CIVL471 DESIGN OF RC STRUCTURES

LECTURE NOTE #12

CHAPTER V

FOOTINGS and FOUNDATIONS

Depending on the bearing capacity of the soil and the level of the load different types of footings are designed. In Fig.5.1 some examples are shown. In Fig.5.1a a wall footing, in Fig.5.1b a single column footing is shown. If two columns are very close to each other combined footing as shown in Fig.5.1c is preferred. Sometimes external columns of the building have to be in touch with the next building. A symmetrical footing is not possible. A non-symmetrical footing which has only one side may cause great difficulty in design. For this reason combined footings such as shown in Fig.5.1d are designed.

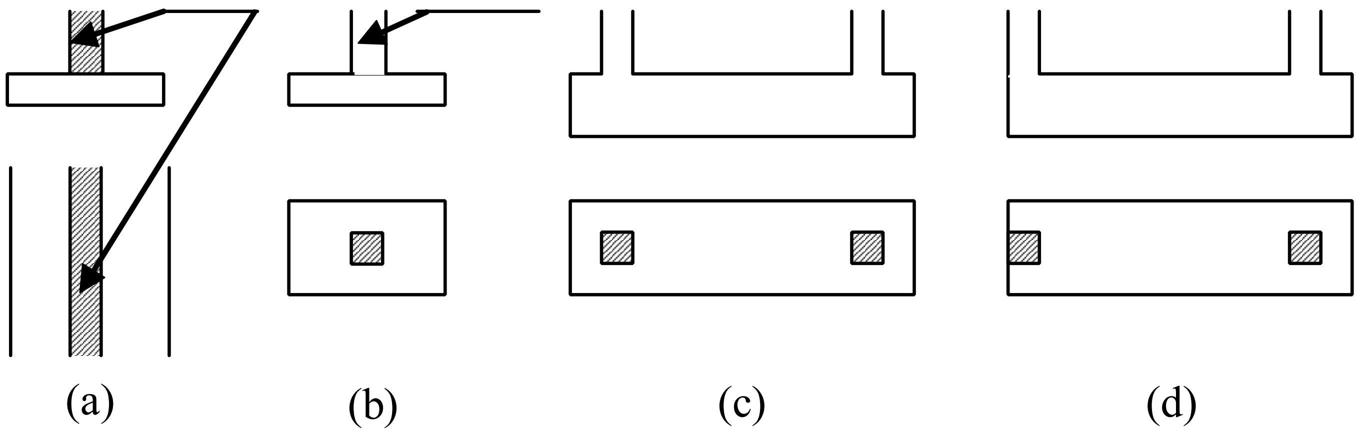
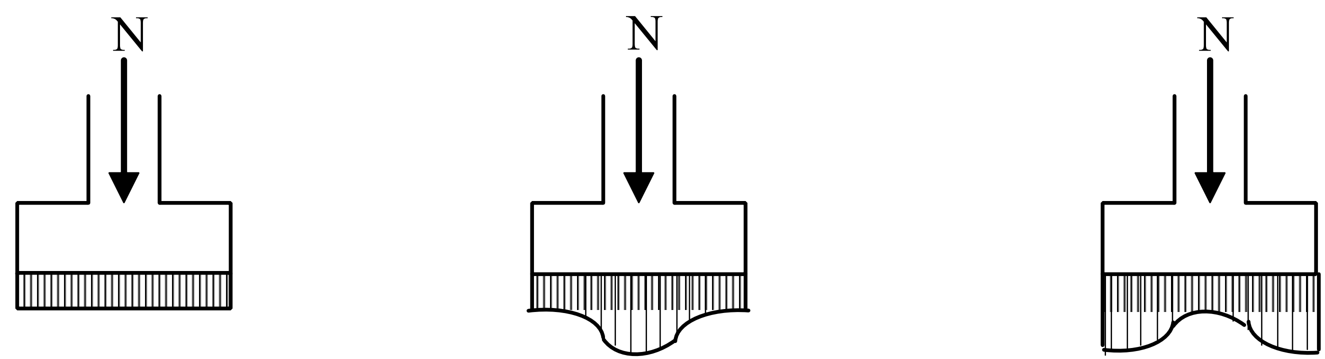


