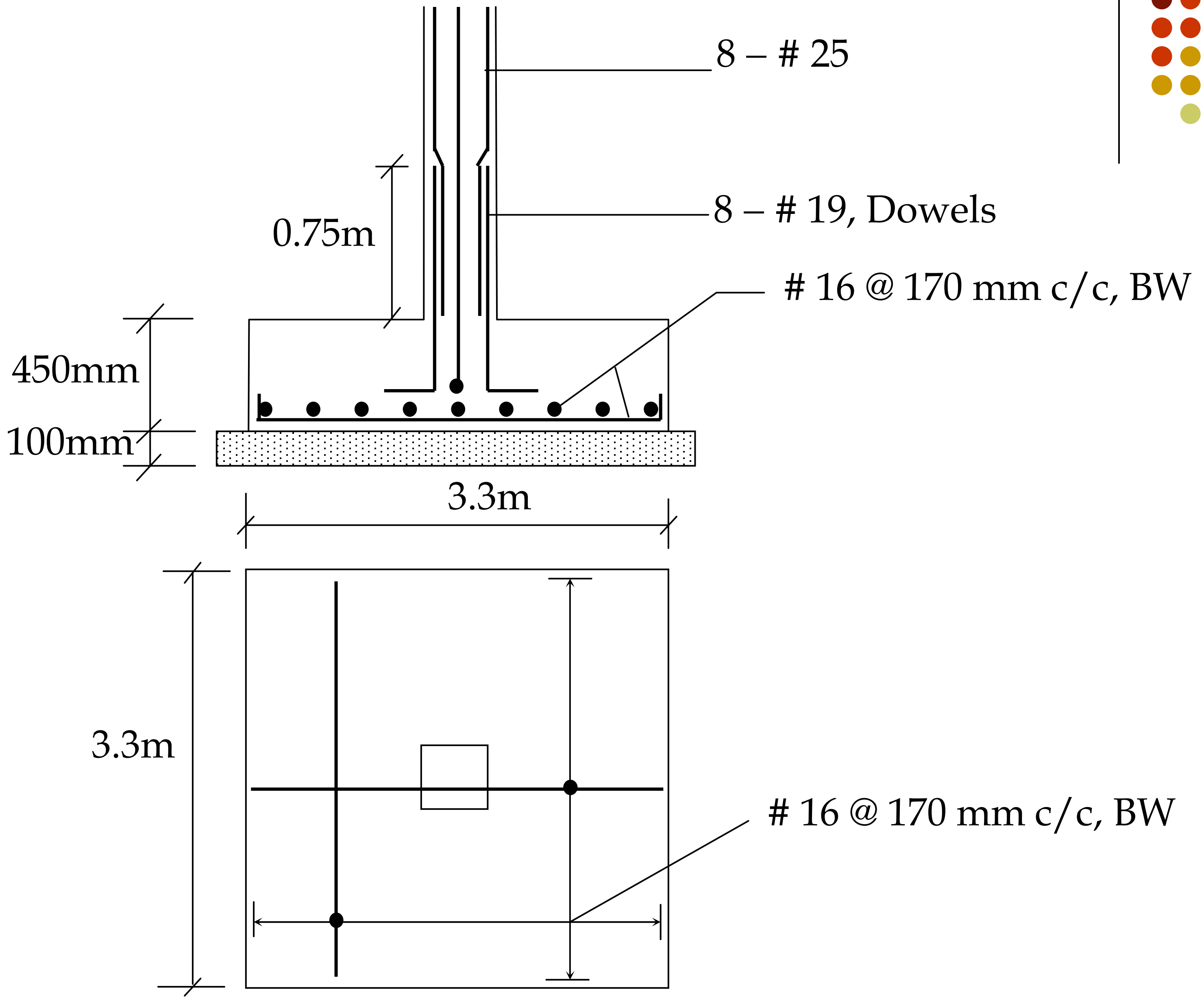
- Compression splice length for #19 bars

