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Handout of:
Foundation design
Course and corrected application exercises

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For students of
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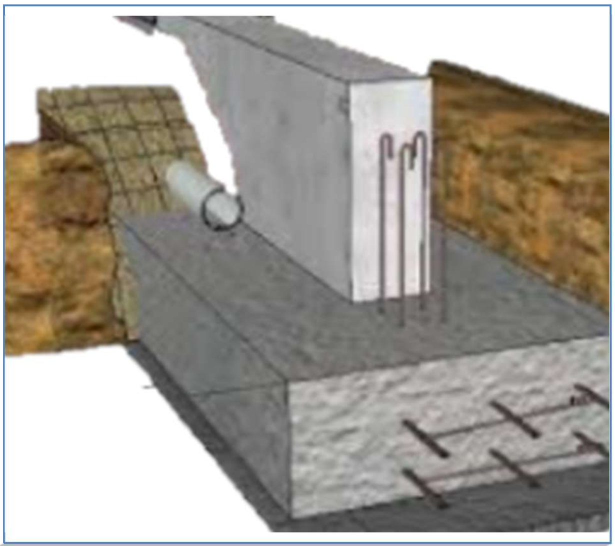

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Table of contents

List of figures 4
List of tables 6
Foreword 7
CHAPTER 1 8
1. Terminology 9
2. Algerian regulatory texts 10
3. Isolated footing 10
   3.1 Limit state justification of superficial foundations 10
   3.2 Connecting rod method 12
   3.3 Footing under the wall 13
   3.4 Footings under columns 14
      3.4.1 Column footings subjected to vertical loads 14
      3.4.2 Footings under columns subjected to vertical loads and bending moments 15
      3.4.3 Checking the footings under columns 15
      3.4.4 Reinforcement of footings under columns 15
   3.5 Circular footing under pillar 16
      3.5.1 Dimensioning circular footings 16
      3.5.2 Calculating reinforcement of circular footings 17
      3.5.3 Distribution of reinforcement 17
   3.6 Ribbed footing under pillar 18
      3.6.1 Reinforcement of ribbed footings 19
      3.6.2 Reinforcement of cross-ribbed footings 19
4. Strip footing 20
   4.1 The height of the strip footing 22
   4.2 Checks on the strip footing 23
5. Raft foundation 23
   5.1 Choice criteria 23
   5.2 Mode of operation 24
   5.3 Different types of raft foundation 25

5.4 Raft calculation methods 27

CHAPTER 2 31
1. Different types of deep foundations 32
2. Pile technology and installation 33
2.1 Piles installed with soil displacement 33
2.1.1 Prefabricated piles 33
2.1.2 Concrete piles in place 34
2.2 Piles installed without soil displacement 35
2.3 Wells 35
3. Pile behaviour 35
4. Bearing capacity of piles 36
5. Determining the bearing capacity of bored piles (using static formulae) 37
5.1 Principle of bearing capacity calculation 37
5.2 Bearing force due to the tip of the pile 37
5.3 Load-bearing capacity due to lateral friction 38
5.4 Safety coefficient 40
6. Bearing capacity of deep foundations (LCPC pressure-meter methods) : 40
7. Elastic pile settlement 43
CHAPTER 3 46
Exercise 1 47
Exercise 2 47
Exercise 3 51
Exercise 4 51
Exercise 5 54
Exercise 6 55
Exercise 7 58
Exercise 8 60
Exercise 9 69
Exercise 10 70
Conclusion 72
Bibliography 72

List of figures

Figure 1 : Foundation terminology 9
Figure 2 : Field of superficial foundations 9
Figure 3 : Linear stress distribution under footings 11
Figure 4 : Meyerhof model 12
Figure 5 : Connecting rod method 12
Figure 6 : Massive footings under walls 13
Figure 7 : Homothetic and equal overhang methods 14
Figure 8 : Reinforcement of the footing 15
Figure 9 : Circular footing 16
Figure 10 : Reinforcement of a circular footing 17
Figure 11 : Distribution of reinforcement in circular footings for $1m \leq D \leq 3m$ 17
Figure 12 : Distribution of reinforcement for circular footings for $D > 3m$ 18
Figure 13 : Distribution of reinforcement in circular footings arranged in hoops 18
Figure 14 : Ribbed footing under pillar 18
Figure 15 : Reinforcement of ribbed footings 19
Figure 16 : Cross-ribbed footings 19
Figure 17 : Reinforcement of cross-ribbed footings 20
Figure 18 : Diagram of a strip footing 20
Figure 19 : Stress diagram of a rigid footing for a uniform soil 21
Figure 20 : Stress diagram of a flexible footing for a uniform soil 21
Figure 21 : Stress diagram of a rigid footing for a non-uniform soil 21
Figure 22 : Stress diagram of a flexible footing for non-uniform ground with the flexible part between 2 columns. 22
Figure 23 : Stress diagram of a flexible footing for non-uniform ground with the flexible part under a column. 22
Figure 24 : Mechanical actions acting on the raft foundation 24
Figure 25 : Reinforcement of flat raft foundation to the right of the carrier elements 25
Figure 26 : The slab of the raft in the lower part 26
Figure 27 : The slab of the raft in the upper part 26
Figure 28 : Diagram of a diaphragm wall 32
Figure 29 : Barrette diagram 32
Figure 30 : Flowchart of how piles work 33

Figure 31 : Reinforced concrete piles 34
Figure 32 : Beaten-tube cast piles 34
Figure 33 : Global behaviour of a pile 36
Figure 34 : Negative pile friction 36
Figure 35 : Schematic diagram of how a pile works.. 38
Figure 36 : Soil reaction due to lateral friction 39
Figure 37 : Calculation diagram for the peak resistance of a pile.. 41
Figure 38 : Lateral friction curves.. 43
Figure 39 : Unit friction distribution along the pile 44
Figure 40 : Vertical cross-section of soil 47
Figure 41 : Foundation reinforcement plan.. 50
Figure 42 : Drawing of reinforcement 54
Figure 43 : Elevated section of the footing 58
Figure 44 : Position of the resultant force.. 58
Figure 45 : Plan section of a building 60
Figure 46 : The busiest panel 66
Figure 47 : Elevation and plan section of piles 70

List of tables

Table 1 : Distribution of paving loads. 24
Table 2 : Values of k_p sin (-φ) and k_p sin (- 2/3 φ) according to Caquot 39
Table 3 : Values of k_c 39
Table 4 : Values of the pressure bearing factor K_P 41
Table 5 : Conventional soil categories 42
Table 6 : Choice of lateral friction curve 43
Table 7 : Checking the no-lift condition 64
Table 8 : Compression check G + Q + E 64
Table 9 : Checking rollover stability 65
Table 10 : Summary of reinforcement chosen for ELU 67
Table 11 : Summary of reinforcement chosen for ELS 67
Table 12 : Calculated reinforcement of the edge at ELU 68
Table 13 : Calculated reinforcement of the edge at ELS 68
Table 14 : Reinforcement adopted for the edge 68
Table 15 : N values for penetration tests (SPT) 69

Foreword

The foundation is the part of a building or civil engineering structure that transmits loads (own weight, climatic forces, seismic forces and operating loads) to the ground. Foundations are very important elements in construction, as they form the structural part that ensures its load-bearing capacity and controls settlement due to the loads it applies to the ground and infiltration due to the possible presence of water in the soil.

Depending on the bearing capacity of the soil, the environment in which the structure is to be built and the admissible settlements, the builder can choose the type of foundation: shallow, semi-submerged or deep.

This handout is intended for students of master's degree in civil engineering. The course material and corrected exercises are divided into 3 chapters. The first chapter is devoted to shallow foundations, with a detailed discussion of insulated footings, threaded footings and inverts. The second chapter is devoted to deep foundations and finally a third chapter covers 10 application exercises with solutions relating to the first two chapters.

CHAPTER 1

Superficial foundations

1. Terminology

The function of a foundation is to transmit to the ground the loads (Figure 1) that result from the actions applied to the structure it supports; this presupposes that the designer knows:

- The load-bearing capacity of the footing. The soil must not break up or settle under the footing in an inconsiderate manner.
- The actions caused by the structure at foundation level. The footing must resist the actions to which it is subjected.

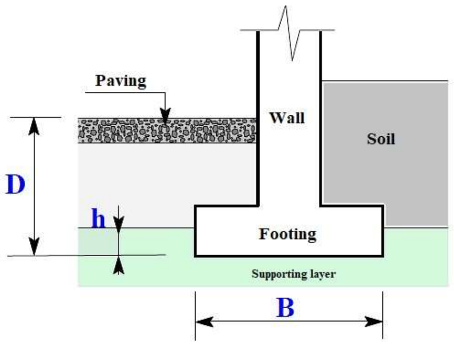


Figure 1 : Foundation terminology

With:

L : Length of the footing or the longest side of the footing.

B : Width of the footing or shortest side of the footing.

- Circular footing : $B = 2\ R$
- Square footing : $B = L$
- Rectangular footing : $B < L < 5\ B$
- Continuous or spinning footing : $L > 5\ B$

D : Height at which the base is embedded.

h : Base anchorage.

A shallow foundation is defined by the D/B ratio. Above a ratio of 6 (Figure 2), we are talking about deep foundations (the D.T.R - B.E.1.31 recommends that D/B < 1.5 for a shallow foundation).

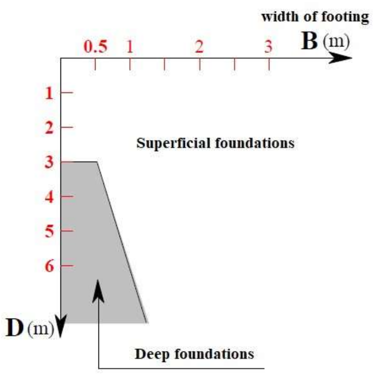


Figure 2 : Field of superficial foundations

2. Algerian regulatory texts

The texts used in Algeria for the design, calculation and verification of foundations are listed below:

- D.T.R. - B.C:2.2 : Permanent loads and operating loads (1989).
- D.T.R. - B.C.2.1 : General principles for verifying the safety of structures (1989).
- D.T.R. - B.E.2.1 : Rules for the execution of reinforced concrete structures (1991).
  D.T.R. - B.E.1.2 : Rules for the execution of earthworks for buildings (1991).
- D.T.R. - B.E.1.31 : Rules for the execution of surface foundations (1991).
- D.T.R. - B.C.2.33.1 : Rules for calculating surface foundations (1992).
- D.T.R. - B.C.2.31 : Provisional naming of soils and rocks (1993).
- D.T.R. - B.C.2.32 : Soil drilling and testing methods (1992).
- D.T.R. - B.E.2.31 : Deep foundation works (1994).
- D.T.R. - B.C.2.33.2 : Calculation methods for deep foundations (1994).
- D.T.R. - B.C.2.41 : Design and calculation rules for reinforced concrete structures "C.B.A 93". (1994).
- D.T.R. - B.E.11 : Soil sounding and testing work (1995).

3. Isolated footing

Dimensions are determined according to the following conditions:

- Limitation of soil stress and differential settlement under the footing.
- No punching
- Correct transmission of forces by oblique concrete compression rods.
- Good embedding of reinforcement.

3.1 Limit state justification of superficial foundations

- **Limit state of static stability**

The condition of no sliding of the foundation on the ground by ensuring that the inclination of the, resultant with respect to the normal to the plane of contact of the foundation with the ground remains within the sliding cone of half-angle at the apex such that:

$$tg\,\delta = \frac{H_u}{N_U} \leq 0.5$$

- **Ultimate limit state with regard to the ground and the footing**

If a linear pressure distribution under the footing is accepted, the stress diagram will take the form of a trapezium or a triangle (Figure 3), with no soil traction allowed. The diagram must satisfy the following conditions:

- Trapezoidal stress distribution : The stress at a quarter of the width of the footing must not exceed the value of the permissible stress q_a, i.e. we have:

$$q_a \geq \frac{(3\sigma_{max} + \sigma_{min})}{4}$$

$$\boldsymbol{\sigma} = \frac{N_u}{A.B} \mp \frac{M_u.y}{I_{ZG}}$$

- Triangular stress distribution: This is considered as the limiting case of the previous one, the minimum stress σ2 being equal to zero, the maximum stress σ1 is then equal to 1,33 q_a.

However, in the case of a normal force that is relatively small in relation to the bending moment, i.e., for an eccentricity greater than B/6, the triangular diagram is still admissible if the value of the stress at the point of the triangle remains limited to 1.33 q_a.

This condition is met if the vertical load Q satisfies the relationship :

$$Q \leq q_a(B - 2\,e)L$$

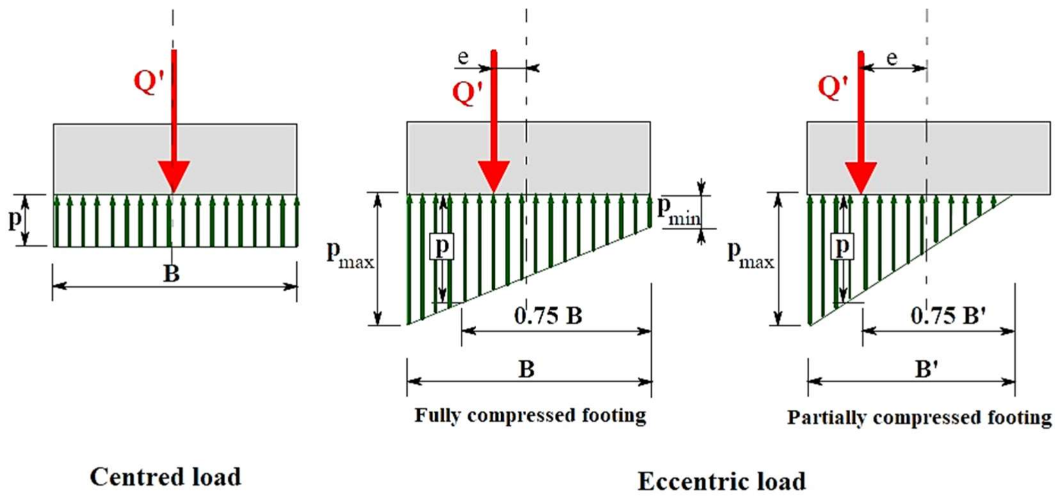


Figure 3 : Linear stress distribution under footings

If the footing is only partially compressed (Figure 4), B is replaced by B' = B - 2 e (Meyerhof model).

From the ultimate limit stress, we can deduce the design stress that will be used to justify the design of the footing. in general, $q = \frac{q_u}{2}$

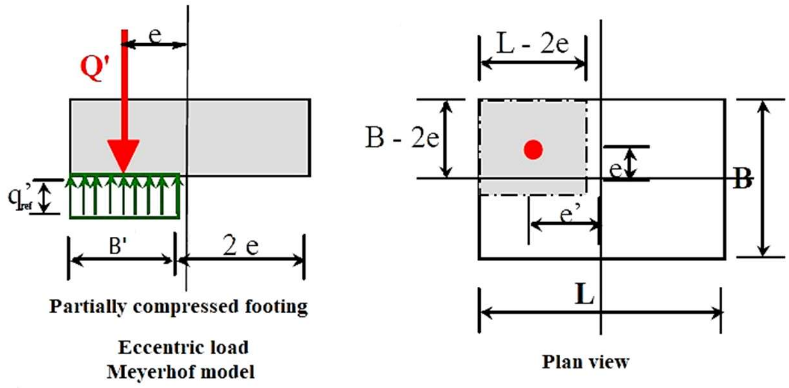


Figure 4 : Meyerhof model

3.2 Connecting rod method

The connecting rod method (Figure 5) requires the application of the following conditions :

- The connecting rod condition :

$$\frac{B-b}{4} \leq d \leq B-b$$

- The acting moment must remain less than the resisting moment :

$$\frac{p_u}{2}\frac{B-b}{4} \leq A_{st}\frac{F_e}{\gamma_s}d$$

- The reinforcement cross-section can be deduced from the previous inequality, equal to :

$$A_{st} \geq \frac{p_u\,(B-b)\,\gamma_s}{8\,F_e\,d}$$

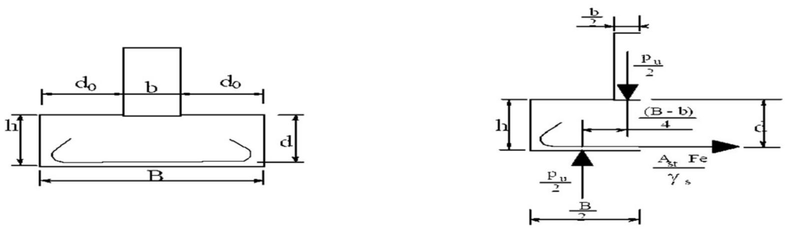


Figure 5 : Connecting rod method

- **Service limit state with respect to fissure opening**

In the absence of more precise justification under service conditions, the section of reinforcement calculated at the ELU of resistance must be increased by:

- 10% if the fissuring is prejudicial.
- 50% if the fissuring is very prejudicial.