Figure 5.1

5.2 BASIC DESIGN PRINCIPALS AND ASSUMPTIONS

Soil pressure distribution below the footing changes as properties of the soil change. In Fig.5.2 soil pressures below an axially loaded symmetrical single column footing are shown. Although actual pressure distributions are not uniform, such an assumption is quite reasonable since soil pressure distributions are not known exactly and they do not influence the internal forces significantly.



a) Assumed   b) Actual (Granular soil)   c) Actual (Cohesive soil)

FIGURE 5.2

Similarly for the sake of simplicity pressure distribution is assumed linearly varying as shown in Fig.5.3 if the footing is subjected to eccentrically acting column load. Stress distributions should be determined by the loads at the

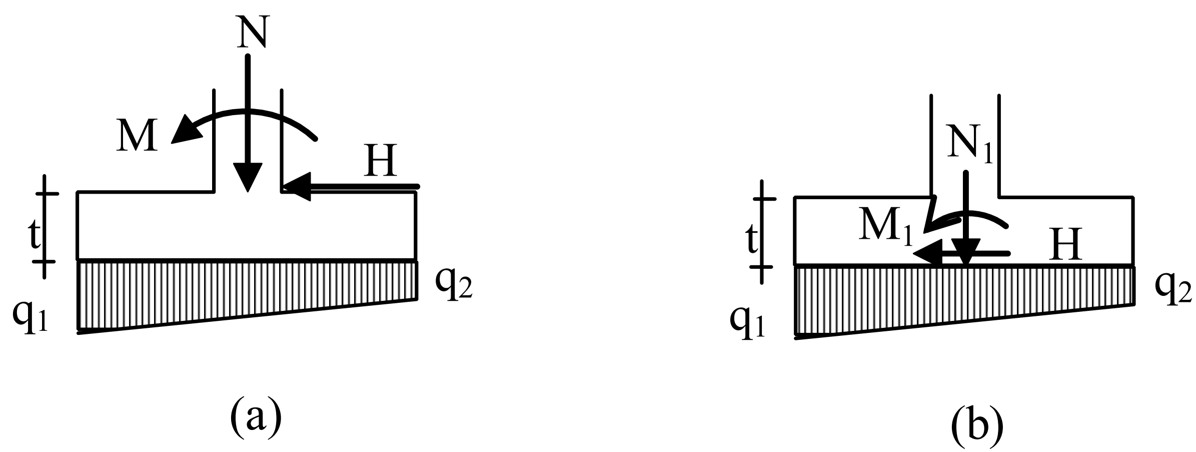


Figure 5.3

base of the footing as shown in Fig.5.3b. At the base N_1 = N + (the weigths of the footing and filling material over the footing) and M_1 = M + H*t. However in practice an alternative process is used in bearing area calculations. The forces at the bottom of the column (M and N) are used but the weigths of footing and fill material substracted from the allowable soil pressure. This reduced pressure is called "effective soil pressure".

On the highly compact soils eccentrical loading do not cause any problem. The end of the column can be assumed as fixed. On the other hand the footing and the end of the column may rotate on the compressable soils. Therefore if the soil is compressable either the end of the column should be made as hinged or less than full fixity should be assumed at the end of the the colum. Alternativly, uniform pressure can be obtained by arrenging the footing such that the column load will pass trough the centroid of the bearing area of the footing. This arrengment is shown in Fig. 5.4.

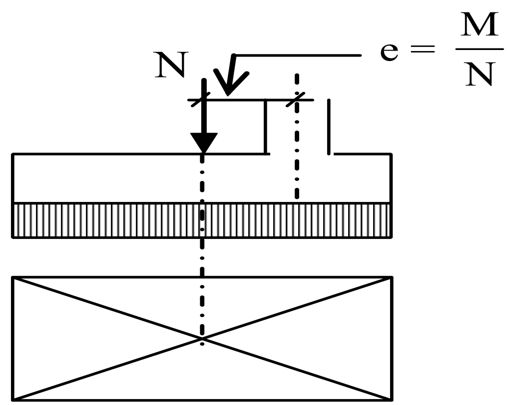


Figure 5.4

Bearing area calculations of the footing are based on the allowable soil pressure. Therefore at this stage of the design, service loads should be used. However, once dimensions of the bearing area are established design should be done by strength method using factored loads and corresponding soil stresses. According to the turkish regulations if the soil is not very weak (like loose sand, soft clay etc.) allowable soil pressure may be increased up to 50% for the load combinations where earthquake forces are included.