$$= 0.071\, f_y\, F\, d_b \geq 300\text{mm}$$

$$= 0.071 \times 420 \times 1.3 \times 19 = 737 \text{ mm}$$

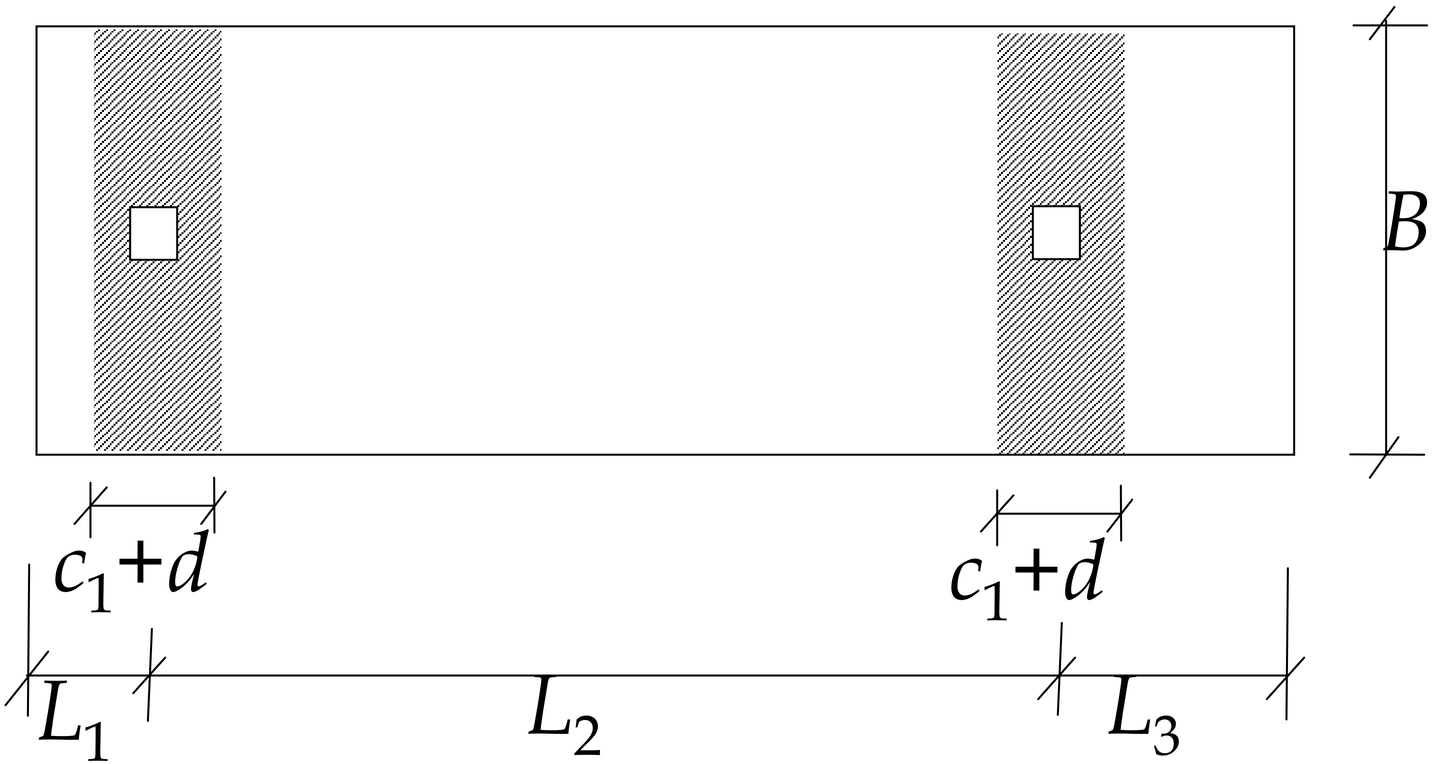
- The lap splice length for #25 column bars and #19 dowels is to be taken larger of the development length in compression for #25 bar (564 mm) and splice length for #19 bar (737 mm), that is, approximately 0.75 m.
- This lap will be provided above the footing level.
- The length of dowels into the footing must not be less than l_{dc} for #19 bars equal to 429 mm, which is less than the effective depth of the footing slab and the bars are turned and developed beyond this distance.

- The required development length for flexure of footing steel at the face of column is 0.521 m and the available length is (3.3 – 0.45) / 2 = 1.425 m.
- Hence the steel is fully developed at the critical section.
- The footing steel may not be curtailed near the edges of the footing in this example because it is greater than the minimum steel only by a small margin.
- However, if the footing thickness is tapered towards the ends, half of the steel may also be curtailed at a well-determined location.


8 – # 25
8 – # 19, Dowels
16 @ 170 mm c/c, BW
0.75m
450mm
100mm
3.3m
3.3m
16 @ 170 mm c/c, BW

COMBINED FOOTING DESIGN FOR TWO COLUMNS

- The combined footing resting on two columns behaves as a longitudinal beam along the length of the footing, resting on columns assumed to extend along the full width of the footing.
- The transverse strips shown in Fig. 9.17 expand the support effect over the full width of the footing.
- The width of this strip may be considered equal to column width in the same direction plus lesser of d / 2 distance and the available size on both sides of the column.
- The salient features of design of this type of footing are discussed below:


B
c1+d
c1+d
L1
L2
L3

- The centroid of the combined footing should coincide with the location of the resultant load on the footing. This makes the resultant moment of the footing transferred to soil equal to zero, which produces uniform pressure under the footing and avoids tilting of the foundation.
- Depending upon the clearances available, the footing is to be extended beyond the edge of the column near the boundary. This makes the footing safer against punching shear and helps in economizing the depth of footing.

- The distance between the shorter edge of the footing towards the column closer to the boundary and the resultant of the load is evaluated. The footing is longitudinally extended on the other side of the resultant of the load by the same distance. This fixes the length of the footing.
- The required area of the footing is calculated by dividing the total service load on the footing with the net allowable bearing capacity. This area is then divided by the decided length of the footing to establish the width of the footing.
- It is perhaps preferable to first decide the slab depth for one-way shear.