- **Service limit state with respect to deformations**

The service limit state must only be justified in the following cases:

- Hyperstatic structures, taking into account the displacement or rotation of foundations.
- Differential settlement to be taken into account for foundations and structures.

- As specified by the project owner.

3.3 Footing under the wall

A footing under a wall is calculated for a linear length of 1.00 m. When the load carried by the wall is known, the width of the footing will be equal to :

$$a' \geq \frac{N_u}{\overline{\sigma}_s}$$

With :

N_u : Maximum normal force in MN.

$\overline{\sigma}_s$: Admissible soil stress in MPa.

a' : Width of the footing in m (a' ≥ 0.4 m).

Calculation of height :

- Massive footings (Figure 6) without transverse reinforcement if :

The wall transmits a vertical, uniform and centred load.

$h \geq a' - a$ or $h \geq 2d_0$

With :

a : Width of the wall.

a' : Width of the footing.

d_0 : Overhang of the footing.

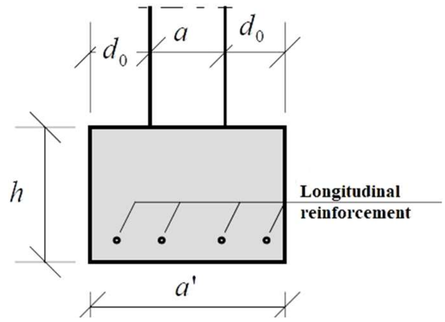


Figure 6 : Massive footings under walls

- Rigid footing :

A rigid footing is one that satisfies the following relationship :

$\frac{(a'-a)}{4} \leq d \leq a' - a$ or again $\frac{d_0}{2} \leq d \leq 2d_0$

With :

d : Useful height of the base.

h – d : Coating varying from 3 to 5 cm.

- Flexible footing :

A flexible footing is one that satisfies the following relationship :

$$d_0 \geq 2d$$

3.4 Footings under columns

3.4.1 Column footings subjected to vertical loads

The horizontal dimensions of the footings under the columns are represented by the symbols a' and b' with b' ≤ a'.

The surface area of the footing is calculated from this relationship :

$$S = a'\,b' \geq \frac{N_u}{\bar{\sigma}_s}$$

N.B : Under a round column, it is easier to provide a square footing than a circular footing, in which case a' = ∅ of the column.

If the column is rectangular, the footing can be calculated using the homothetic method or the equal overhang method (Figure 7).

For the homothetic overhang method, the relationship is $\frac{b}{a} = \frac{b'}{a'}$, whereas for the equal overhang method, it becomes $a' = a + 2x$ and $b' = b + 2x$

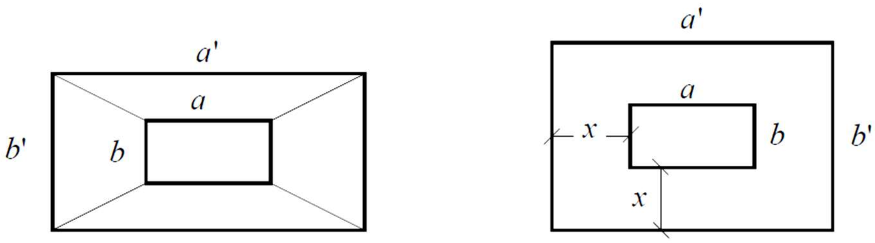


Figure 7 : Homothetic and equal overhang methods

The calculations are carried out using footings with homothetic overhangs (connecting rod method), but when the length of the column is greater than three times its width, the equal overhang method is chosen.

S, a and b are known, then a' and b' must be calculated for each case.

For a footing with a homothetic overhang, $a' = \frac{a}{b} b'$ and $b' \geq \sqrt{\frac{b}{a} \frac{N_u}{\bar{\sigma}_s}}$

For even-edged footings, $S = (a + 2x)(b + 2x) \geq \frac{N_u}{\bar{\sigma}_s}$, where x is the solution to this second-order equation and its root is positive.

The vertical dimensions of the footing are determined in the same way as for footings under friendly walls in both directions, i.e. :

$\frac{(a'-a)}{4} \leq d \leq a' - a$ and $\frac{(b'-b)}{4} \leq d \leq b' - b$ with $b' \leq a'$

For even-edged footings, $d \geq \frac{a'-a}{4}$ and $d = \frac{d_0}{2} = \frac{(a'-a)}{4} = \frac{(b'-b)}{4}$.

For footings with a homothetic overhang, $d = \max\left[\frac{a'-a}{4} ; \frac{b'-b}{4}\right]$. In general, h = d + 5 cm

3.4.2 Footings under columns subjected to vertical loads and bending moments

$$q_a \geq \frac{(3\,\sigma_{max} + \sigma_{min})}{4}$$

By using the rigidity condition if the footing is rigid (homothetic or with equal overhang), which amounts to solving an equation of order 3.

$$\sigma = \frac{N_u}{a'b'} + \frac{M_u.y}{I_{ZG}} = \frac{N_u}{a'b'} + \frac{M_u.b'/4}{\frac{ab'^3}{12}} \leq \bar{\sigma}_{sol}$$

3.4.3 Checking the footings under columns

The previous calculations led us to footing dimensions for which no account was taken of either the own weight of the footing or the own weight of the embankment soil. As these values are not negligible, the stresses need to be rechecked.

$$\sigma = \frac{N_{Ur}}{a'b'}\left[1 \pm \frac{6e}{a'}\right] \leq \frac{4}{3}\,\overline{\sigma}_{soil} \text{ or } \overline{\sigma}_{soil}$$

(Depending on the stress diagram and the type of load)

With :

$N_{Ur} = N_U + 1{,}35\,(G_0 + G_1).$

G_1 : Weight of the footing.

G_2 : Weight of embankment.

In the case of footings on soil giving rise to relatively high soil stresses ($\sigma_{sol} > 0.6$ MPa) under the effect of localised load s, the behaviour of the footing with respect to punching must be checked. This check is unnecessary if $h \geq \frac{a'' - a}{2}$

3.4.4 Reinforcement of footings under columns

The reinforcement of the footing (Figure 8) is calculated as follows : $A_s = \frac{P_u}{8}\,\frac{a'' - a}{d}\,\frac{1}{\frac{f_e}{\gamma_s}}$

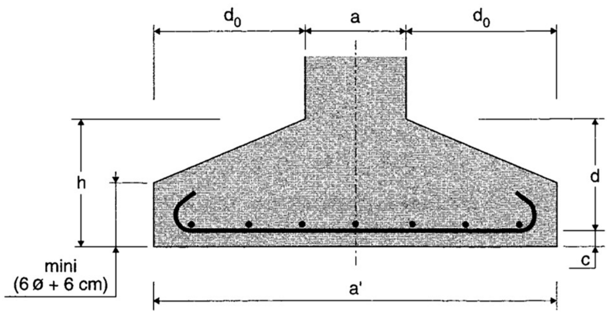


Figure 8 : Reinforcement of the footing

The calculation is made in both directions if the homothetic overhang method is applied, but the reinforcement is identical in both directions if the equal overhang method is applied.

For calculating distribution steels (spinning steels) :

$$A_r = \max\left(\frac{A_s}{4}, \text{Min cross section for chaining}\right)$$

The maximum value between 1/4 of the calculated cross-section and the minimum cross-section for a chain link should be taken, i.e. :

- 3.0 cm² for smooth rounds Fe E215.
- 2.0 cm² for bars Fe E400.
- 1.6 cm² for welded mesh or bars Fe E500.

The spacing between the reinforcing bars must not exceed 30 cm.

The reinforcement is terminated by standardised hooks, and three cases are distinguished depending on the values of the embedment length ls and the width of the footing a' :

- $\mathbf{l_s} > \frac{\mathbf{a'}}{\mathbf{4}}$: Anchoring with standardised hooks.
- $\frac{\mathbf{a'}}{\mathbf{8}} \leq \mathbf{l_s} \leq \frac{\mathbf{a'}}{\mathbf{4}}$: Straight anchoring along the entire length of the footing.
- $\mathbf{l_s} < \frac{\mathbf{a'}}{\mathbf{8}}$: There are two possibilities: Either straight anchoring of every second bar across the entire width of the footing, with the next bar 0.71·a' long, or straight anchoring of all the bars 0.86.a' long, extended alternately to the edge of the footing on one side and the other.

In addition, we must not forget the start of the wall reinforcement, which must be lowered to at least the bottom layer of the footing.

3.5 Circular footing under pillar

3.5.1 Dimensioning circular footings

The minimum diameter D (Figure 9) of the underside is given by the bearing capacity of the soil, i.e. : $\sigma = \frac{Q}{S} \leq \overline{\sigma_{soil}} \rightarrow \frac{Q}{\pi . D^2/4} \leq \overline{\sigma_{soil}}$, So $D^2 \geq \frac{4.Q}{\pi . \overline{\sigma_{soil}}}$

To use the connecting rod method, you need to have :

$$(D - d) \geq h \geq \frac{(D-d)}{4} \qquad \text{With} \quad h_t = h + e$$

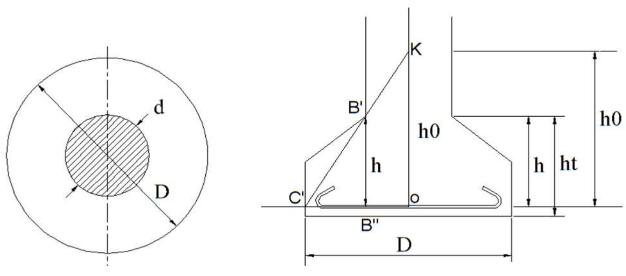


Figure 9 : Circular footing

3.5.2 Calculating reinforcement of circular footings

If the two reinforcement systems are perpendicular to each other, i.e., parallel to Ox and Oy.

$$F_x = F_y = \frac{Q.D}{3\pi.h_0} \text{, With } h_0 = \frac{D.h}{D-d}$$

The reinforcement cross-section will be equal to :

$$A_x = A_y = \frac{F_x}{\sigma_a}$$

The reinforcement is arranged as shown in Figure 10.

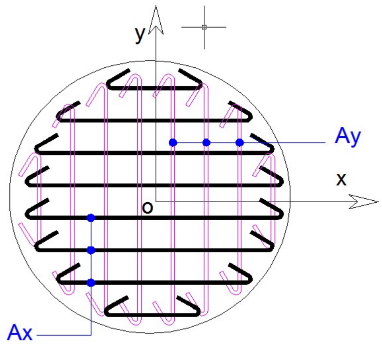


Figure 10 : Reinforcement of a circular footing

3.5.3 Distribution of reinforcement

If 1 m ≤ D ≤ 3 m , we divide the diameter D into three equal parts and place the Ax and Ay reinforcements according to the rule illustrated in Figure 11 ::

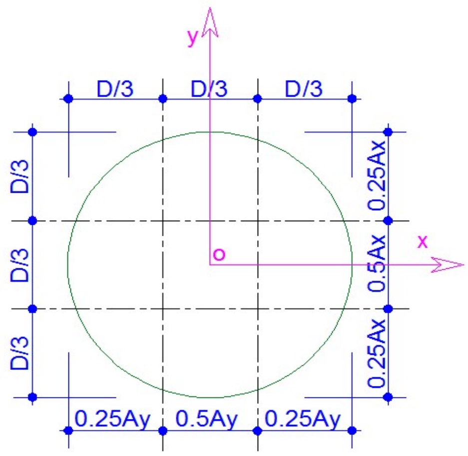


Figure 11 : Distribution of reinforcement in circular footings for 1m ≤ D ≤ 3m

If D > 3 m, divide the diameter D into 5 equal parts and place the Ax and Ay reinforcing bars according to the rule shown in Figure 12. It is important to note that the spacing between the reinforcing bars must not exceed 25 cm.

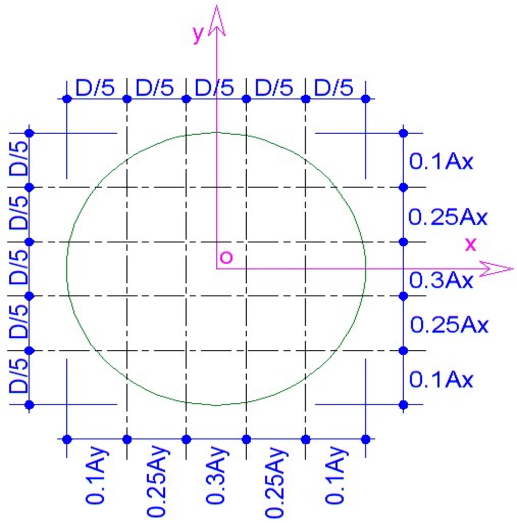


Figure 12 : Distribution of reinforcement for circular footings for D > 3m

If the reinforcement system is arranged in hoops (Figure 13) :

$$A'_c = \frac{Q.(D-d)}{6\pi.h.\overline{\sigma_a}}$$

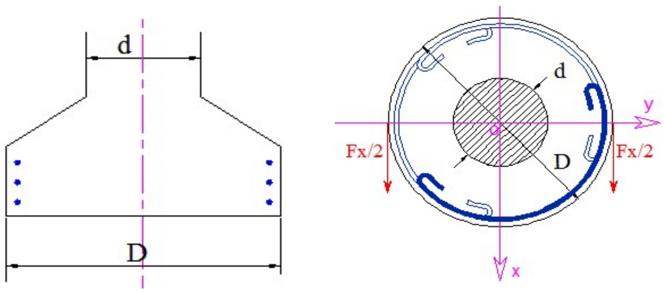


Figure 13 : Distribution of reinforcement in circular footings arranged in hoops

3.6 Ribbed footing under pillar

In this foundation system, the load, which is generally centred, is transmitted from the pillar to the rib and then in turn to the footing, which acts as a double inverted bracket embedded in the rib (Figure 14).

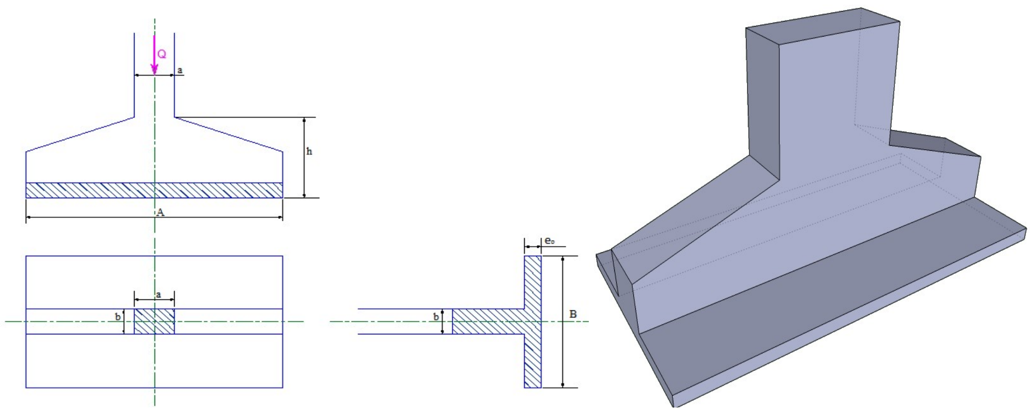


Figure 14 : Ribbed footing under pillar

3.6.1 Reinforcement of ribbed footings

The thickness of the footing is taken to be $e_0 = 10 \text{ to } 15 \text{ cm}$

The reinforcement of the rib is calculated using the moment M_n (Figure 15):

$$F_n = \sigma.B.\frac{A-a}{2} \qquad M_n = F_n.\frac{A-a}{4} \qquad M_n = \sigma.B.\frac{(A-a)^2}{8}, \quad \text{Hence} \quad A_S = \frac{M_n}{z_n.\sigma_a} \quad \text{With} \quad z_n = \frac{7}{8}h$$

The reinforcement of the footing (double bracket slab) is calculated using the moment M_s:

$$F_s = \sigma.A.\frac{B-b}{2} \qquad M_s = F_s.\frac{B-b}{4} \qquad M_s = \sigma.A.\frac{(B-b)^2}{8}, \quad \text{Hence} \quad A_S = \frac{M_s}{z_s.\sigma_a} \quad \text{With} \quad z_s = \frac{7}{8}e_0$$

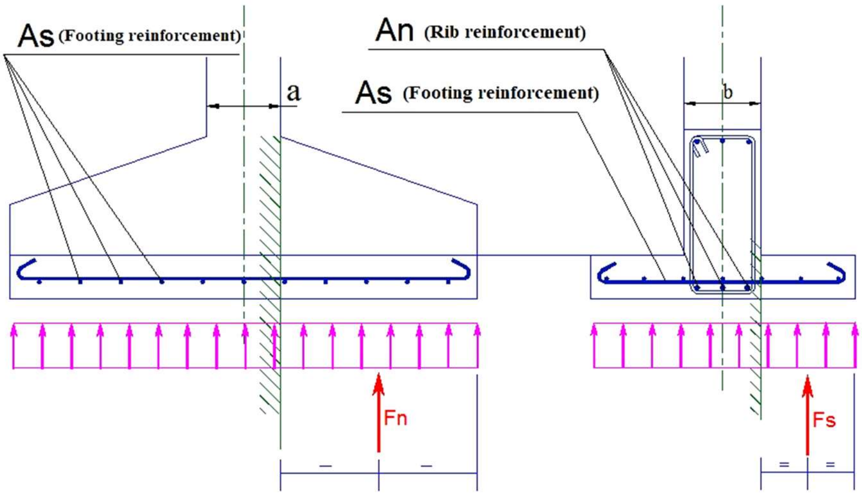


Figure 15 : Reinforcement of ribbed footings

3.6.2 Reinforcement of cross-ribbed footings

Footings support very high loads and generally have a square cross-section anchored at the ends of the diagonal ribs (Figure 16).