5.3 WALL FOOTINGS

Wall footings behave as cantilevers in two sides of the footing. (See Fig.5.5). However, according to TS500 wall footings should not be designed as

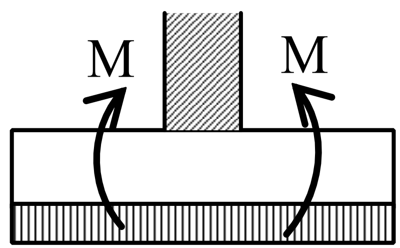


Figure 5.5

cantilevers. The heigth of the footing is selected such that any bending and shearing reinforcement will not be necessary. It means that concrete should not crack because of bending and maximum shear force should not be geater than $V_{cr} = 0.65 f_{ctd} bd$. Design shear force is calculated at the section 1-1 and design bending moment is calculated at section 2-2 (See Fig.5.6).

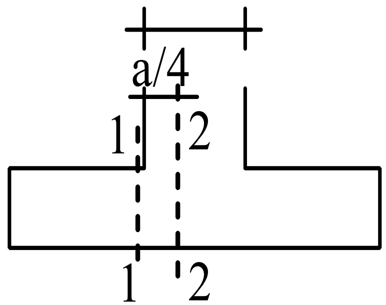


Figure 5.6

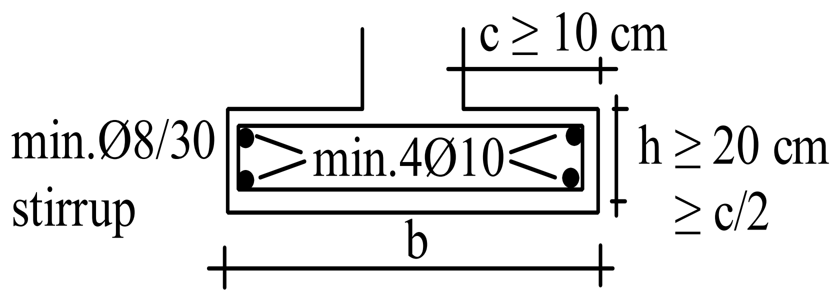


Figure 5.7

Section 2-2 is assumed as an uncracked homogeneous section. Calculated maximum tensile stress due to bending moment should be equal or less than $1.3f_{ctk}$ (stress causing cracking of concrete). All calculations mentioned above are carried out for one meter length of the footing. The minimum requirements for the wall footings designed in this way (including the reinforcement) are shown in Fig. 5.7.

Minimum clear concrete cover below the reinforcement should be 5 cm in all footings.

Example 5.1

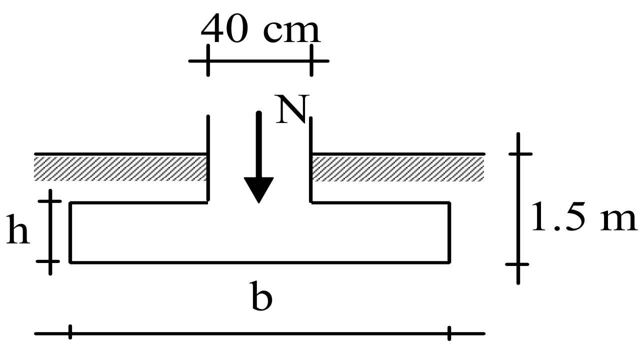


Figure 5.8

In the wall footing shown in Fig.5.8:
Dead load: $N_d = 210$ kN/m
Live load: $N_l = 150$ kN/m
Allowable soil pressure: $q_a = 300$ kN/m^2

Materials: C16 S220

Design this wall footing.