- The depth is then checked for punching of both the columns.
- Pattern loading for the live load is not considered for any footing design even if live load is included in the total load. Reasons are that the live loads are usually a smaller part of the total load for concrete structures and secondly the effect of patterns loads for a multi-story building dies out at the foundation level.
- A longitudinal section of the footing is then considered, with columns acting as supports over the full width of the footing, to determine the moments for design of longitudinal steel and one-way shears.

- In case the columns are widely spaced and there are sufficiently larger cantilever lengths at the two ends, the moments near the columns produce tension at the bottom fibers, requiring bottom steel, while the moments at the mid-span will produce tension on the upper side due to upward contact pressure, requiring top steel.
- In case the columns are widely spaced but the cantilever lengths at both sides are only equal to half the column dimensions, the moment everywhere will be producing tension on the upper side requiring top steel. For the situation when the columns are closely spaced but the cantilever lengths on both sides are excessive, the moment everywhere may produce tension on the lower side requiring bottom steel throughout.

- The width of the footing multiplied with the net contact pressure gives the load per unit length acting on the footing slab in the longitudinal direction acting like a beam.
- The shear force and bending moment diagrams may then be plotted to determine the forces at the critical sections.
- To plot these diagrams, the column loads may be considered as point loads acting at their centerlines if the spacing between the columns is sufficiently large.
- This will make insignificant difference from the actual results considering the column load to be uniformly distributed over their width.
- However, if the column spacing is lesser, we have to consider the column loads as uniformly distributed load acting over the width of the columns.

- It is found that a transverse width of the footing under the columns having a width equal to approximately $c + d$ also undergoes transverse bending. Hence, transverse bending moment is calculated for this strip just like an isolated footing and the corresponding transverse steel is decided. The load per unit length of the strip is calculated by dividing the total column factored load with the width of the footing.

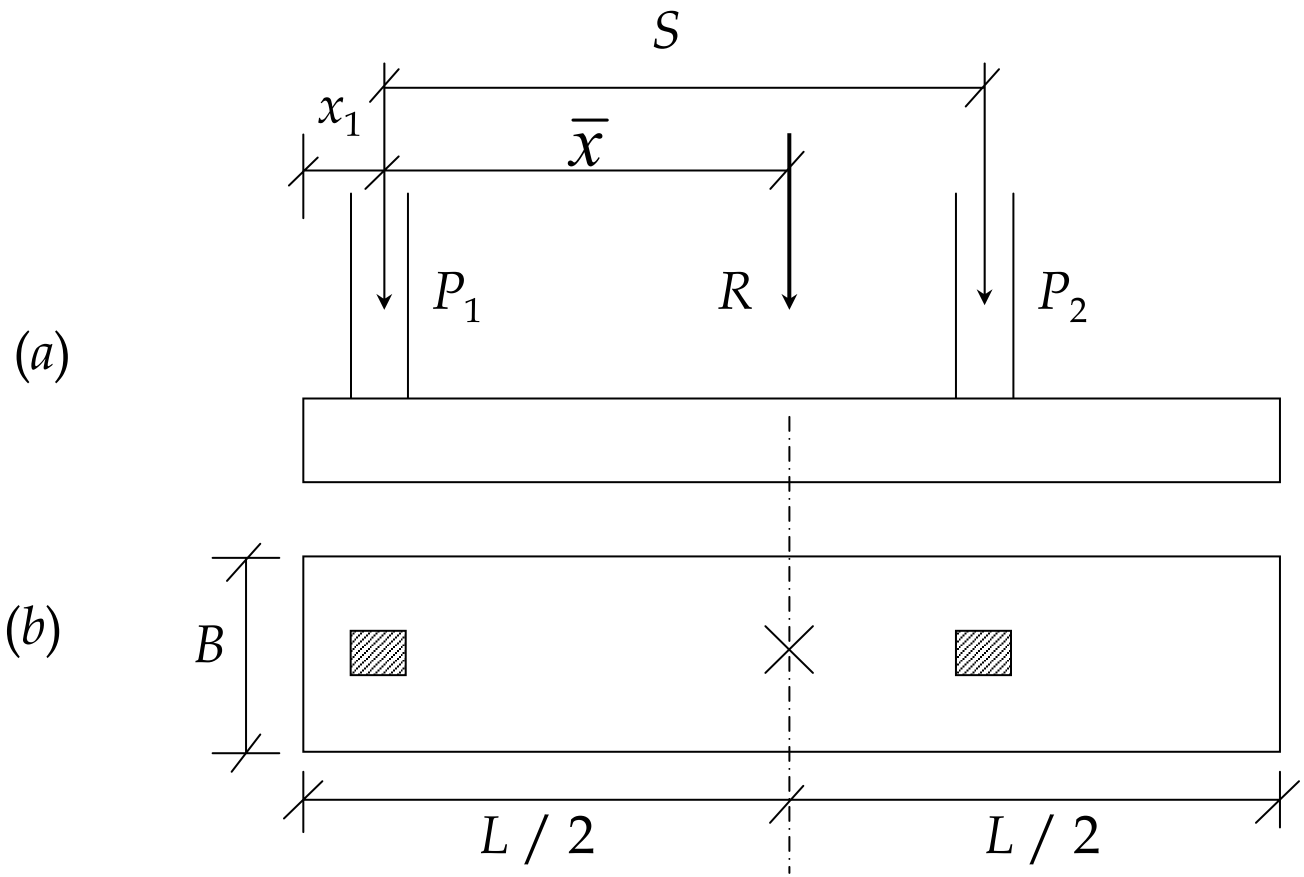
TYPICAL TYPES OF COMBINED FOOTINGS

- There are three commonly used types of the combined footings for two columns, as shown in Fig. 9.18.
- The first type (Fig. 9.18b) has a constant width and is the most popular type. The dimensions may be decided in the following sequence:

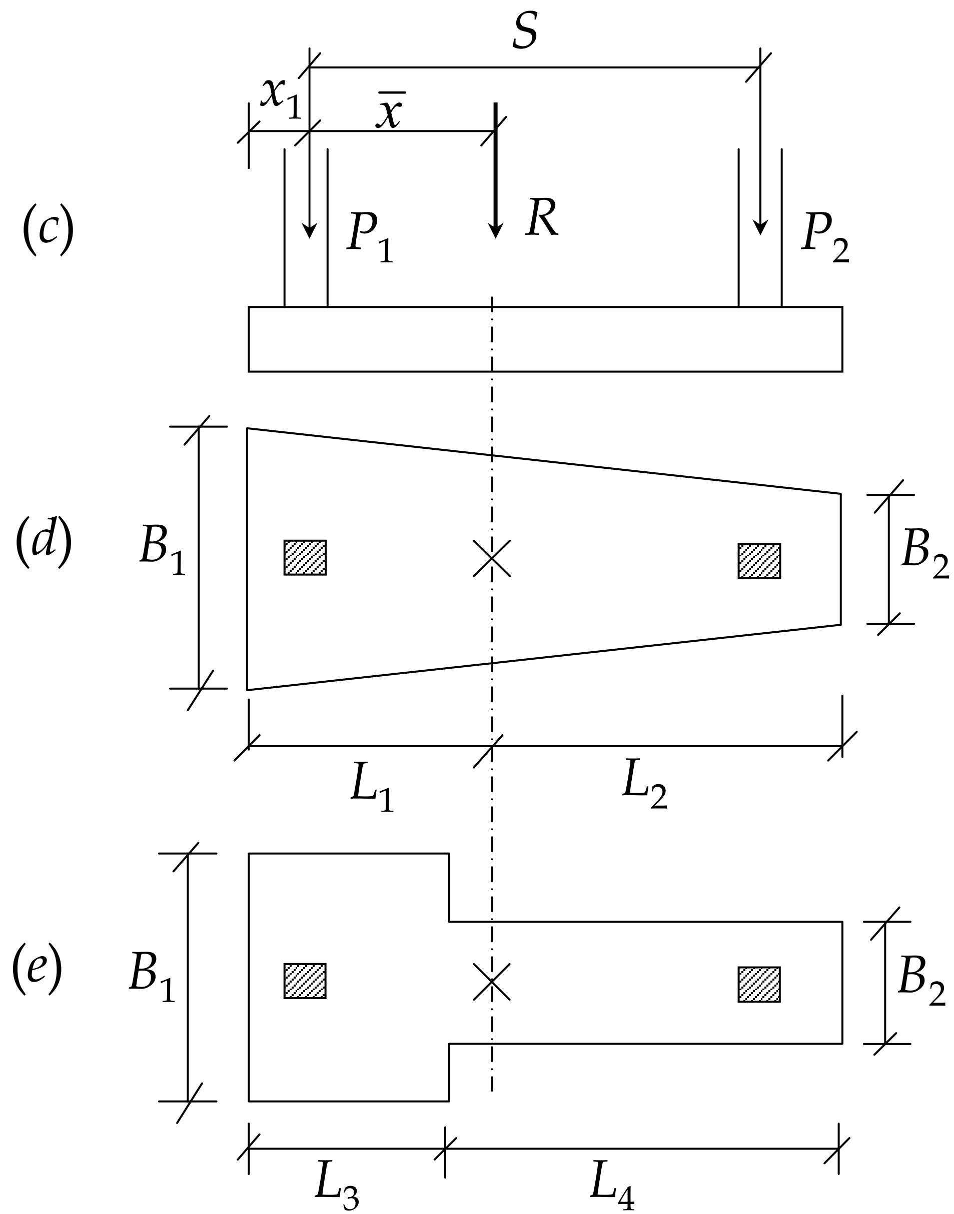
$$\bar{x} = \frac{P_2 \times S}{P_1 + P_2}$$

$x1$ = decided earlier

$L = 2\,(\bar{x} + x1)$ $\qquad B = \dfrac{R}{q_n L}$



S
x_1
$\bar{x}$
P_1
R
P_2
(a)
(b)
B
L / 2
L / 2



S
x_1
$\bar{x}$
(c)
P_1
R
P_2
(d)
B_1
B_2
L_1
L_2
(e)
B_1
B_2
L_3
L_4

Example 9.4:

An exterior column of size 450 mm × 300 mm having a dead load of 700 kN and a live load of 400 kN and an interior column of size 450 mm × 450 mm having a dead load of 1000 kN and a live load of 650 kN are 5.0m apart center-to-center. Design a rectangular combined footing only (not the dowels or splice) carried to a depth of 1.5m, where the gross allowable bearing capacity is 165 kPa. The outer edge of the exterior column is the property line. The average density of the backfill material and the footing concrete together may be taken as 2100 kg/m^3, while the filling from ground to plinth level is to be 0.75m of material with average density of 1900 kg/m^3. Design the footing using $f_c' = 20$ MPa and f_y for all steel = 300 MPa.