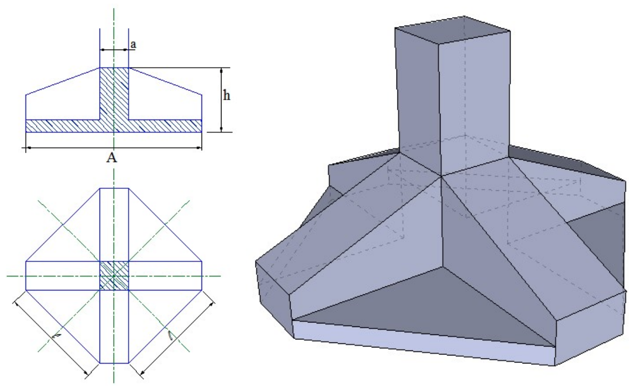


Figure 16 : Cross-ribbed footings

There are 4 triangular slabs supported as brackets on two ribs. These slabs are calculated parallel to the dimensions of the "square" or "pseudo-square" footing using strips 10 to 20 cm long (l).

As a first approximation, we can take :

- On support : $M_a = -\frac{\sigma l^2}{20}$
- In span : $M_t = -\frac{\sigma l^2}{10}$ Hence $A_s = \frac{M_t}{z_s.\overline{\sigma_a}}$ With $z_s = \frac{7}{8} e_0$

To reinforce the cross ribs of the footings (Figure 17), there are 4 pieces of bracket ribs embedded in the column each bracket supports ¼ of the footing without the column extension.

$$F_n = \sigma.\left[\frac{(A-a)^2}{4} + \frac{a.\frac{(A-a)}{2}}{2}\right] \qquad M_n = \sigma.(A+a).\frac{(A-a)^2}{8},$$

$$M_s = F_n.\frac{A-a}{4}$$

$$\text{Hence} \quad A_n = \frac{M_n}{z_n.\overline{\sigma_a}} \quad \text{With} \quad z_n = \frac{7}{8} h$$

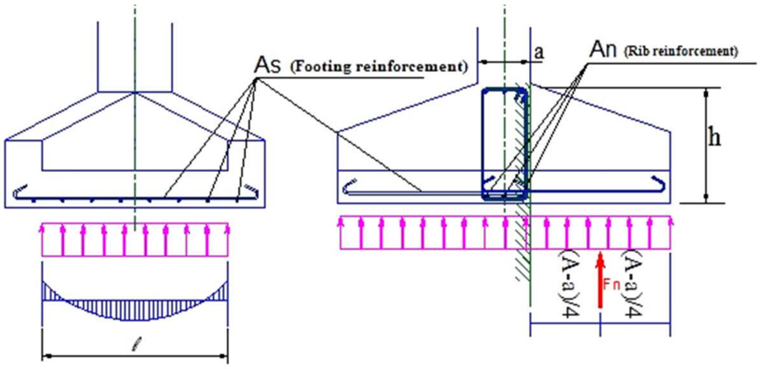


Figure 17 : Reinforcement of cross-ribbed footings

4. Strip footing

It is used to distribute loads over a larger area so that they do not sink into the ground. It is a continuous straight footing (Figure 18) supporting a wall or a row of pillars/columns.

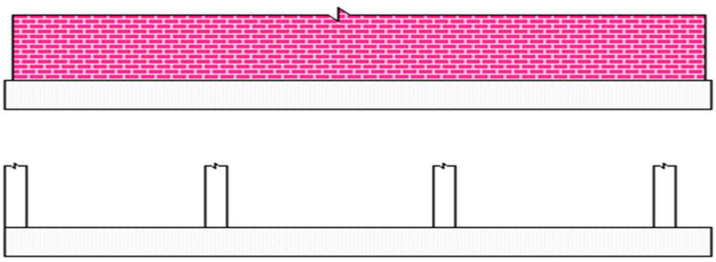


Figure 18 : Diagram of a strip footing

A strip footing surmounted by a wall is easy to dimension because we take a unit length and use the same calculations as for an insulated footing, whereas when it is surmounted by columns, there are three main parameters influencing the design of this strip footing as follows:

- Soil compressibility
- Rigidity of the footing
- Distribution of forces

In practice, a simplified method will be used, which will allow a credible dimensioning of the footing depending on the type of footing and soil. There are four possible cases :

- **Uniform soil**

The footing is rigid, so the load-bearing capacity at any point on the footing is the same (Figure 19).

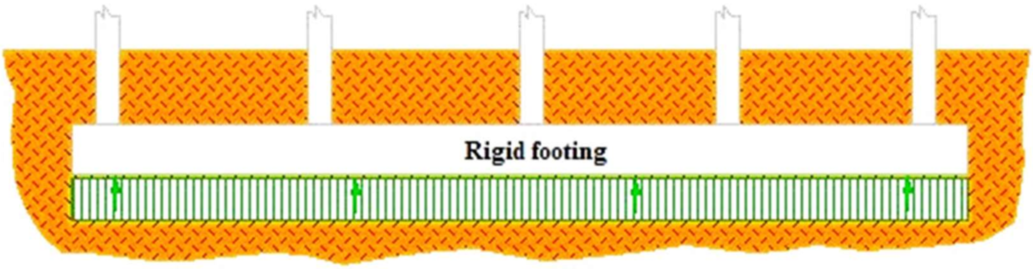


Figure 19 : Stress diagram of a rigid footing for a uniform soil

Flexible footing, the load-bearing capacity is greater below a column than between two columns (Figure 20).

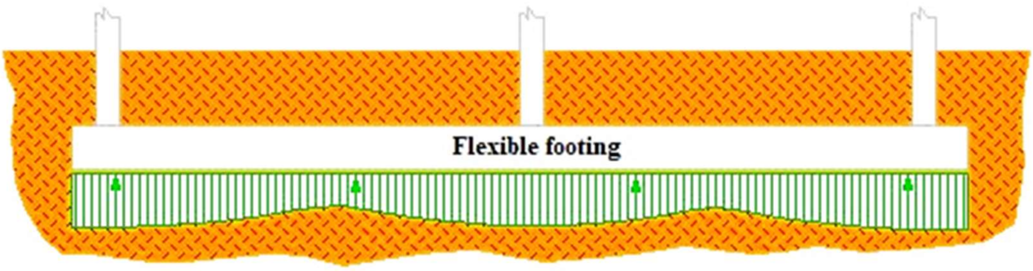


Figure 20 : Stress diagram of a flexible footing for a uniform soil

- **Non-uniform soil**

If the footing is rigid, where the ground is soft, the load-bearing capacity of the footing is reduced, whereas it is increased at the ends of this softer section (Figure 21).

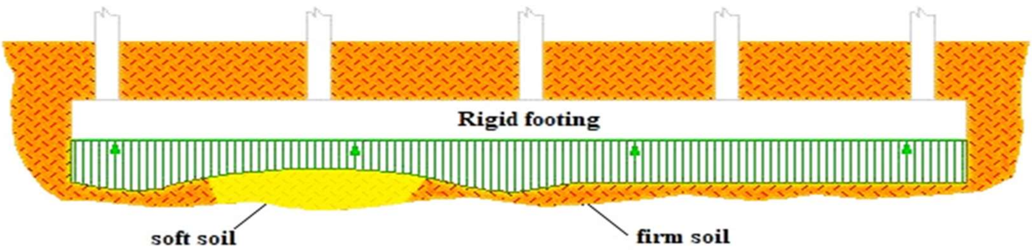


Figure 21 : Stress diagram of a rigid footing for a non-uniform soil

If the footing is flexible, there are two possibilities :

- Either the softer part is located between 2 columns, in which case the base undergoes a slight upward deformation, reducing the load-bearing capacity and increasing it at the ends of this less rigid zone (Figure 22).

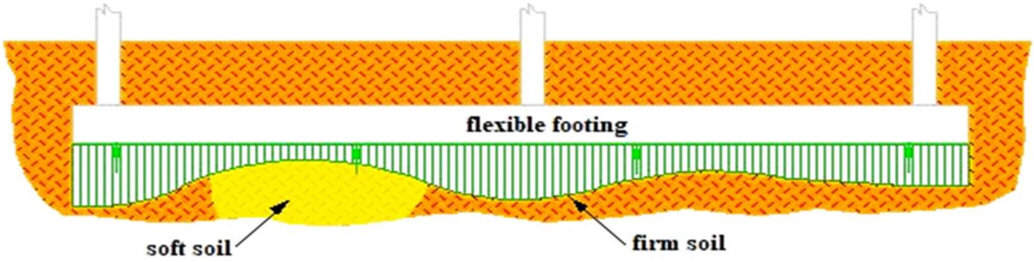


Figure 22 : Stress diagram of a flexible footing for non-uniform ground with the flexible part between 2 columns.

- Either the softer part is under a column, in which case the load-bearing capacity is homogenised (Figure 23).

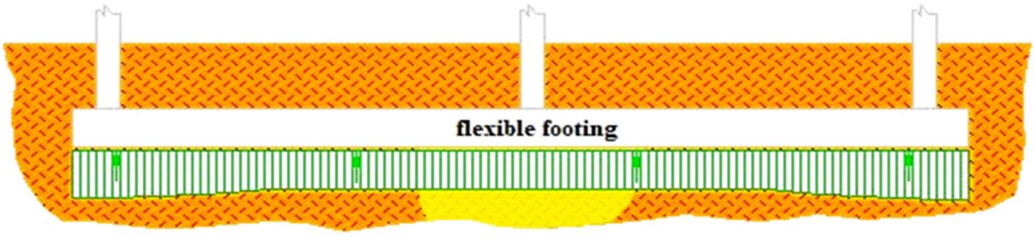


Figure 23 : Stress diagram of a flexible footing for non-uniform ground with the flexible part under a column.

These elastic models do not take into account the plastic behaviour of the soil. In reality, the distribution of the load-bearing capacity of the footing is more uniform, which is why footings can be calculated using this simplified model, as long as the composition of the soil is more or less homogeneous. The 4 cases do not affect the design of the footings too much, despite what the simplified model might suggest. This is because the model does not take into account the plasticity of the soil.

4.1 The height of the strip footing

The height of the footing is calculated by :

$$H = Max\ (h_1, h_2, h_3)$$

- **The height h_1 (connecting rod method)**

The height of the footing is given by the relationship : $h = d + 0.05$

$$h_1 \geq \frac{B-b}{4} + 0.05\ m$$

- **The height h2 (Checking the elastic length)**

Based on the elastic length, a distinction is made between different types of footings:

- Rigid strip footing, if $L_{max} \leq L_e \times \pi/2$

- Flexible strip footing, if $L_{max} \geq L_e \times \pi/2$

L_{max} : The greatest distance between columns

$$\text{Elastic length } L_e = \left(\frac{4EI}{K_s B}\right)^{\frac{1}{4}}$$

With :

E : Modulus of elasticity of concrete.

K_s : Soil stiffness coefficient.

I : Moment of inertia of the footing ($B h^3/12$).

B : Width of footing.

$$h_2 \geq (48K_s L^4_{max} / E \pi^4 B)^{1/3}$$

- **Height h3 (Standard condition)**

$$\frac{L_{max}}{9} \leq h_3 \leq \frac{L_{max}}{6}$$

4.2 Checks on the strip footing

- **Non-punching check**

Under the action of localized forces, it is necessary to check the resistance of the threaded footings to punching by the shear force.

$$\text{Check that: } \tau = \frac{N_{ser}}{2h_t}\left[1 - \left(b + \frac{5h_t}{3}\right)/B\right] \leq \tau_{lim}$$

N_{ser} : Normal force at the most stressed column of each footing.

$\tau_{lim} = 0.045 f_{c28} / \gamma_b$: Shear stress limit value.

- **Checking reversal stability**
- **Checking stresses in the ground**

5. Raft foundation

A raft foundation is presented as an inverted floor with or without a beam, receiving distributed upward loads from the ground and supported by columns and walls which in return exert downward burdens on it.

5.1 Choice criteria

The choice is justified if :

- It is a poor soil that requires large surfaces.
- The surface of the footings is less than 50% of the total building area.

$$\frac{S_s}{S_b} < 50\%$$

- When the soil is too compressible.
- To avoid or reduce subsequent disorders in case of differential settlement.
- When the raft foundation forms a waterproof casing in the case of basements.
- Massifs on piles are sometimes used to ensure a uniform distribution over all piles, this type of foundation can be considered rigid.

The methods for calculating the raft foundation are defined once the constraint distribution assumptions under the raft foundation are known (Table 1).

Table 1 : Distribution of paving loads.

Paving types	Maximum distributed load (kN/m^2)	Maximum concentrated load (kN/m^2)
Residential paving.	2.5	15
Storage paving supporting light circulation.	8	25
Paving for common industrial use.	20	60
Paving for heavy industrial use.	120	120
Raft foundation.	> 120	> 120

5.2 Mode of operation

- **Mechanical actions acting on the raft foundation**

As shown in Figure 24, two types of action act on the raft foundation:

- Downward actions (dead weight, superstructure weight and external actions) transmitted by walls and columns.
- Ascending actions of the soil distributed under all its surface.

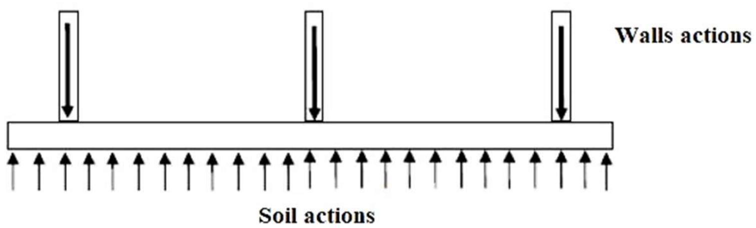


Figure 24 : Mechanical actions acting on the raft foundation

- **Calculation hypotheses**

The pressure distribution on the ground is uniform. This requires a raft foundation of high rigidity (high concrete thickness - high reinforcement density) and if possible, equally distant and equally loaded poles, but generally the poles are unevenly loaded. Simplifications can thus be allowed in the constraints diagram.

The raft foundation behaves like an overturned floor, so it is necessary to strengthen it in line with the supports of walls and columns.
If Concrete does not withstand traction, reinforcement will be placed in the tense areas, that is, in the upper part in the spans and in the lower part in the right of the walls and columns.
The reinforcement of the flat raft foundation to the right of the carriers is necessary and the diagrams of principle of reinforcement of the raft foundation subjected to point loads or to transmitted linear loads are illustrated in Figure 25.

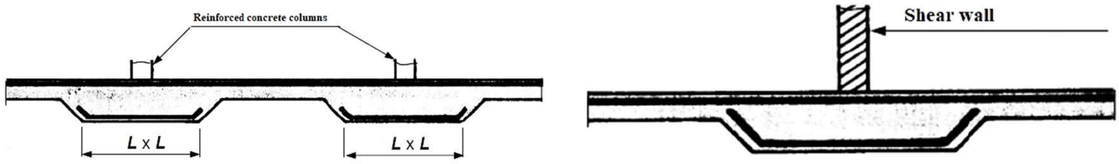


Figure 25 : Reinforcement of flat raft foundation to the right of the carrier elements

The completion of raft foundation can only be considered under certain conditions:

- The loads brought by the building must be evenly distributed, that is, no building with a high part and a lower part to not cause incompatible settlements.
- Stress distribution under the raft foundation is uniform.
- The ground under the raft foundation is only subjected to compression stresses at any point.
- The supporting floor has a regular resistance (no differential settlement, no hard points).
- Archimedes thrust due to a presence of water is not too strong (whole building lift).