Solution:

Let us assume the thickness of the footing h = 50 cm. If the unite weight of the earth fill above the footing is assumed as 18 kN/m^3 (mixed sand and gravel), effective soil pressure can be calculated as follows:

$$q_{ef} = q_a - 1*1*0.5*25 - 1*1*(1.5-0.5)*18 = 300-12.5-18 = 269.5 \text{ kN/m}^2$$

Required footing area for one meter length:

$$A_{req} = \frac{N_d + N_l}{q_{ef}} = \frac{210+150}{269.5} = \frac{360}{269.5} = 1.34 \text{ m}^2 = 1*b_{min} = b_{min}$$

Selected b = 1.35 m. Check: c = (135 − 40) / 2 = 47.5 cm > 10 cm OK

Design soil stress q_u :

$$q_u = \frac{1.4N_d + 1.6N_l}{1*1.35} = \frac{1.4*210 + 1.6*150}{1.35} = \frac{360}{1.35} = 396\,\text{kN/m}^2$$

Shear force and bending moment:

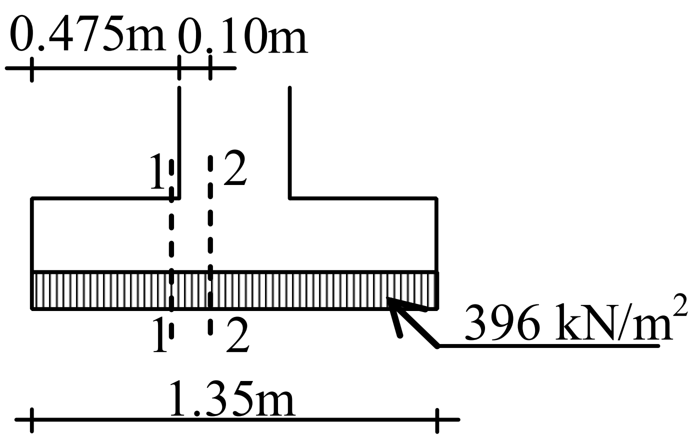


Figure 5.9

Design shear force:

$V_d = V_1 = 396*0.475 = 188.1$ kN

$V_{cr} = 0.65 f_{ctd} bd$

Assume: d' = 6 cm d = 50 − 6 = 44 cm

V_{cr} =(0.65*0.9*1000*440)/1000 =257.4 kN

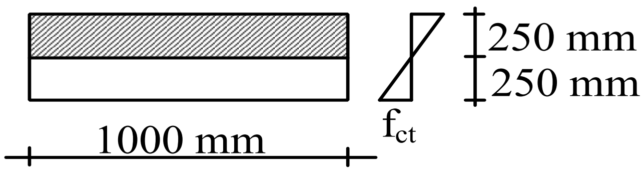
$V_d < V_{cr}$

It shows that the height of the footing is adequate for shear forces.

Design bending moment:

$$M_d = M_2 = 396*0.575^2/2 = 65.46 \text{ kN-m} = 65.46*10^6 \text{ N-mm}$$

Uncracked concrete section:

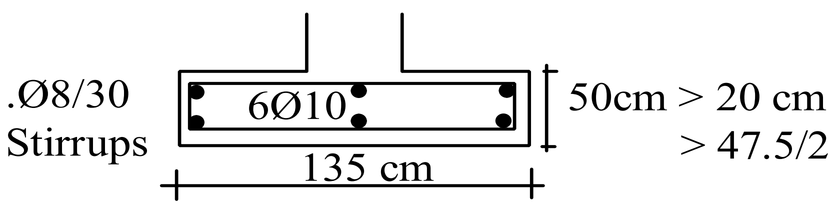


$$f_{ct} = \frac{M}{I}\frac{h}{2} = \frac{M}{\frac{bh^3}{12}}\frac{h}{2} = \frac{6M}{bh^2}$$

$$f_{ct} = \frac{6*65.46*10^6}{1000*500^2} = 1.57 \text{ N/mm}^2$$

$$1.3f_{ctk} = 1.3*1.4 = 1.82 \text{ N/mm}^2 > 1.57 \text{ N/mm}^2$$

The assumption of the cross-section being uncracked is also correct. Details of the footing are shown in Fig.5.10.



6Ø10 longitudinal bars are used since the footing is rather wide.

Figure 5.10