Solution:

- q_a = 165 kPa
- P_D for exterior column = 700 kN
- P_L for exterior column = 400 kN
- P_D for interior column = 1000 kN
- P_L for interior column = 650 kN
- S, spacing of columns = 5.0 m
- x_1 = 0.15 m
- Live load reduction = 20 %
- f_c' = 20 MPa
- f_y = 300 MPa
- Depth of footing = 1.5 m
- Density of fill = 2100 kg/m^3
- Density of surcharge = 1900 kg/m^3
- Depth of surcharge = 0.75 m

- P_1 for exterior column $= P_D + P_L$

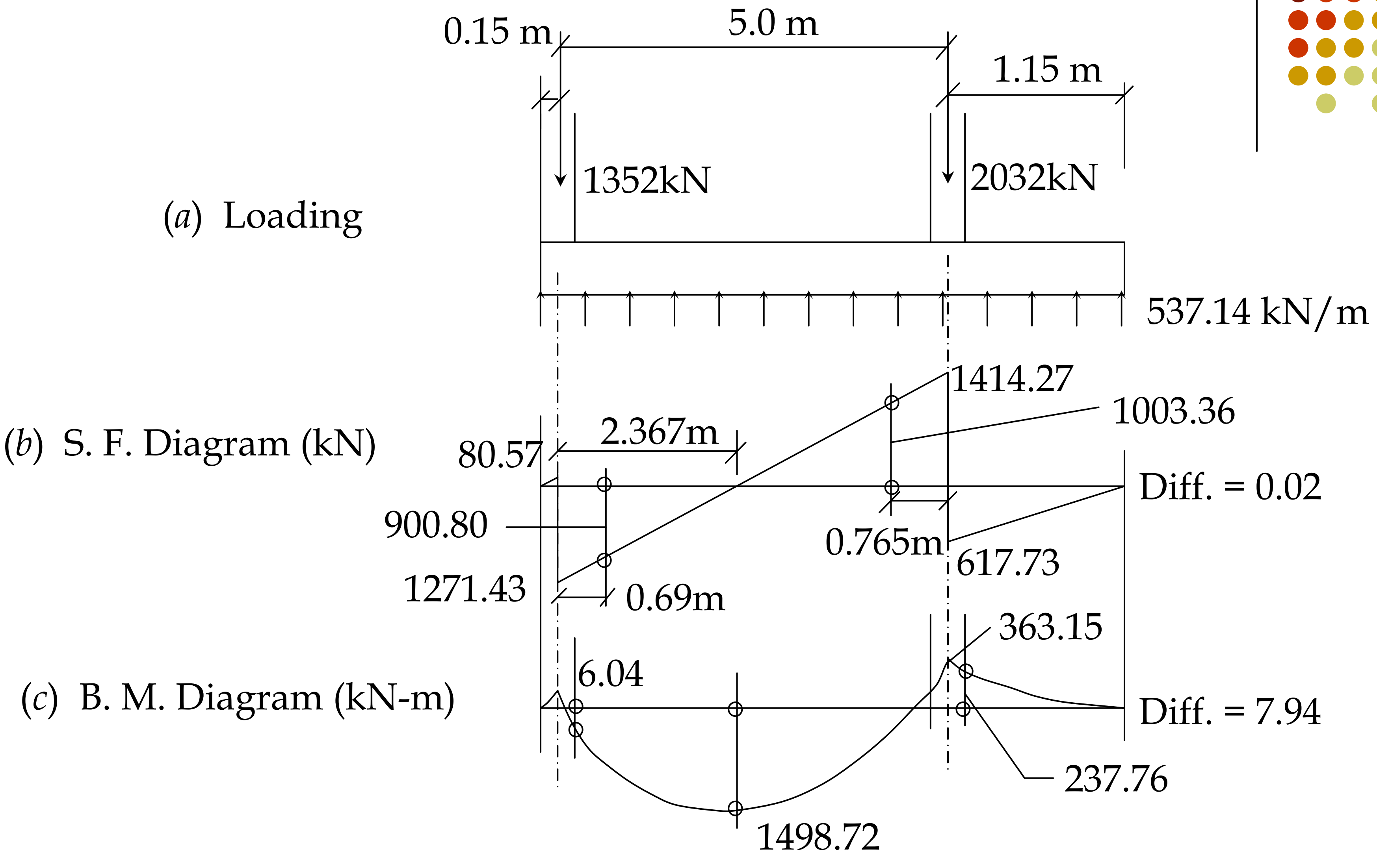
  $= 700 + 0.8 \times 400 = 1020$ kN
- P_2 for interior column $= P_D + P_L$