5.3 Different types of raft foundation

- **Flat raft foundation with a constant thickness**

Suitable for fairly low loads and small footprint buildings, it is easy and fast to execute. The walls or columns are supported directly on the slab with the possibility of reinforcing the concrete sections in line with the supports.

- **Ribbed raft**

When the loads are high, and so that the thickness of the raft does not become excessive, beam spans (ribs) are available to stiffen the slab. They can be arranged in one direction or in both directions and that depends on the scope and the layout of walls or columns. The assembly gives alveoli that it is necessary to backfill if we want to use the basement or make a second slab in the upper part.
The slab of the raft can be located in the lower part (Figure 26) which is the rational solution. The slab, placed in a compressed area, strengthens the beam which, therefore, is in an upside-down T-shaped with great rigidity.

The disadvantages of the raft slab in the lower part:

- Important but simple excavation.
- Complicated and important formwork.
- Need to fill in the gaps between the beams and ribs to make use of the surface area.
- Higher risk of under-pressure.

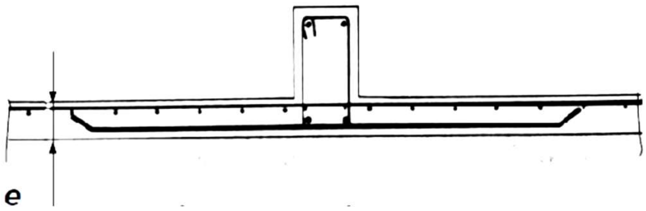


Figure 26 : The slab of the raft in the lower part

If the raft slab is at the top (Figure 27), then the slab is in the tension zone of the beam and does not contribute to its strength. The advantage of this arrangement is that the upper surface of the slab can be used directly.

The disadvantages of the upper part of the raft slab :

- Complex earthworks.
- The reinforcement is more complicated and the secondary reinforcement must take up the loads and transfer them to the compressed areas.
- Greater thickness of the slab, therefore increased weight.

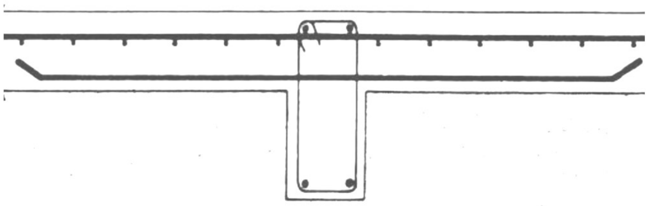


Figure 27 : The slab of the raft in the upper part

- **Mushroom raft**

In the case of frame construction, the floor slab can be treated using the mushroom floor principle. There are no ribs, giving a flat, unobstructed surface for long spans.

The loads are transmitted from the columns to the thick slab by means of capitals, so that the load is distributed gradually. It is necessary to distribute the columns evenly (the span in one direction cannot exceed 2 times the span in the other direction). This type of slab is easy to build, but the capitals take up a lot of floor space.

- **Arched raft**

Vaults make it possible to increase the spans (distance between load-bearing elements) without significantly increasing the thickness of the raft. The axis of the vaults is perpendicular to the long dimension of the raft floor and their installation is fairly complex, but vaulted raft floors are thin (12 to 20 cm) because they work essentially in compression; they are therefore economical in terms of concrete and steel.

Loads need to be distributed symmetrically, as the thrusts of the vaults are taken up by abutments (at the ends) or by tie rods (approximately every 4 m). The tie rods can consist of steel bars or reinforced concrete beams placed perpendicular to the axis of the vaults, and they can be weighted down with sand if necessary (in the event of under-pressure).

The disadvantages of the arched raft :

- Difficulty in shaping the concrete of the vault.
- Complicated tie-rod formwork.
- Difficult to fill in hollows to make the surface usable.

5.4 Raft calculation methods

Generally speaking, it is impossible to know the exact distribution of reactions under a raft. This is because they depend on the nature of the soil and the respective coefficients of elasticity of the soil-radius and radius-structure.

Calculating an invert therefore requires simplifying assumptions to be made about the soil reaction diagrams. However, it is essential to verify the static conditions, i.e., the overall balance between the reactions of the ground and all the loads applied by the superstructure.

For simplicity's sake, the raft is always considered to be infinitely stiff in relation to its superstructure. In other words, the columns and walls resting on the raft are considered to be articulated at their base. On the other hand, the raft is more or less deformable in relation to the foundation soil.

If the raft can be considered rigid, the calculation is carried out on the basis of a linear distribution of ground reactions. The dimensions of the raft must be such that the forces of the superstructure can be transmitted, the internal forces of the raft being determined by the equilibrium of the forces on the left (or right) of any section.

The inverted floor calculation is only valid if the balance between the downward loads contributed by the superstructure and the reactions of the ground under each column is substantially verified.

As a first approximation, the thickness of the elements making up the raft is determined by the following relationships :

- Ribs : $h_1 \geq \frac{l'}{10}$, With l' between column centres parallel to the ribs.
- Slab : $h_2 \geq \frac{l}{20}$, With l between column centres perpendicular to the ribs.

In addition, the thickness of the slab must be such that the shear force check is ensured without the need for shear reinforcement. In the case of poor ground conditions, the raft is considered to function either as an inverted ribbed floor or as an inverted floor-slab.
If the invert is considered to be flexible, then this method does not take continuity into account. It consists of checking the static conditions and the non-reinforcement of the soil under the surface corresponding to each column or wall taken in isolation. These surfaces can be discontinuous if the strength of the soil allows. The shape of the diagram chosen can be either rectangular or triangular (powdery ground).
Several methods can be used to calculate the raft floor.

- **Calculation and verification methods**

The simplistic method is still the most widely used today for calculating and checking rafts. All the reactions are distributed according to a trapezoidal or uniform diagram depending on the resultant of the forces and moments.
The calculation is carried out per strip, and the ground reactions are given by the formula :

$$\sigma_{1,2} = \frac{N}{L}\left(1 \pm 6\frac{e_0}{L}\right) \quad \text{With } e_0 = \frac{M}{N} \leq \frac{L}{6} \ (\frac{L}{6} \text{ according to RPA 99 version 2003})$$

Checking the raft surface :

$$S_{raft} = \frac{N}{\sigma_{Soil}} \leq S_{Bui}$$

Calculation of D (overflow):

$$D \geq \text{Max}\left(\frac{h_r}{2}, 30 \text{ cm}\right) \qquad S_r = S + 2D\,(X+Y)$$

With :
Sr : Raft surface area.
S : Total surface area of building.
X, Y : Length and width of building.
Raft thickness :
The raft is considered to be infinitely rigid, so the following conditions must be satisfied :
a- Rigidity condition

$$L_{max} \leq L_e \times \pi/2$$

With

$$L_e = \left(\frac{4EI}{K_S B}\right)^{\frac{1}{4}}$$

L_{max} : Maximum span length of the strip in question.

E : Modulus of elasticity of the concrete.

I : Moment of inertia of the section of the span considered = $B.h^3/12$

B : Width of the section of the strip under consideration (B = 1m).

K_s : Soil stiffness coefficient with $K_s = 1.33\ E'/(L.l^2)0.333$

E' : Compression modulus measured using a laboratory oedometer.

L, l : Length and width of the plate under consideration.

b- Punching verification

The raft must be thick enough to resist the shear stresses due to the punching of the columns on the surface of the plate. Under the action of localized forces, the resistance of the raft to shear stress punching must be checked.

This check is carried out in accordance with article A.5.2.4 of CBA93 as follows :

$$N_u \leq 0.045.\mu_c.hr.f_{c28}/\gamma_b$$

Nu : Calculation load with respect to the ultimate limit state of the most stressed column.

μ_c : Perimeter of sheared contour, projected onto the mean plane of the raft.

With $\mu_c = 2\ (L + b + 2\ h_r)$

c- Shear resistance condition

The thickness of the raft will be determined according to the shear stress of the raft.

According to article A.5.1 of the CBA93 regulations, this is equal to :

$$\tau_u = \frac{V_u}{bd} \leq 0.07\ \frac{f_{cj}}{\gamma_s}$$

With

V_u : Design value of the shear force with respect to the

ELU. b : Width of the raft.

γ_b =1.15; d = 0.9 h and b = 1m.

We deduce the value of the height which is equal to:

$$h_r \geq \frac{V_u\ .L_{max}.\gamma_s}{0,9.2S.0,07.f_{cj}}$$

With

L_{max} : Longest span of the slab.

d- Standard condition

The thickness of the raft must satisfy the following condition :

$$\frac{L_{max}}{8} \leq h_r \leq \frac{L_{max}}{5}$$

With

L_{max} : Maximum distance between two successive walls.

For the inverted floor method, the following assumptions are made :

- Uniform distribution in the case of rock, with loads concentrated near the columns.
- The pressure under the raft is taken to be equal to: $q = \sigma_{moy} - \frac{N}{S}$
- The raft is divided into several panels according to the position of the columns.
- Each panel is considered as a slab supported on four sides.

  $M_x = \rho_x.q.L_x$ and $M_y = \rho_y.M_x$

  With $\rho = L_x/L_y$

  L_x : Length of next panel x x and L_y next y.

  ρ_x , ρ_y : Moment distribution coefficient according to x and y (Pigeaud tables).

Taking continuity into account, we have :

For an edge panel :

- Moment in span : 0.85 M.
- Moment at edge support : 0.3 M.
- Moment at intermediate support : 0.5 M.

For an intermediate panel :

- Moment in span : 0.75 M.
- Moment on support : 0.5M.

Other methods can be used, such as Westergaard's method, Hoog and Burmister's method (improved by Peltier and Jeuffroy), Dantu's method, Pickett's method, Chary's method (square slabs) and finite element methods.

CHAPTER 2

Deep foundations

1. Different types of deep foundations

- **Diaphragm walls**

Diaphragm walls are reinforced concrete walls poured in situ into the soil (Figure 28). Before the reinforced concrete is poured, a trench must be dug, which is constantly filled with mud during excavation.

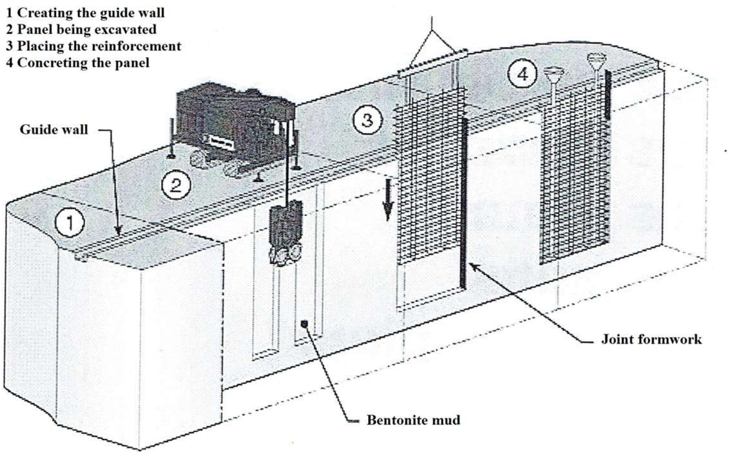


Figure 28 : Diagram of a diaphragm wall

- **Barrettes**

These are diaphragm wall elements (0.60 m to 1.00 ml wide), 2 to 6 m long, used as load-bearing elements. They can be secant or parallel (Figure 29).

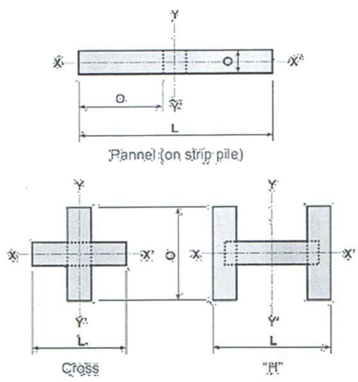


Figure 29 : Barrette diagram

- **Piles**

These are elements made of wood, metal, reinforced concrete, sometimes prestressed concrete, whose length is 5 to 10 times greater than its transverse dimension (B).

- If B < 0.80 m : A pile
- If B > 0.80 m : A well.

However, in common parlance, the term " well " is only used for hand-made elements.

- **Micro-piles**

These are piles with a diameter of ϕ < 250 mm. There are 4 types of micro-piles, classified according to drilling equipment and injection techniques.

- Type 1: Tubed drilled pile, filled with mortar (FONDEDILE type micropiles).
- Type II : Bored pile, fitted with reinforcement, sealed with cement grout or mortar by gravity using a dip tube.
- Type III : Bored pile equipped with reinforcement and an injection system consisting of a sleeve tube placed in a casing grout. The cement grout is injected at the head at a pressure equal to or greater than 1 MPa. Injection is global and unitary (IGU).
- Type IV : Bored pile, fitted with reinforcement and an injection system consisting of a sleeve tube placed in a casing grout. The cement grout is injected at each sleeve level using a single or double shut-off valve at a pressure equal to or greater than 1 MPa. Injection is repetitive and selective (IRS).

2. Pile technology and installation

There are two main types (Figure 30) :

- Piles installed with soil displacement.
- Piles installed without soil displacement.

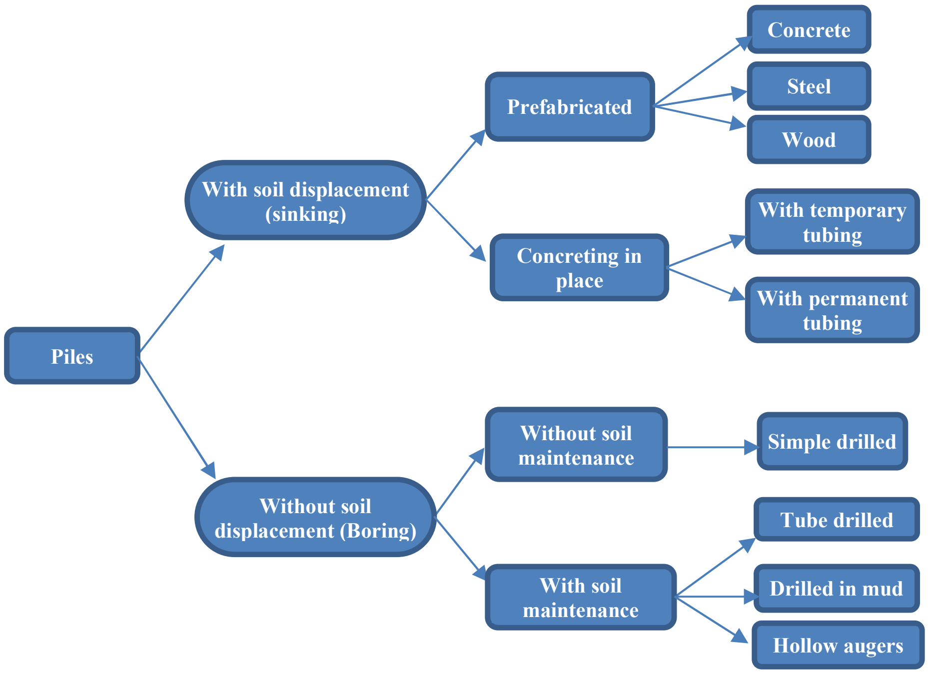


Figure 30 : Flowchart of how piles work

2.1 Piles installed with soil displacement

2.1.1 Prefabricated piles

There are three main families :

- **Wooden piles**

These were the first types of pile to be used. The wood used was oak (hardness, resistance to air and water), beech, elm (hardness), maritime pine, spruce, Scots pine and Oregon pine, and their heads were shrunk with hoops and the point was fitted with a metal shoe.

- **Reinforced or prestressed concrete piles**

The cross-section of these piles is square, circular or polygonal and the transverse dimension varies from 25 to 60 cm. The most common are 30 x 30 or 40 x 40 cm^2 square. They can be up to 40 to 50 m long, with one element joined to another (Figure 31).

Figure 31 : Reinforced concrete piles

- **Steel piles**

There are several types :

- H or I profile.
- Sheet piling.
- Tubular piles.

Metal H or I piles are rarely used, but they are very interesting if they are used at an angle (to take up horizontal forces) and can be up to 100 m long.

Metal sheet piles are welded assemblies of sheet piles (2, 3 or 4) with a closed profile and a length of up to 30 m.

Tubular piles (pile driver/ vibratory pile driver) are helically welded tubes, decommissioned pipelines with a diameter of ϕ 200 to 600 mm, thicknesses ranging from 7 to 50 mm and lengths of up to 100 m.

2.1.2 Concrete piles in place

There is only one family, which is cast piles with beaten tubes.