  $= 1000 + 0.8 \times 650 = 1520$ kN
- $q_{net} = 165 - (1.5 \times 0.5 \times 2100 + 0.75 \times 1900) \times 9.81/1000 = 135.57$ kPa
- $R = P_1 + P_2 = 1020 + 1520 = 2540$ kN
- $\overline{x} = \dfrac{P_2 \times S}{R} = \dfrac{1520 \times 5}{2540} = 2.992$ m
- $x_1 = 0.15$ m

- $L = 2(\bar{x} + x1) = 2(2.992 + 0.15)$
  $= 6.284$ m (***say 6.3 m***)
- $B = \dfrac{R}{q_n L} = \dfrac{2540}{135.57 \times 6.284} = 2.981$ m
  (***say 3.0 m***)
- $A_f = 6.3 \times 3.0 = 18.9$ m^2

Bending Moments And Shears Along Longer Direction

- The load along the length of the footing per unit length will be $q_u \times B = 179.05 \times 3.0 = 537.15$ kN/m.
- The resulting shear force and bending moment diagrams are shown in Fig. 9.19.


0.15 m
5.0 m
1.15 m
1352kN
2032kN
(a) Loading
537.14 kN/m
1414.27
1003.36
(b) S. F. Diagram (kN)
2.367m
80.57
Diff. = 0.02
900.80
0.765m
617.73
1271.43
0.69m
363.15
6.04
(c) B. M. Diagram (kN-m)
Diff. = 7.94
237.76
1498.72

- Trial depth of footing,

$$H = \sqrt{\frac{P + 1.5M_x + 1.5M_y}{0.01\alpha_s f_c'}} + 60 \text{ mm} \geq 250 \text{ mm}$$

- *H* for exterior column

$$= \sqrt{\frac{1020 \times 1000}{0.01 \times 30 \times 20}} + 60 \geq 250 \text{ mm} = 473 \text{ mm}$$

- *H* for interior column

$$= \sqrt{\frac{1520 \times 1000}{0.01 \times 40 \times 20}} + 60 \geq 250 \text{ mm} = 496 \text{ mm}$$

Depth For One-Way Shear

- The critical factored shear may be calculated by assuming *d* equal to 500 – 60 = 440mm.
- Max. one-way shear for flexure = 1057 kN
- $\phi V_c = 0.75 \times 0.17\sqrt{f_c'}\, bd = V_u$
- For steel in slab to be relatively lesser, it is advantageous to leave another margin of 20% in one-way shear.

$$0.8 \times 0.75 \times 0.17 \frac{\sqrt{20} \times 3000 \times d}{1000} = 1057$$

$\Rightarrow$ d = 773 mm

- Let H = 825 mm, d = $H - 60$ = 765 mm

Check For Punching Shear

- P_{u1} for exterior column = $1.2P_D + 1.6P_L$

  = $1.2 \times 700 + 1.6 \times 0.8 \times 400$ = 1352 kN

- P_{u2} for interior column = $1.2P_D + 1.6P_L$

  = $1.2 \times 1000 + 1.6 \times 0.8 \times 650$ = 2032 kN

- $q_u = (P_{u1} + P_{u2}) / A_f$
  $= (1352 + 2032) / 18.9 = 179.05$ kPa

For Exterior Column:

- $b_o = 2 \times (c_1 + d / 2) + (c_2 + d)$
  $= 2 \times (300 + 765 / 2) + (450 + 765)$
  $= 2580$ mm
- $\beta = 450 / 300 = 1.5$

- ϕv_c = lesser of $0.75\times, 0.33\sqrt{f_c'}$

$0.75\times 0.17\left(1+\frac{2}{\beta}\right)\sqrt{f_c'}$ and $0.75\times 0.083\left(\frac{\alpha_s d}{b_o}+2\right)\sqrt{f_c'}$

- ϕv_c = lesser of the following:

1. $0.75\times\ 0.33\sqrt{20}$ = 1.107 MPa
2. $0.75\times 0.17\left(1+\frac{2}{1.5}\right)\sqrt{20}$ = 1.330 MPa
3. $0.75\times 0.083\left(\frac{30\times 765}{2580}+2\right)\sqrt{20}$ = 3.033 MPa

= 1.107 MPa

- $\phi V_c = \phi v_c \, b_o d$

  $= 1.107 \times 2580 \times 765 / 1000 = 2184.9 \text{ kN}$

- $V_u = \text{Column load} - q_u \times \{ (c_1 + d / 2) (c_2 + d)\}$

  $= 1352 - 179.05 \times (0.6825 \times 1.215)$

  $= 1204 \text{ kN}$

- $V_u < \phi V_c$ (OK)

For Interior Column:

$b_o = 2 \times (c_1 + c_2 + 2d)$

$= 2 \times (450 + 450 + 2 \times 765) = 4860 \text{ mm}$

$\beta = 450 / 450 = 1.0$

- $\phi v_c =$ lesser of the following:

1. $0.75 \times 0.33\sqrt{20}$ = 1.107 MPa
2. $0.75 \times 0.17\left(1+\frac{2}{1.0}\right)\sqrt{20}$ = 1.710 MPa
3. $0.75 \times 0.083\left(\frac{40 \times 765}{4860}+2\right)\sqrt{20}$ = 2.310 MPa

= 1.107 MPa

- $\phi V_c = \phi v_c\, b_o d$

$= 1.107 \times 4860 \times 765 / 1000 = 4115.7$ kN

- $\phi V_c = \phi v_c\, b_o d$

  $= 1.107 \times 4860 \times 765 / 1000 = 4115.7$ kN
- V_u = Column load – $q_u \times \{ (c_1 + d)\,(c_2 + d)\}$

  $= 2032 - 179.05 \times (1.215 \times 1.215)$

  $= 1768$ kN
- $V_u < \phi V_c$ (OK)

Design Of Longer Top Steel

- Bending moment for longer steel,

  $M_{u1} = 1498.72$ kN-m / m

$$\frac{M_{u1}}{bd^2} = \frac{1498.72 \times 10^6}{3000 \times 765^2} = 0.854 \Rightarrow \rho = 0.0033$$

- $A_s = \rho \times bd = 0.0033 \times 3000 \times 765 = 7574 \text{ mm}^2$
- $A_{s,\,min} = 0.002 \times bh$

  $= 0.002 \times 3000 \times 825 = 4950 \text{ mm}^2$

Selected steel: 20 – #22 (spacing = 150 mm c/c)

- Curtailing 1/3rd of the bars may just be allowed because it gives area of steel just greater than the minimum. This option will not be utilized here due to less difference from the minimum steel ratio.

Design Of Longer Bottom Steel Under Interior Column

- Mu = 237.76 kN-m / m
- $\frac{M_u}{bd^2} = \frac{237.76 \times 10^6}{3000 \times 765^2}$ = 0.1354 $\Rightarrow$ $\rho = \rho_{\text{min}}$
- As = 0.002 × *bh*

  = 0.002 × 3000 × 825 = 4950 mm^2
- Selected steel:18 – #19 (spacing = 167 mm c/c)

- Development length in tension for #19 bars, f_y = 300 MPa, is:

  $l_d \;=\; 0.485 d_b \;\geq\; 300\text{mm} \text{ for } d_b \;\leq\; \text{No. } 20$

  $\;\;\;\;=\; 0.485 \times 19 \;=\; 618 \text{ mm}$ (***say 0.62m***)
- Available length on cantilever side

  $= \; 1150 - 450 / 2 \; = \; 925 \text{ mm} \; > \; l_d$ (OK)
- Distance of inflection point on inner side from face of the column plus larger of *d* and 12 d_b = 765 + larger of 765 and 12 $\times$ 19 = 1530 mm (say 1.6 m).

- ***Design Of Longer Bottom Steel Under Exterior Column***
- Moment is almost negligible; provide some steel only for additional safety. For example, the same steel as for the interior column may be provided.
- Selected steel: 18 – #19 (spacing = 167 mm c/c)
- Provide standard 90° hooks on the column side to get the full development length at the interior face of the column.

Design Of Transverse Beam Under Exterior Column

- Approximate width of beam $= c_1 + d / 2$

  $= 300 + 765 / 2 = 682$ mm
- Load on transverse beam $= P_{u1} / B$

  $= 1352 / 3.0 = 450.67$ kN/m
- Bending moment M_u = = 366.31 kN-m / m

  $$\frac{M_u}{bd^2} = \frac{366.31 \times 10^6}{682 \times 765^2} = 0.918 \qquad \Rightarrow \rho = 0.0036$$
- $A_s = \rho \times bd = 0.0036 \times 682 \times 765$

  $= 1878 \text{ mm}^2$

- $A_{s,\ min}$ = $0.002 \times bh$
  = $0.002 \times 682 \times 825$ = 1125 mm^2
- Selected steel: 5 – #22 (spacing = 136 mm c/c)

Design Of Transverse Beam Under Interior Column

- Approximate width of beam = $c_1 + d / 2$
  = 450 + 765 = 1215 mm
- Load on transverse beam = P_{u1} / B
  = 2032 / 3.0 = 677.33 kN/m
- Bending moment, M_u = $\dfrac{677.33 \times 1.275^2}{2}$

  = 550.55 kN-m / m

- $\dfrac{M_u}{bd^2} = \dfrac{550.55 \times 10^6}{1215 \times 765^2} = 0.7743 \Rightarrow \rho = 0.0030$
- $A_s = \rho \times bd = 0.0030 \times 1215 \times 765$

  $= 2789 \text{ mm}^2$
- $A_{s,\,min} = 0.002 \times bh = 0.002 \times 1215 \times 825$

  $= 2005 \text{ mm}^2$
- Selected steel: 8 – #22 (spacing = 152 mm c/c)
- ***Distribution Steel***
- $A_{s,\,min} = 0.002 \times bh = 0.002 \times 1000 \times 825$

  $= 1650 \text{ mm}^2 \text{/m}$
- Selected steel: #22 @ 230 mm c/c

- ***Check For Development Length***
- Development length in tension for #22 bottom bars,

  f_y = 300 MPa, is:

- $L_d \;=\; 0.606 \dfrac{f_y}{\sqrt{f_c'}}\, d_b \;\geq\; 300\text{mm}$ for d_b > No. 20

  $= \; 0.606 \dfrac{300}{\sqrt{20}} \times 22$

  $= \; 895$ mm < 1275mm

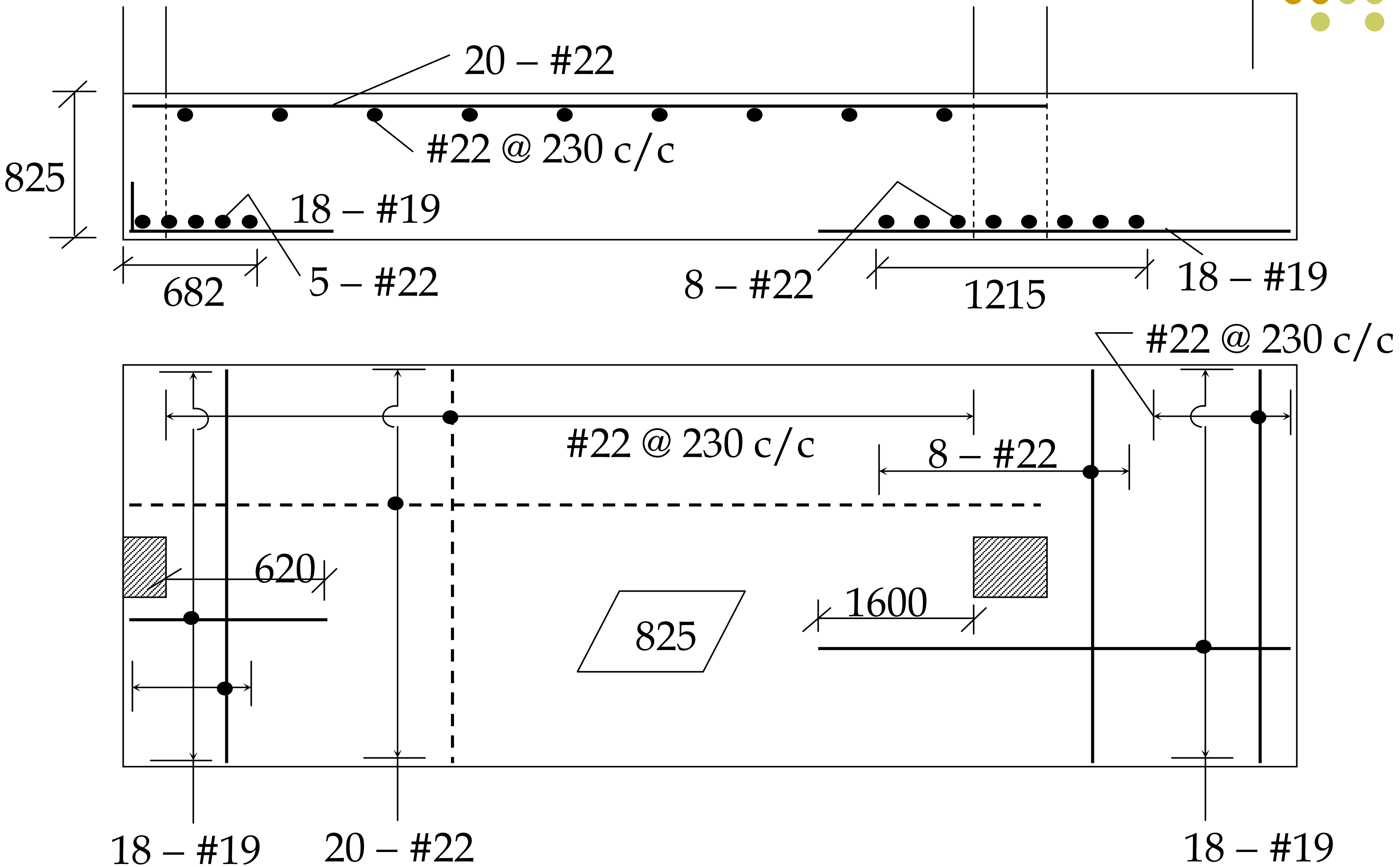
$\therefore$ The transverse steel is fully developed.

One Face Minimum Steel

- This steel is provided on all faces in both the directions where there is no other steel and the total depth of the footing is greater than 450mm.
- $A_{s,\,min} \quad = \; 0.001 \times bh \;\; = \; 0.002 \times 1000 \times 825$
  $\quad = \; 825 \text{ mm}^2 \text{ /m}$
- Selected steel: #13 @ 150 mm c/c

Sketch Of Reinforcement

- The reinforcement details for the main and the distribution steel are shown in Fig. 9.20.
- An extra steel equal to #13 @ 150 mm c/c is to be provided on all other faces in both the directions having proper overlap with the other reinforcement.


20 – #22
#22 @ 230 c/c
825
18 – #19
682
5 – #22
8 – #22
1215
18 – #19
#22 @ 230 c/c
#22 @ 230 c/c
8 – #22
620
825
1600
18 – #19
20 – #22
18 – #19

End of Topic