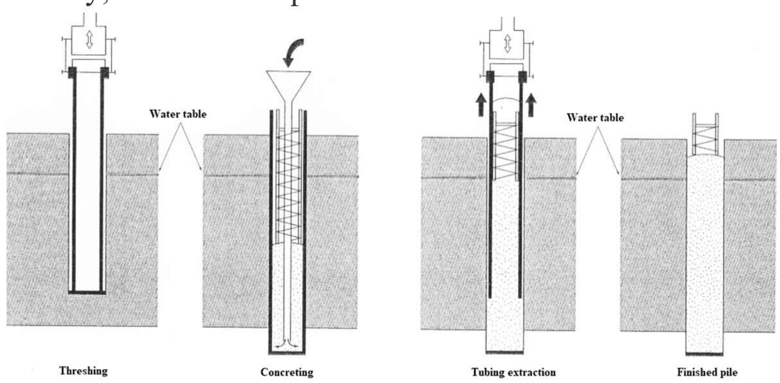


Figure 32 : Beaten-tube cast piles

2.2 Piles installed without soil displacement

- **Concrete piles in place**

There are several families :

- Tubed or un-tubed piles.
- Mud-bored piles.
- Jacketed piles.

2.3 Wells

These are hand-made elements, generally unarmed, whose walls are supported by armour, possibly of variable width (elephant's foot).

3. Pile behaviour

When the surface foundation soil is of poor quality, it is necessary to provide a support level at depth. The loads applied by the structure's supports are transmitted to the soil foundation via steel or concrete interposed elements.

The loads are transmitted at depth to the end of the interposed element (the "tip") but also by friction on the side walls of the foundation, which is very different from the way shallow foundations work.

Depending on the depth to be reached, a distinction is made between deep foundations, which enable layers with a bearing capacity of over 2 MPa (20 bar) to be reached at very great depths, and so-called semi-deep or massive foundations, which are shallower but have a larger cross-section, enabling soils with a bearing capacity of 1 MPa (10 bar) to be reached.

The limit load Q_l (or Q_u) s the sum of the peak force Q_p and the lateral friction Q_s. It corresponds to the punching of the soil by the pile (Figure 33).

- $Q_p = q_p A_p$ (Under-peak unit resistance per cross-section of the pile tip).
- $Q_s = q_s A_s$ (Resistance due to lateral friction per lateral surface of the pile).

The creep load Q_c is the load above which there is permanent pile driving.

If the piles work in tension (rare), Qtc and Qtu are also defined. In the absence of pile loading tests (the majority of cases), the design regulations allow the various loads defined above to be determined.

At a depth z, the intensity of the negative friction stress is noted τ_n (value given by the regulations unless tested). The neutral point is the point above which the sections are subject to negative friction.

The total negative friction is the sum of the negative friction stresses over the entire lateral section located above the neutral point.

Q_c and Q_u are not modified in the case of lateral friction.

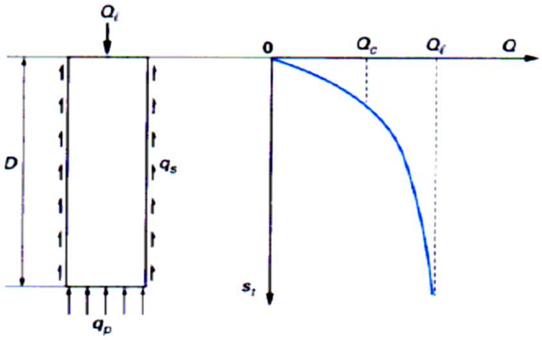


Figure 33 : Global behaviour of a pile

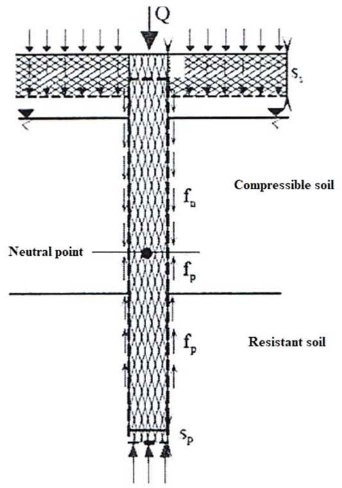


Figure 34 : Negative pile friction

4. Bearing capacity of piles

The expression for limit loads is :

- $Q_u = Q_{pu} + Q_{su}$ (compression)
- $Q_{tu} = Q_{su}$ (traction)

The expression for creep loads, for elements installed without soil displacement, is :

- $Q_c = 0.5\ Q_{pu} + 0.7\ Q_{su}$ (compression)
- $Q_{tc} = 0.7\ Q_{su}$ (traction)

The creep loads for elements installed with soil displacement are expressed as follows :

- $Q_c = 0.7\ Q_{pu} + 0.7\ Q_{su} = 0.7\ Q_u$ (compression)
- $Q_{tc} = 0.7\ Q_{su}$ (traction)

The expression for the peak limit force (full point piles) is :

- $Q_{pu} = A\ q_u$

A : Tip section

q_u : Peak rupture stress

The expression for the limit lateral friction force (solid point piles) is :

- $Q_{su} = P \int_0^h q_s\ (z)\ dz$

P : Perimeter of the foundation element

$q_s(z)$: Lateral friction unit limit at dimension z

The expression of the peak limit stress (open tubular piles, H-piles, sheet piles) is :

- $Q_{pu} = \rho_p A q_u$

A : Tip cross-section

q_u : Peak rupture stress

ρ_p : Reducing coefficient

The expression for the limit lateral friction force (open tubular piles, H-piles, sheet piles) is :

- $Q_{su} = \rho_s P \int_0^h q_s (z) dz$

P : Perimeter of the foundation element

$q_s (z)$: Lateral friction unit limit at dimension z

ρ_s : Reducing coefficient

5. Determining the bearing capacity of bored piles (using static formulae)

This calculation method requires knowledge of the mechanical characteristics of the soil, which must be determined either in the laboratory or in situ. For each layer of soil, the densities (γ and γ'), cohesion (C) and angle of internal friction (φ°) must be known. Deep foundations mobilise the soil by lateral friction and by the peak effect in proportion to their cross-section.

5.1 Principle of bearing capacity calculation

The ultimate bearing capacity Ql of a bored pile is given by the equation as follows :

$$Q_l = Q_p + Q_f$$

Q_l: Ultimate bearing capacity

Q_p: Bearing capacity due to the tip of the pile

Q_f: Bearing capacity due to pile/soil lateral friction

Let be,

A: Lateral friction surface of the pile in the bearing layers

S: Cross-sectional area of the pile tip

q_f: Stress due to unit lateral friction

q_p: Limit stress due to the pile tip

This gives:

$$Q_l = S.\, q_p + A.\, q_f$$

5.2 Bearing force due to the tip of the pile

The following relationship is used :

$$Q_p = \left(\frac{\pi . B^2}{4}\right)\left[N_q \sum_{i=1}^{i=n} \gamma_i . D_i + 1{,}3 . C . N_c\right]$$

Tests carried out by Caquot and Kerisel on small-diameter piles have led to the following proposals the following values for the bearing capacity factor N_q :

$$N_q = 10^{N.tg\varphi}$$

φ: Angle of friction of the supporting layer

B: Diameter of the pile.

N = 3,7 if B < 32 cm

N = 2,7 if B = 32 cm

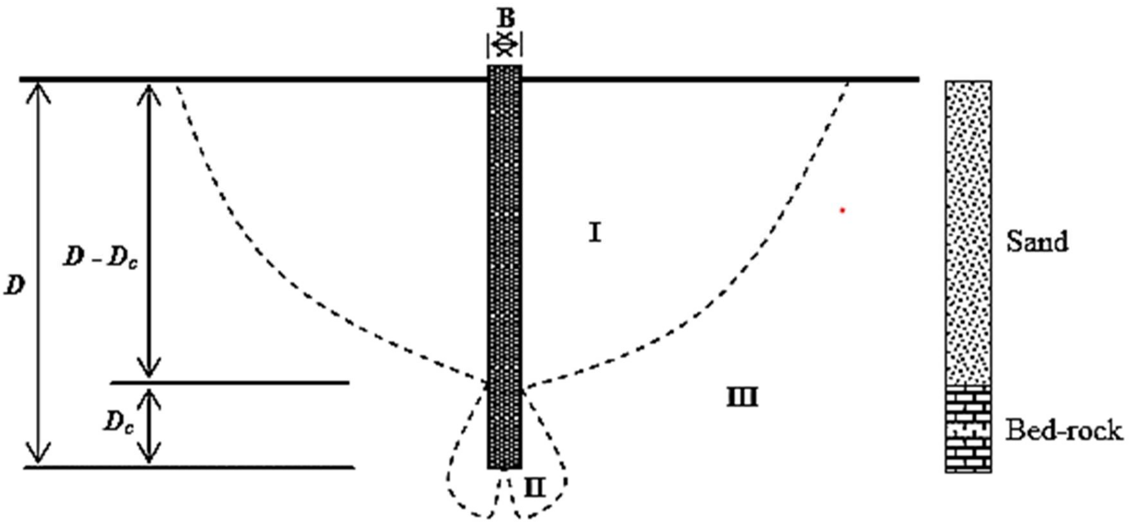
If: B > 32 cm $N_q = tg^2\left(45 + \frac{\varphi}{2}\right)e^{\pi.tg}$

The values proposed for Nq by Caquot and Kerisel are valid on condition that the tip of the pile has a minimum engagement in the good soil equal to the critical plug D_c.

with :

$$D_c = \frac{B}{4}N_q^{\frac{2}{3}}$$

The equilibrium state of a pile can be represented as follows (Figure 35) :



Zone I: Corresponds to lateral friction along the shaft, the medium is in near-stop equilibrium.
Zone II: Corresponds to the force at the tip, the medium is in abutment equilibrium.
Zone III: Located beyond the sliding lines, in these zones the medium is in pseudo-elastic equilibrium.

Figure 35 : Schematic diagram of how a pile works

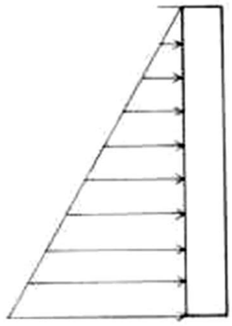
5.3 Load-bearing capacity due to lateral friction

Lateral friction will only be taken into account over the height (D - Dc)

- **Powdery environment**

As the pile is driven into the soil, it pushes back the ground, generating an abutment reaction in the solid mass inclined at an angle (δ) to the horizontal such that (Figure 36) :

$$q_f = \gamma.Z.k_p.\sin\delta$$



$\gamma . Z. k_p. \sin \delta$

Figure 36 : Soil reaction due to lateral friction

With k_p the stop coefficient corresponding to a vertical screen.

If we denote by P the perimeter of the section of the pile, at dimension Z, we obtain the following relationship for the load-bearing force Q_f :

$$Q_f = P.q_f.\frac{Z}{2} = \frac{1}{2}\, P.\gamma.Z^2.k_p.\sin\delta$$

In practice, is in particular :

- For less dense media, i.e. those for which $\gamma \leq 1.8$ t/m^3 it is advisable to $\delta = -(2/3)\,\varphi$

- For dense media where $\gamma > 1.8$ t/m^3, we recommend: $\delta = -\varphi$

In order to facilitate calculations of the load-bearing capacity, Caquot proposes in Table 2 the values of $k_p \sin(-\varphi)$ and $k_p \sin(-2/3\,\varphi)$.

Table 2 : Values of $k_p \sin(-\varphi)$ and $k_p \sin(-2/3\,\varphi)$ according to Caquot

φ°		**0°**	**10°**	**15°**	**20°**	**25°**	**30°**	**35°**	**40°**	**45°**
dense environments	$k_p \sin(-\varphi)$	0	0,285	0,567	1,03	1,81	3,21	5,85	11,30	23,70
low-density environments	$k_p \sin(-2/3\varphi)$	0	0,186	0,364	0,641	1,10	1,88	3,27	5,90	11,40

- **A medium that is both frictional and cohesive**

To the unit friction calculated for powdery media, the effect of cohesion is added.

$$q_{fc} = k_p(C.cotg\ sin\delta) = C.k_c$$

With ;

$$k_c = (1+\sin\varphi).e^{\left(\frac{\pi}{2}+\varphi\right).tg\varphi}$$

The values of kc are given in Table 3 for $\delta = -\varphi$ and $\delta = -(2/3)\,\varphi$

Table 3 : Values of k_c

φ°		**0°**	**10°**	**15°**	**20°**	**25°**	**30°**	**35°**	**40°**	**45°**
dense environments: $\delta = -\varphi$	k_c	1	1.6	2.06	2.70	3.62	5.01	7.27	10.36	17.97
low-density environments: $\delta = -(2/3)\,\varphi$	k_c	1	1.24	1.43	1.67	2.00	2.47	3.14	4.04	5.39

- **Purely coherent medium (φ = 0)**

In the case of a purely coherent soil the value of the stress due to friction is such that:

$$q_f = C.\ k_c = 1$$

Caquot and Kerisel suggest adopting the following experimental value for q_f :

$$q_f = C_u . \frac{1 + C_u^2}{1 + 7C_u^2}$$

With,

C_u is the apparent cohesion expressed in bars.

5.4 Safety coefficient

It is usual to adopt the safety factors below to obtain the admissible bearing capacity of a Q_{ad} pile, i.e. $F_s = 2$ for the peak force alone and $F_s = 3$ for lateral friction.

Hence :

$$Q_{ad} = \frac{Q_p}{2} + \frac{Q_f}{3}$$

6. Bearing capacity of deep foundations (LCPC pressure-meter methods) :

The pressure-meter method takes account of soil heterogeneity by basing itself on the concept of the equivalent homogeneous soil, characterised by an equivalent limit pressure P_{le} and surrounding a pile with an equivalent plug D_e. The equivalent limit pressure is used to calculate the peak resistance. It is an average of the values measured in a zone in the vicinity of the point, which is (3a) below the point and (b) above it (see Figure 37).

$$P_{le}^* = \frac{S}{b + 3a} \int_{D-b}^{D+3a} P_l^*\,(z)dz$$

With :

P_{le}^* : Equivalent net limit pressure

$P_l^*(z)$: limit pressure obtained at depth (z) by linear interpolation between P_l^* measured immediately on either side of this depth,

a = max (B/2 and 0,50) in metres, b = min (a, h) ;

h is the height of the resistance layer in which the point is embedded.

In homogeneous soil h = b = 0 and a is equal to B/2 for diameters greater than 1m. The zone of influence of the tip resistance will, in this case, be 1.5 B thick under the tip.

The above expression can be simplified using a summation:

$$P_{le}^* \approx \frac{1}{b + 3a} \sum_{D-b}^{D+3a} P_l^*\,.\Delta z$$

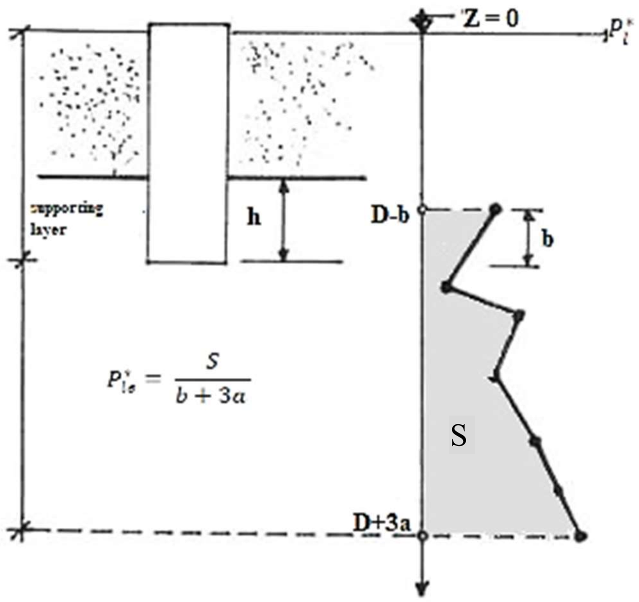


Figure 37 : Calculation diagram for the peak resistance of a pile.

As in the case of a shallow foundation, the equivalent form is that which allows the same pile to be studied in a homogeneous soil characterised by a limit pressure equal to P_{le}^* .

In practice, we use the approximate formula for summing the N net measurements P_{le}^* along the pile, considering a step ΔZ_i between two consecutive measurements :

$$D_e = \frac{1}{P_{le}^*} \sum_{i=l}^{i=n} P_{li}^* . \Delta Z_i$$

P_{le}^* and D_e obviously reduce to P_l and D respectively in the case of an ideal homogeneous soil.

- **The peak resistance**

This is calculated as follows :

$$q_p = K_p . P^*$$

The pressure bearing factor K_P, which depends on the nature of the soil and the method of installation of the soil, is given in Table 4.

Table 4 : Values of the pressure bearing factor K_P

Nature of the soil		Pile does not displace the soil (drilling....)	Pile driving the soil (driving, sinking, ...)
Clay / Silt	A	1,10	1,40
	B	1,20	1,50
	C	1,30	1,60
Sand / Gravel	A	1,00	4,20
	B	1,10	3,70
	C	1,20	3,20
Chalk	A	1,10	1,60
	B	1,40	2,20
	C	1,80	2,60
Marls, Chalky marl		1,80	2,60
Altered pockets		1,10-1,80	1,80-3,20

The LCPC soil classification and Table 5, are used to define the class of the material and its category.

Table 5 : Conventional soil categories

Soil class	Pressure-meter P1 (MPa)
A- Soft clays and silts B- Firm clays and silts C- Very firm to hard clays	< 0,7 1,2 to 2,0 > 2,5
Sand, gravel A- Loose B- Means C- Compacts	< 0,5 1,0 to 2,0 > 2,5
Chalks A- Looses B- Altered C- Compacts	< 0,7 1,0 to 2,5 > 3,0
Marls A- Tenders Marno - limestones B- Compacts	1,5 to 4,0 > 4,5
Rocks A- Altered B- Fragmented	2,5 to 4,0 > 4,5

The limit lateral friction q_s increases linearly with the net limit pressure, at the same depth. It actually depends on the nature of the soil surrounding the pile and the method of installation of the pile. The choice of the curve $q_s = f(P_{le}^*)$ is selected from Table 6 and q_s is determined from Figure 38.

The vertical limit load is calculated as follows :

$$Q_l = Q_p + Q_s = S.q_p + P\int_0^D q_s(z)dz$$

With :

S: Section of the pile,

P: Perimeter of the shaft.

It should be noted that in the absence of a pile loading test for the experimental determination of the critical load Qc can be estimated as follows :

−For piles that do not displace the soil (bored piles, bars, shafts, etc.) by :

$$Q_c = 0{,}5Q_p + 0{,}7Q_s$$

−For piles that do displace the soil (darkened piles, driven piles....) by :

$$Q_c = 0{,}7Q_p + 0{,}7Q_s$$

Table 6 : Choice of lateral friction curve

	Clay - Silt			Sand - Gravel			Chalk			Marls		Rocks
	A	B	C	A	B	C	A	B	C	A	B	
Simple drilled	Q_1	Q_1, Q_2 (1)	Q_2,Q_3 (1)	-			Q_1	Q_3	Q_4, Q_5 (1)	Q_3	Q_4, Q_5 (1)	Q_6
Mud drill	Q_1	Q_1, Q_2 (1)		Q_1	Q_2, Q_1 (2)	Q_3, Q_2 (2)	Q_1	Q_3	Q_4, Q_5 (1)	Q_3	Q_4, Q_5 (1)	Q_6
Tube drilled (recovered)	Q_1	Q_1, Q_2 (3)		Q_1	Q_2, Q_1 (2)	Q_3, Q_2 (2)	Q_1	Q_2	Q_3, Q_4 (3)	Q_3	Q_4	-
Cased borehole (lost pipe)	Q_1			Q_1		Q_2	(4)			Q_2	Q_3	-
Well (5)	Q_1	Q_2	Q_3	-			Q_1	Q_2	Q_3	Q_4	Q_5	Q_6
Closed battered metal	Q_1	Q_2		Q_2		Q_3	(4)			Q_3	Q_4	Q_4
Prefabricated conc casing	Q_1	Q_2		Q_3			(4)			Q_3	Q_4	Q_4
Moulded casing	Q_1	Q_2		Q_2		Q_3	Q_1	Q_2	Q_3	Q_3	Q_4	-
Embedded casing	Q_1	Q_2		Q_3		Q_4	(4)			Q_3	Q_4	-
Low pressure injection	Q_1	Q_2		Q_3			Q_2	Q_3	Q_4	Q_5		-
High-pressure injection (6)		Q_4	Q_5	Q_5		Q_6	-	Q_5	Q_6	Q_6		Q_7 (7)

(1) Re-boring (3) and grooving at the end of drilling.
(2) Long piles (over 30 m).
(3) Dry drilling, tube not swaged.
(4) In the case of chalk, lateral friction may be very low for certain types of pile. It is advisable to carry out a specific study in each case.
(5) Without casing or dark lost ferrule (rough walls).
(6) Selective and repetitive injection at low flow rates.
(7) Selective and repetitive injection at low flow rates and prior treatment of fissured or fractured massifs with sealing of cavities.

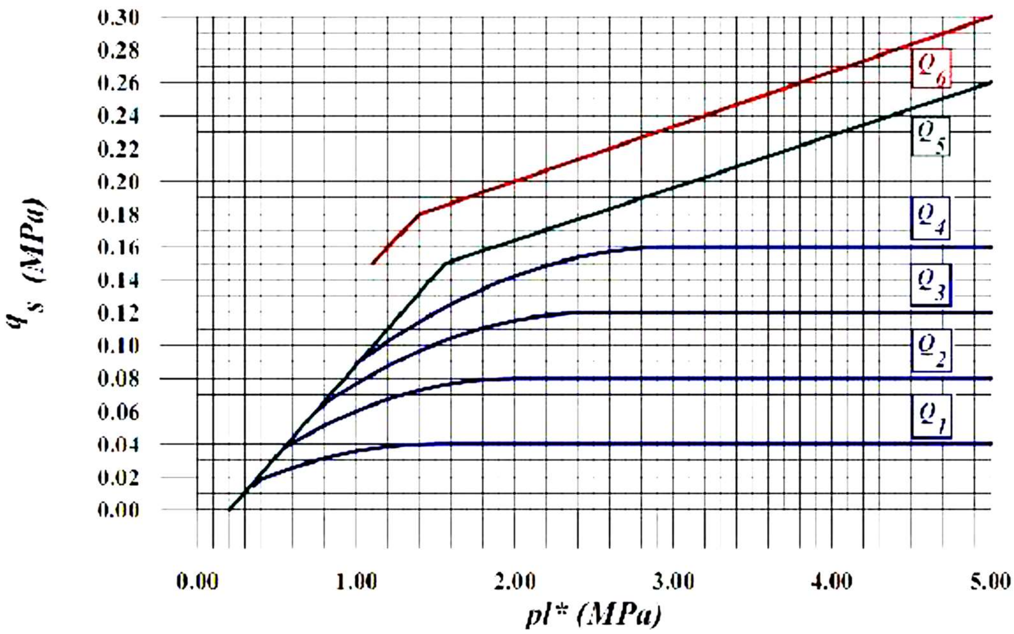


Figure 38 : Lateral friction curves

7. Elastic pile settlement

The elastic settlement of the piles is calculated as follows:

$$S = S_1 + S_2 + S_3$$

Where :

S: Total elastic settlement of the pile.

S_1: Elastic settlement of the pile.

S_2: Elastic settlement due to the tip of the pile.

S_3: Elastic settlement due to load transmission along the pile.

- Calculation of settlement S1 :

The material constituting the pile is assumed to be elastic (i.e. behaviour in the elastic range).

The elastic settlement of the pile is given by the following equation:

$$S_1 = \frac{Q_{wp} + \xi Q_{ws}}{A_p . E_p} . L$$

Where :

Q_{wp}: The load supported by the tip of the pile.

Q_{ws}: Load due to pile friction.

A_p: Cross-sectional area of the pile.

L: Length of the pile.

E_p: Young's modulus of the pile material.

The value of ξ depends on the distribution of frictional resistance along the pile (Figure 39).

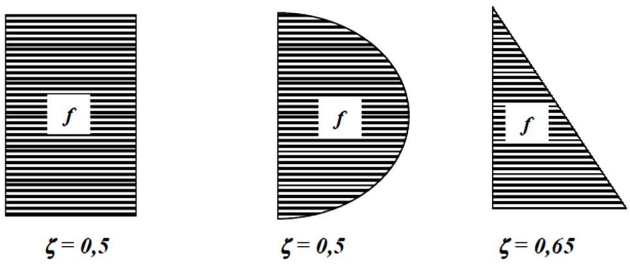


Figure 39 : Unit friction distribution along the pile

- **Calculation of settlement S2** :

The elastic settlement due to the tip of the pile is calculated using the following equation:

$$S_2 = \frac{q_{wp} D}{E_s} (1 - \nu_s^2) . I_{wp}$$

Where

D: Diameter or width of the pile.

q_{wp}: Stress supported by the tip of the pile, $q_{wp} = Q_{wp} / A_p$.

E_s: Young's modulus of the soil.

ν_s: Fish coefficient of the soil.

I_{wp}: Influence coefficient given by Harr's abacus.

- **Calculation of settlement S3**

Similarly, the settlement due to load transmission along the pile is given by the following equation :

$$S_3 = \frac{q_{ws} D}{P.L.E_s}(1 - \nu_s^2).I_{ws}$$

Where

Q_{ws}: Load due to pile friction.

q_{ws}: Represents the average friction value along the pile, $q_{ws} = Q_{ws} / P.L.$

P: Perimeter of the pile.

L: Anchorage length of the pile.

I_{ws}: Influence coefficient, $I_{ws} = 2 + 0.35\sqrt{\frac{L}{D}}$.

CHAPTER 3

Application exercises

Exercise 1

The footing shown in the figure below (Figure 40) exerts a pressure of 250 kPa on contact with the soil.

Evaluate the ultimate stress for the 2 cases.

1) Strip footing.

2) Square footing.

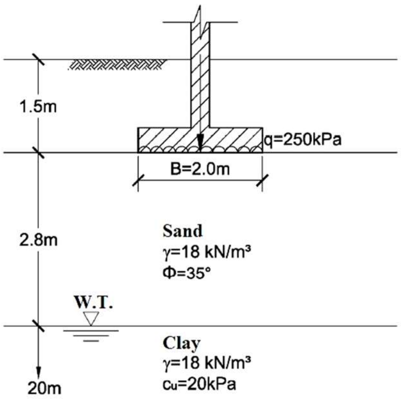


Figure 40 : Vertical cross-section of soil

Solution

Strip footing :

$q_u = (1/2\ \gamma\ BN_\gamma) + (\gamma\ D\ N_q) + (C\ N_c)$

$q_u = (1/2\ \gamma\ BN_\gamma) + (\gamma\ D\ N_q)$

$c = 0$

For $\varphi = 35°$, $Nq=33.3$ $N\gamma=41.1$

$q_u = (1/2\ .18.\ 2.\ 41{,}1) + (18\ .\ 1{,}5\ .\ 33{,}3) = 1638{,}90$ kPa

Square footing :

$q_u = 1/2\ (1\text{- } 0{,}2B/L)\ \gamma\ B.N_\gamma + (\gamma\ D\ N_q) + (1\text{-}02B/L)\ C\ N_c$

$q_u = 1/2\ (1\text{- } 0{,}2B/L)\ \gamma\ B.N_\gamma + (\gamma\ D\ N_q)$

$q_u = 1/2\ (1\text{- } 0{,}2.2/2)\ 18.2.41{,}1 + (18\ 1{,}5.\ 33{,}3) = 1490{,}94$ kPa

Exercise 2

Isolated rigid symmetrical footing subjected to normal forces (connecting rod methods). Use of the homothetic method ($a \approx b$).

Determine the dimensions and reinforcement of the insulated footing under the column subjected to a centred load.

Self-weight of reinforced concrete: $g_0 = 25$ kN/m^3 ; Self-weight of backfill: $g_1 = 20$ kN/m^3.

Data :

Column dimensions : a = 0.40 m; b = 0.30 m

Minimum steel cover : c = 5 cm

Ultimate strength of the soil : q_u = 1.40 MPa

Concrete : f_{c28} = 25 MPa

Steel : f_e = 500 MPa

Permanent loads : G = 300 kN/m

Live load : Q= 183 kN/m

Prejudicial cracking

Solution

$\overline{\sigma}_{soil} = \frac{q_u}{2} = 0{,}7$ MPa

$f_{bu} = \frac{0.85}{1.5} f_{c28}$ = 14.16 MPa

$f_{su} = \frac{f_e}{1.15}$ = 435 MPa

Ultimate load calculation :

Combination of actions N_u = 1.35G + 1.5Q = 680 kN/m

Footing size :

$S \geq \frac{N_u}{\overline{\sigma}_{soil}} =$ 0,972 m^2 = 9720 cm^2

Let a' and b', footing dimensions

Homothetic relation : a'/b' = a/b

a'b' ≥ 9720 cm^2 a' = (a/b)b' $\frac{a}{b} b'^2 \geq 9720$ b' = 85,38 cm a' = 113,84 cm

a' = 120 cm and b' = 90 cm

d ≥ max $[(a' - a)/4; (b' - b)/4]$ = max $[(120 - 40)/4; (90 - 30)/4]$ = max(20, 15)

It is possible to take 20,25 30, 35,…, 60 without exceeding min(b'-b; a'-a) = (60, 80) = 60 cm

we choose to take d = 40 cm

h = 45 cm

Checking stresses :

The depth at which the footing is embedded is given by p_e = 80 cm

Dead weight of the footing : G_0 = a'b'hg_0 = 0,0122 MN

Dead weight of backfill : G_1 = (a'b' – ab) (p_e – h)g_1 = 0,0067 MN

N_u^* = 0,680 + 1,35.(0,0122 + 0,0067) = 0,7055 MN

$\sigma = \frac{N_u^*}{a' \cdot b'} = \frac{0{,}7055}{1{,}20 \, . \, 0{,}90} = 0{,}653 \text{ MPa} < 0{,}7 \text{ MPa}$

Verification of non-punching condition:

$\overline{\sigma}_{soil}$ $= 0{,}7$ Mpa $= 7$ bars $> 6\ bars$

we have h = 45 cm ≥ (a' - a)/2 = (120-40)/2 = 40 cm

Condition verified

NB : the condition of non-punching verification is checked, i.e., $P_u \leq 0{,}045.U_c.h.\frac{f_{cj}f}{\gamma_b}$

with $P_u = N_u^*.\left[1 - \frac{(a+2h)(b+2h)}{a'.b'}\right]$ if and only if $\overline{\sigma}_{soil} > 6$ bars and h < (a' - a)/2

Reinforcement (Figure 41) :

Steel sense a : $A_{sta} = \frac{N_u^*.(a' - a)}{8\ .\ d\ .\ \frac{f_e}{\gamma_s}}$

$A_{sta} = \frac{0{,}7055\ .(1{,}2-0{,}4)}{8\ .\ 0{,}4\ .\ 435} = 4{,}05.10^{-4}\ m^2 = 4{,}05\ cm^2$

Surcharge for prejudicial cracking : 1,1 . 4,05 = 4,46 cm^2

Minimal reinforcement :

1.6cm²/ml for welded mesh or bars FeE500 1,6.0,9 = 1.44 cm^2

Minimal reinforcement (CBA 93) $A_{min} = 0{,}23\ \frac{a'd\ f_{tj}}{f_e} = 0{,}23\ \frac{120.40\ \ 2{,}1}{500} = 4{,}64\ cm^2$

Choice of steels : A_{sta} = 5T12 = 5,65 cm^2 (spacing 20 cm)

Anchoring :

ls = 40□ = 40 .1,2 = 48 cm (RPA zone IIa)

$\frac{a'}{4} = \frac{120}{4} = 30$ cm and $\frac{a'}{8} = \frac{120}{8} = 15$ cm

$l_s > \frac{a'}{4}$ Anchoring with standardized hooks is required

Steel sense b : $A_{stb} = \frac{N_u^*.(b' - b)}{8\ .\ d\ .\ \frac{f_e}{\gamma_s}} = \frac{0{,}7055\ .(0{,}9-0{,}3)}{8\ .\ 0{,}4\ .\ 435} = 3{,}04.10^{-4}\ m^2 = 3{,}04\ cm^2$

Surcharge for prejudicial cracking : 1,1 . 3,04 = 3,34 cm^2

Minimal reinforcement :

1.6cm²/ml for welded mesh or bars FeE500 : 1,6. 1,2 = 1,92 cm^2

Minimal reinforcement (CBA 93)

$f_{tj} = 0{,}6 + 0{,}06.\ f_{cj}$ if $f_{c28} < 60$ MPa $f_{tj} = 2{,}1$ MPa

$A_{min} = 0{,}23\ \frac{b'd\ f_{tj}}{f_e} = 0{,}23\ \frac{90.40\ \ 2{,}1}{500} = 3{,}48\ cm^2$

Choice of steels : $A_{stb} = 4T12 = 4{,}52cm^2$ (spacing 36,67 cm)

It's a big spacing, so we're taking $A_{stb} = 5T12 = 5{,}65cm^2$

Important: In practice, the following minimum reinforcement is used :

Minimum diameter : 12 mm (RPA) and Maximum spacing : 33 cm

Anchoring : $l_s = 40\phi = 40\ .1{,}2 = 48$ cm (RPA zone IIa)

$$\frac{b'}{4} = \frac{90}{4} = 22{,}5 \text{ cm}$$

$$\frac{b'}{8} = \frac{90}{8} = 11{,}25 \text{ cm}$$

$l_s > \frac{b'}{4}$ Anchoring with standardized hooks is required

Skate dimensions :

$e \geq 6\varphi + 6 = 6 \times 1{,}2 + 6 = 13{,}2$ cm

e = (1/3 – 1/2)h = (45/3 ; 45/2) we take e = 20 cm

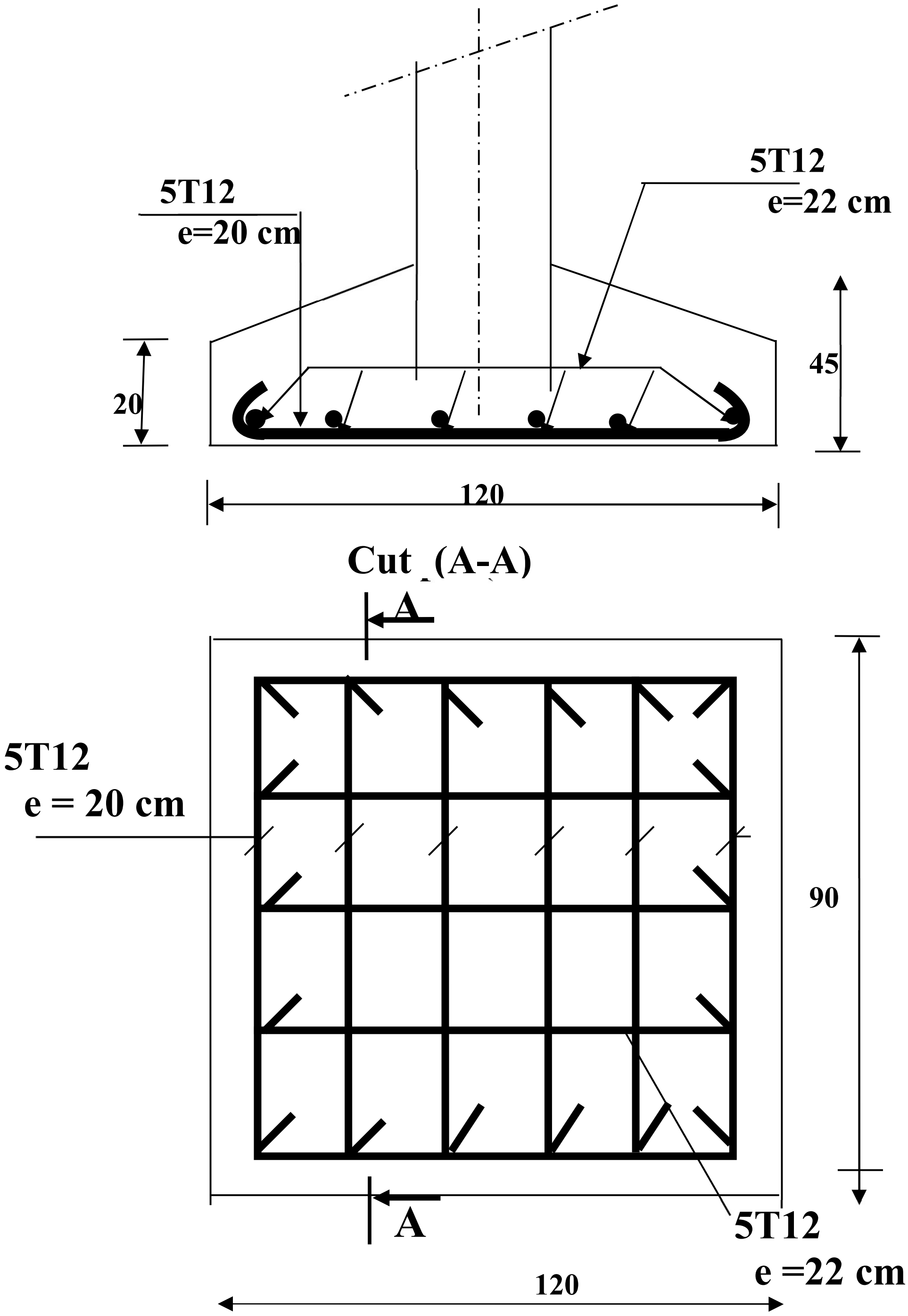


Figure 41 : Foundation reinforcement plan

Exercise 3

Isolated rigid symmetrical footing subjected to normal forces (connecting rod method). Use of the equal overhang method (a>>b).

Determine the dimensions and reinforcement of the footing under the column subjected to a centred load.

Data: Same as Exercise 2 except a = 30 cm and b = 60 cm.

Solution

footing dimensions :

$S \geq \frac{N_u}{\bar{\sigma}_{sol}} =$ 0,972 m^2 = 9720 cm^2

Let a' and b' be the dimensions of the footing

Relation of equal edges : a' – a = b' – b

a'.b' ≥ 9720 cm^2 a' = b' – b + a = b' – 60 + 30 = b' – 30

$b'.(b' - 30) \geq 9720\ cm^2$

Δ = 39780 b' = 114,72 cm a' = 84,72 cm

a' = 90 cm ; b' = 120 cm

d ≥ max $[(a' - a)/4; (b' - b)/4]$ = max $[(90 - 30)/4; (120 - 60)/4]$ = max(15, 15)

We can take 20,25 30, 35,..., 60 without exceeding min(b'-b, a'-a) = (60, 60) = 60 cm

We choose to take d = 40 cm..

h = 45 cm

The rest of the solution to this Exercise is the same as Exercise 2, except that a' = 90 cm instead of 120 cm and vice versa.

Exercise 4

Isolated rigid symmetrical footing subjected to normal forces and bending moments (connecting rod methods). Use of the homothetic method (a ≈ b).

Data

Column dimensions : a = b = 0.55 m

Minimum steel cover : c = 5 cm

Concrete : f_{c28} = 25 MPa

Steel : f_e = 400 MPa

Prejudicial cracking

ELS : N_{ser} =1203,52 kN M_{ser} = 5.60 kN.m

ELU : N_u = 1657,025 kN M_u = 7,728 kN.m

$\bar{\sigma}_{soil}$ =2,5 bars

N.B: If N >> M you can size with $a'.b' \geq \frac{N}{\sigma_{soil}}$

If M >> N you use the rule of the central third, i.e., $a' \geq 6.e$ with $e = \frac{M}{N}$, but the exact method involves solving a third-degree equation.

Solution

Determining footing dimensions :

$\frac{A}{B} = \frac{a}{b} \quad \Rightarrow \frac{A}{B} = 1 \quad \Rightarrow A = B$

$$e_0 = \frac{M_{ser}}{N_{ser}} = \frac{5.60}{1203.52} = 0.005\ m$$

Rigid footing (M. Connecting rods) $\sigma = \frac{N_{ser}}{A.B}(1 + \frac{3e_0}{A}) \leq 250\ MPa$

$\frac{N_{ser}}{A^2} + \frac{3e_0 N_{ser}}{A^3} \leq 250\ MPa \quad \Rightarrow \quad 250A^3 - N_{ser}A - 3e_0 N_{ser} \geq 0$

$250A^3 - 1203.53A - 18{,}0529 \geq 0$ So we adopt : **A = B = 2,3 m**

Note: if you take A = 2.20 m, the equation is not verified (-3.82< 0) but with A = 2.25 m, the equation is verified, so A could be taken to be equal to 2.25 m.

Stability condition "no uplift" :

According to CBA93 we have that :

$e_0 \leq \frac{A}{6} \quad \Rightarrow 0{,}005 \leq 0{,}383$ Condition verified

According to RPA 99 version 2003 (Art10.1.5 - page 81) we have :

$e_0 = \frac{M}{N} = 0{,}129\ m \leq \frac{2{,}3}{4} = 0{,}575$ Condition verified

Rigidity condition :

$$d \geq \max\left\{\frac{A-a}{4} ; \frac{B-b}{4}\right\}$$

$d \geq \max(45\,;45)$

We use : d = 45 cm

$h = d + 5cm = 45 + 5 = 50cm$

h = 50 cm

Checking stresses :

NB: The weight of the footing and soil is neglected in this exercise for calculation purposes only, but in reality it is taken into account.

$\sigma_1 = \left(1 + \frac{6.e_0}{B}\right)\frac{N_{ser}}{AB} =$ 230,27 kN/m² < 250 kN/m²

$\sigma_2 = \left(1 - \frac{6.e_0}{B}\right)\frac{N_{ser}}{AB} =$ 224,75 kN/m² < 250 kN/m²

Reinforcement (Figure 42) :

$\sigma_1 = \left(1 + \frac{6.e_0}{B}\right)\frac{N_{ser}}{AB} =$ 230,27 kN/m^2

$\sigma_2 = \left(1 - \frac{6.e_0}{B}\right)\frac{N_{ser}}{AB} =$ 224,75 kN/m^2

1st method :

$\sigma_{moy}(A/4) = \frac{\sigma_{min}+3\,\sigma_{max}}{4}$ = 226,13 kN/m^2 < $\overline{\sigma}_{soil} = 250\ \text{kN/m}^2$

$\overline{Q} = \sigma\left(\frac{A}{4}\right).A.B$ = 1196,22 kN

$A_X = A_Y = \frac{\overline{\mathbf{Q}}\,.\,(\mathbf{A-a})}{\mathbf{8}\,.\,\mathbf{d}\,.\,\frac{\mathbf{f_e}}{\mathbf{\gamma_s}}}$ = 16.71cm²

2nd method :

$A_X = A_Y = \frac{N\times(A-a)}{8\times d\times f_{su}}\ \frac{1203{,}52\,.(2{,}3-0{,}55)}{8\,.0{,}45\,.348\,.10^{-1}}$ =16,81cm²

NB: it is advisable to work with the 1st method, as we have used the rigid footing by the connecting rod method as a hypothesis.

Surcharge for prejudicial cracking : 1,1 . 16,71 = 18,38 cm^2

Minimal reinforcement :

2 cm² for welded mesh or bars FeE400 : 2.2,3 = 4,60 cm^2

The minimum section of CBA 93 :

$A_{min} = 0{,}23\ \frac{A.d.f_{tj}}{f_e} = 0{,}23\ \frac{230.45.2{,}1}{400} =$ 12,48 cm^2

We adopt 13HA14 = 20.02 cm2 (spacing = 18.33 cm)

Calculation of the free height of the skate e:

e ≥ 6 ϕ + 6 cm =14.4 cm

e = (1/3 – 1/2) h = (50/3; 50/2) = (16.67 ; 25)

we take e = 20 cm

Verification of the punching verification condition :

$\overline{\sigma}_{soil} = 0{,}25\ \text{MPa} = 2{,}5\ \text{bars} < 6\ \text{bars}$

Condition verified

Anchoring :

l_s = 40ϕ = 40 .1,4 = 56 cm (RPA zone IIa)

$$\frac{A}{4} = \frac{230}{4} = 57{,}5\ \text{cm}$$

$$\frac{A}{4} = \frac{230}{8} = 28{,}75\ \text{cm}$$

$\frac{A}{8} < l_s < \frac{A}{4}$: Straight anchoring across the entire width of the footing

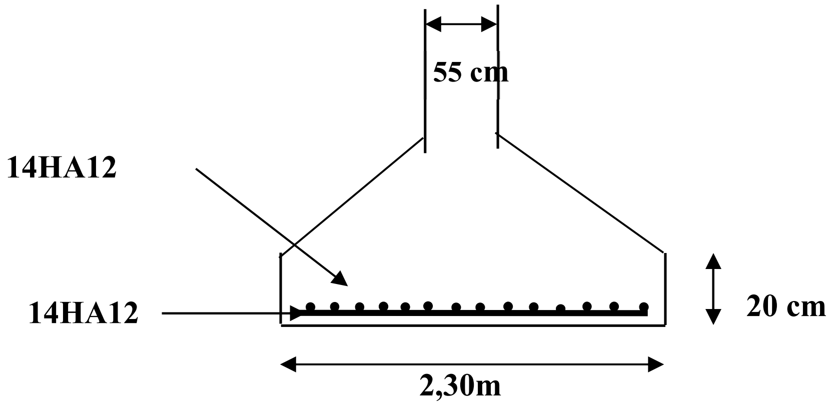


Figure 42 : Drawing of reinforcement

Exercise 5

A footing on homogeneous soil at - 1 m, supporting a 0.20 m wide wall (b = 0,20 m). Pre-dimension and reinforce this footing.

Data :

Ultimate load at the base of the wall P_u = 0.22 MN/m

$\overline{\sigma}_{soil}$ = 0,175 MPa

Concrete f_{c28} = 25 MPa

Density of soil : 18 kN/m^3

Prejudicial cracking

Solution

L = 1 m

b' : Width of footing

$b' \geq \frac{P_u}{\overline{\sigma}_{soil}} \frac{0,22}{0,175} = 1,25 \text{ m}$ we take **b' = 1,50 m**

$\frac{b' - b}{4} \leq d \leq b' - b$

0,33 m $\leq$ d $\leq$ 1,30 m

d = 0,45 m and h = 0,45 + 5 = **0,5 m**

g_0 : Dead weight of the footing and dead weight of the soil above it.

g_0 = 0,0305 MN/m

Pu +1,35g_0 = 0,261 MN/m

Non-punching verification :

$\overline{\sigma}_{soil} = 0,175 \text{ Mpa} = 1,75 \text{ bars} < 6 \; bars$

Condition verified

Reinforcement :

Steel in transverse direction : $As = \frac{N_u(b'-b)}{8d\, f_{su}}$

For L = 1 m Nu = 0,261. 1 = 0,261 MN

$A_s = \frac{Nu \times (a'-a)}{8 \times d \times \sigma_s} = \frac{0{,}261.(1{,}5-0{,}2)}{8\ .0{,}45\ .348}.10^4 = 2{,}\ 71\ cm^2$

Increase for prejudicial cracking : 1,1 . 2,44 = 2,98 cm^2

Minimal reinforcement :

2 cm^2/ml for bars FeE400

So we need to take the minimum section of CBA 93

$A_{min} = 0{,}23\ \frac{a'd\, f_{tj}}{f_e} = 0{,}23\ \frac{150.45\ \ 2{,}1}{400} = 8{,}15\ cm^2$

Choice of steel : $As = 6T14 = 9{,}24cm^2$ (spacing 20 cm)

Anchoring :

ls = 40ϕ = 40 .1,4 = 56 cm (RPA zone IIa)

$\frac{b'}{4} = \frac{150}{4}$ =37,5 cm

$\frac{b'}{8} = \frac{150}{8}$ =18,75 cm

$l_s > \frac{b'}{4}$ Anchoring with standardized hooks is required

Running in the longitudinal direction footing:

$A_{st} = \frac{A_s}{4}. = 2{,}31\ cm^2$

Increase for prejudicial cracking : 1,1 . 2,31 = 2,54 cm^2

minimum chaining : 2 cm^2 for bars FeE400: 2 x 1,5 = 3 cm^2

Choice of steels :

$A_{st} = 3T12 = 3{,}39cm^2$ (spacing 70 cm)

It's a big spacing, so we're taking $A_{stb} = 6T12 = 6{,}78cm^2$ (spacing 28 cm)

Skate dimensions :

$e \geq 6\varphi + 6 = 6 \times 1{,}4 + 6 = 14{,}4$ cm

e = (1/3 – 1/2)h = (50/3, 50/2) we take e = 20 cm

Exercise 6

A footing on homogeneous soil and under a wall subjected to normal forces and bending moments (connecting rod method).

The footing is founded at -1 m, supporting a 0.20 m wide wall. Pre-dimension and reinforce this footing.

Data :

Nu = 0,22 MN/ml ; Mu = 0,01 MN.m/ ml ; $\overline{\sigma}_{soil}$ = 0,175 MPa

Concrete f_{c28} = 25 MPa

Density of soil : 18 kN/m^3

Prejudicial cracking of the footing

Solution

L = 1 m and b' : Width of footing

$e_0 = \frac{0,01}{0,22} = 0.045\ m$ $\quad \frac{N_u}{A}\left(1 + \frac{3.e_0}{A}\right) \leq \overline{\sigma}_{soil}$ $\quad \frac{0,22}{A}\left(1 + \frac{3\ .\ 0,045}{A}\right) \leq 0,175$

- 0,175 A^2 + 0,22 A + 0,0297 ≤ 0 $\quad \Delta$ = 0,06919 so A =1,38 m

So we adopt : **A = 1,5 m**

$\frac{A-a}{4} \leq d \leq A - a$ $\quad \frac{150-2}{4} = 32,5\ cm \leq d \leq 150 - 20 = 130\ cm$

d = 0,45 m and h = 0,45 + 5 = **0,5 m**

g_0 : Dead weight of the footing and dead weight of the soil above it.

g_0 = 0,0305 MN/m

Nu = 0,261 MN/m

Checking stresses :

$e_0 = \frac{0,01}{0,261} = 0,038\ m$

$\sigma_1 = \left(1 + \frac{6.0,038}{1,5}\right)\frac{261}{1,5}$ = 200,448 kN/m^2 < 350 kN/m^2

$\sigma_2 = \left(1 - \frac{6.0,038}{1,5}\right)\frac{261}{1,5}$ = 147,552 kN/m^2 < 350 kN/m^2

For the ELU checks, we take the ultimate stress of the soil, which is $q_u = 2\ \overline{\sigma}_{soil}$ and in ELS, we check with $\overline{\sigma}_{soil}$.

Non-punching verification :

$\overline{\sigma}_{soil}$ = 0,175 MPa = 1,75 bars < 6 bars $\quad$ Condition verified

Stability condition "No uplift" :

According to CBA93 :

$e_0 \leq \frac{A}{6} \Rightarrow 0.038 \leq 0,25$ $\quad$ Condition verified

According to RPA 99 :

According to RPA 99 version 2003 (Aart10.1.5) we have :

$e_0 = 0.038\ m \leq \frac{1,5}{4} = 0.375\ m$ $\quad$ Condition verified

Reinforcement :

$\sigma_{moy}(A/4) = \frac{\sigma_1+3\sigma_2}{4} = 187.224$ kN/m^2

$\sigma_{moy} = 226.13$ kN/m^2 $< q_{su} = 350$ kN/m^2

Q'= $\sigma\left(\frac{A}{4}\right) \times A \times B = 280,.84$ kN

$A_s = \frac{Q' \times (A-a)}{8 \times d \times \sigma_s}$ = 2,91 cm^2

Increase for prejudicial cracking : 1,1 . 2,91 = 3,2 cm^2

Minimal reinforcement :

2 cm^2/m for bars FeE400

The minimum cross-section according to CBA 93

$A_{min} = 0{,}23\ \frac{Ad\ f_{tj}}{f_e} = 0{,}23\ \frac{150.45\ \ 2{,}1}{400} =$ 8,15 cm^2

We adopt 8HA12 = 9.04 cm^2

Calculating the free height of the skate e :

e $\geq$ 6 ϕ + 6 cm = 13,2 cm

e = (1/3 – 1/2)h = (50/3, 50/2) = (16,67, 25) we take e = 20 cm

Anchoring :

ls = 40ϕ = 40 .1,2 = 48 cm (RPA zone IIa)

$\frac{A}{4} = \frac{150}{4}$ =37,5 cm

$\frac{A}{8} = \frac{150}{8}$ =18,75 cm

$l_s > \frac{A}{4}$ Anchoring with standardized hooks is required

Running in the longitudinal direction footing :

$A_{st} = \frac{A_s}{4}.$ = 2,26 cm^2

Increase for prejudicial cracking : 1,1 . 2,31 = 2,49 cm^2

Minimum chainage :

2 cm^2/m for bars FeE400 either 2 x 1,5 = 3 cm^2

Choice of steels : $A_{st} = 3T12 = 3{,}39\ cm^2$ (spacing 70 cm)

It's a big spacing, so we're taking $A_{st} = 6T12 = 6{,}78\ cm^2$ (spacing 28 cm)

In practice, the minimum diameter is 12 mm (RPA99 Version 2003) and the maximum spacing is 33 cm.

Skate dimension :

$e \geq 6\ \varphi + 6 = 6 \times 1{,}2 + 6 = 13{,}2$ cm

e = (1/3 – 1/2)h = (50/3, 50/2) we take e = 20 cm

Exercise 7

The footing subject to the following loads P_1 = 0,85 MN; P_2 = 1,15 MN; P_3 = 1,76 MN and P_4 = 0,90 MN (Figure 43). Pre-dimension the footing.

We give :

$\overline{\sigma}_{soil} = 3$ bars

Column dimensions : 40 x40 cm^2

Concrete : f_{c28} = 25 MPa ; Steel : f_e = 400 MPa

Density of soil : 18 kN/m^3

Footing founded on – 1,50 m

Prejudicial cracking of the footing

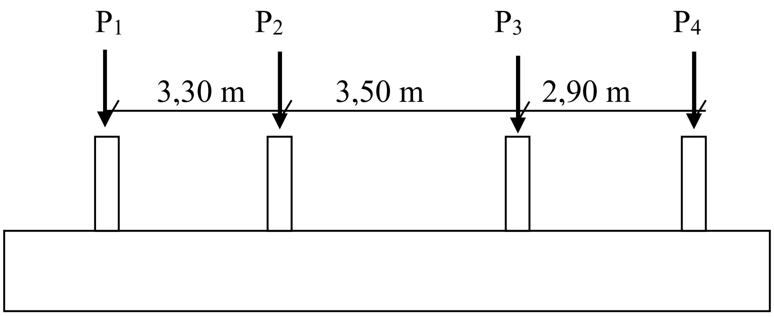


Figure 43 : Elevated section of the footing

Solution

$$\sum P_i = 4{,}66 \text{ MN}$$

The resultant of the loads (Figure 44) is located at a distance from P1 equal to :

$$\frac{1{,}15\,.\,3.3 + 1{,}76\,.\,6{,}8 + 0{,}9\,.\;9{,}70}{0{,}85 + 1{,}15 + 1{,}76 + 0{,}9} = \frac{24{,}493}{4{,}66} = 5{,}256 \text{ m}$$

It is assumed that the edge equal to 0.50 m

L = 2 (5,256 +0,5) = 11,512 m = 11,51 m

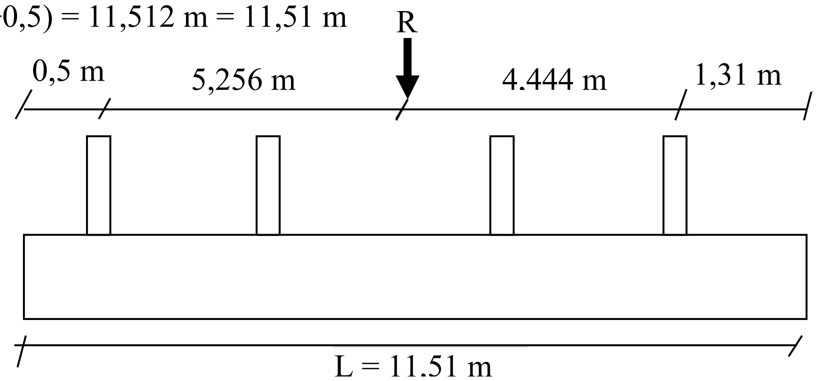


Figure 44 : Position of the resultant force

Width of footing B is equal to :

$$B = \frac{\sum P_i}{\overline{\sigma}_{soil} . L} = \frac{4,66}{0,3 .\ 11,51} = \mathbf{1,35\ m}$$

Height of footing :

M. connecting rods : $\frac{B-b}{4} \leq d \leq B-b \qquad \frac{135-40}{4} \leq d \leq 135-40$

$23,75$ cm $\leq d \leq 95$ cm ; So

$28,75$ cm $\leq h_1 \leq 100$ cm

Rigid footing : $h_2 \geq (48K\ L^4_{max} / E\ \pi^4 B)^{1/3}$

$h_2 \geq (48.4.10^4 .3,5^4 /3,16.10^7\ \pi^4 1,35)^{1/3} = 0,411$ m = 41,1 cm

E : Modulus of elasticity of concrete = 3,16 10^7 kN/m^2

K : Coefficient of stiffness of the soil = 40000 kN/m^2

L_{max} : The largest distance between posts in a post string

Flat-rate condition :

$\frac{L_{max}}{9} \leq h_3 \leq \frac{L_{max}}{6} \qquad \frac{350}{9} \leq h_3 \leq \frac{350}{6} \qquad 38,89$ cm $\leq h_3 \leq 58,33$ cm

The height h is that which has the maximum value and verifies the three previous conditions.

$$h = \text{Max}\ (h_1, h_2, h_3) = \mathbf{50\ cm}$$

$N_S^* = 0,025 . 11,51 . 0,50 . 1,35 + 0,018 . 1 . (1,35. 11,51 - 0,4.0,4.4) + 4,66 = 5,1$ MN

Transversally: connecting rod methods (most unfavorable column)

$\sigma = \frac{N_S^*}{S} = \frac{5,1}{1,35 .\ 3,20} = 0,33$ MPa $> \overline{\sigma}_{soil}$ Unverified

We therefore need to resize.

Increase B; then B = 1.5 m

The height of the footing (connecting rod method) :

$\frac{150-40}{4} \leq d \leq 150-40$ So $\quad 32,5$ cm $\leq h_1 \leq 115$ cm

Rigidity condition :

$h_2 \geq (48.4.10^4 .3,5^4 /3,16.10^7\ \pi^4 1,5)^{1/3} = 0,40$ m = 40 cm

Standard condition :

$\frac{350}{9} \leq h_3 \leq \frac{350}{6} \qquad 38,89$ cm $\leq h_3 \leq 58,33$ cm

The height h remains $h = \text{Max}(h_1, h_2, h_3) = 50$ cm

$N_S^* = 0,025 . 11,51 . 0,50 . 1,50 + 0,018 . 1 . (1,50. 11,51 - 0,4.0,4.4) + 4,66 = 5,17$ MN

$\sigma = \frac{N_S^*}{S} = \frac{5,17}{1,50 .\ 11,51} = 0,30$ MPa $= \overline{\sigma}_{soil}$ Condition verified

The load / ml :

$q = \overline{\sigma}_{soil} . B = 0,3 . 1,5 = 0,45$ MN/m

The most heavily loaded column is P3 :

$N_3^* = 0{,}025 \,.\, 3{,}2 \,.\, 0{,}50 \,.\, 1{,}50 + 0{,}018 \,.\, 1 \,.\, (1{,}50.\ 3{,}2 - 0{,}4.0{,}4) + 1{,}76 = 1{,}89$ MN

Non-punching verification :

$\overline{\sigma}_{soil} = 0{,}3 \text{ Mpa} = 3 \text{ bars} < 6 \text{ bars}$ Condition verified

Reinforcement :

$$A_{st} = \frac{N_3^*(B-b)}{8.d.f_{su}} = \frac{1{,}89.(1{,}5-0{,}4)}{8.0{,}45.348}.10^4 = 16{,}59 \text{ cm}^2$$

Increase for prejudicial cracking : 1,1 . 16,59 = 18,25 cm^2

Minimal reinforcement :

2 cm²/ml for bars FeE400

Minimal reinforcement (CBA 93) :

$$A_{min} = 0{,}23\ \frac{Bd\ f_{tj}}{f_e} = 0{,}23\ \frac{150.45.2{,}1}{400} = 8{,}15 \text{ cm}^2$$

Choice of steels : $A_{st} = 12T14 = 18{,}47 \text{cm}^2$

Anchoring :

ls = 40ϕ = 40 .1,4 = 56 cm (RPA zone IIa)

$$\frac{B}{4} = \frac{150}{4} = 37{,}5 \text{ cm}$$

$$\frac{B}{8} = \frac{150}{8} = 18{,}75 \text{ cm}$$

$l_s > \frac{B}{4}$ Anchoring with standardized hooks is required

Longitudinally: The 3-moment or Caquot method can be applied.

Exercise 8

Study the foundations of the building below (Figure 45).

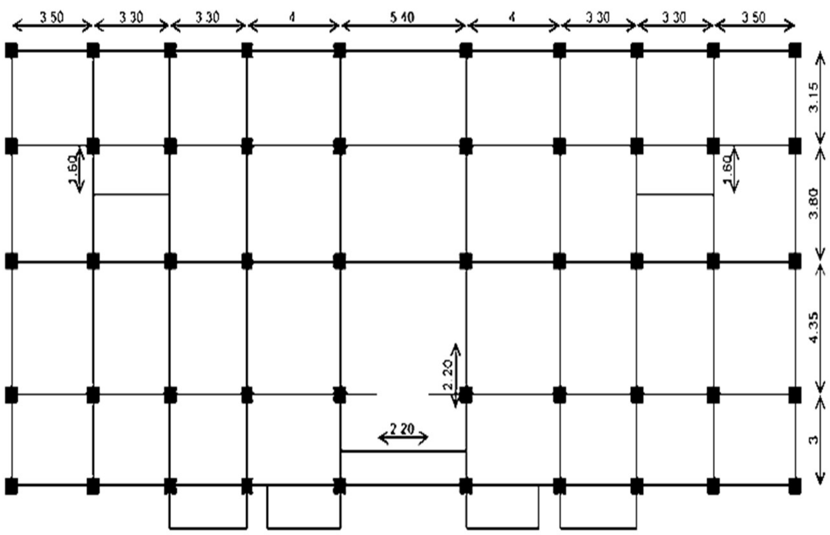


Figure 45 : Plan section of a building

Data :

Columns : 50 x 50 cm^2

Building surface : S_b = 480,48 m²

Most solicited column : N_u = 267,5 t

$\bar{\sigma}_{soil} = 2$ bars

f_{c28} = 25 MPa ; FeE 400; prejudicial cracking.

Infrastructure depth : (h = 1,5 m)

E =32164195 kN/m^2.

K=40000 kN/m^3 (Soil stiffness coefficient)

Total loads absorbed by the raft : Q = 613,80 t and G = 4834,73 t

Solution

The first thing we are proposing is a strip footing, so we are going to carry out a small check like this:

The surface area S_s of the strip footing is given by :

$S_s \geq \frac{N}{\bar{\sigma}_{soil}}$ $\bar{\sigma}_{soil}$ = 2 bars = 20 t/ m^2 with, N is the sum of the loads per row

If the surface area of the footings is greater than 50% of the total surface area of the building (Ss / S_b > 50 %) then a raft is chosen.

In our case, Ss / S_b > 50 %, o we choose a ribbed raft.

$$S_s = \frac{\sum P_i}{\bar{\sigma}_{soil}} = \frac{4834{,}73 + 613{,}8}{0{,}2} = \frac{5448{,}53}{20} = 272{,}43 \text{ m}^2$$

In our case, $\frac{S_s}{S_b} = \frac{272{,}43}{480{,}48} = 0{,}57 > 50\%$, then a ribbed raft is chosen.

Pre-dimensioning the raft :

Thickness of the raft :

Shear strength condition :

The thickness of the raft will be determined according to the shear stress of the raft.

According to CBA93 regulations (Art. A.5.1).

$$\tau_u = \frac{V_u}{b.d} \leq 0.07 f_{cj}/\gamma_b$$

with :

Vu : Shear force in relation to the ELU.

b : Width of the raft.

γ_b : 1.15 in an accident situation and 1.5 in a normal situation.

d = 0.9 h and b = 1m

L_{max} : The longest span of the slab = 5,4 m.

$$h_1 \geq \frac{N_u \times Lb_{max}}{0.9 \times 2S \times 0.07 f_{cj}}$$

For :

$$\begin{cases} N_u = 613{,}80.\ 1{,}50 + 4834{,}73.1{,}35 = 7447{,}59 \text{ t} \\ S = 480{,}48 \text{ m}^2 \\ L_{max} = 5{,}4 \text{ m} \\ \gamma_b = 1{,}5 \\ f_{cj} = 25.10^{+2} \text{ t/m}^2 \end{cases}$$

$h_1 \geq 39{,}86$ cm

Standard condition :

The thickness of the raft must satisfy the following condition :

$L_{max}/9 \leq h_2 \leq L_{max}/6$

With

L_{max} : The maximum distance between two successive sails is equal to =5.4 m.

Hence :

60 cm $\leq h_2 \leq$ 90 cm.

Punching verification :

Under the action of localised forces, the resistance of the raft to punching by shear force must be checked.

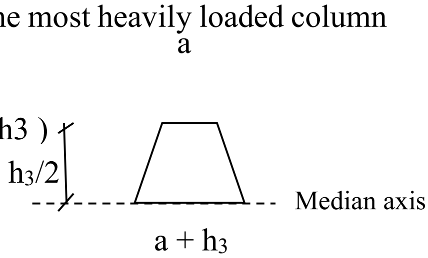
This check is carried out as follows :

$N_u \leq 0.045.\mu_c.h3.f_{c28}/\gamma_b$ (Art 5.2.4 of the CBA93)

Nu : Calculated load with respect to the ultimate limit state of the most heavily loaded column

Nu = 267,5 t

μ_c : Perimeter of the sheared storyteller = 4(a + h3) = 4 (0,5 + h3)



f_{c28} = 25 MPa = 2500 t/m^2

$Nu \leq 0.045 . \mu_c . h3 . f_{c28} / \gamma_b$

$Nu \leq 300 (0{,}5 + h_3).h_3$

$h_3 \geq 66{,}83$ cm

Finally : **$h_r = \max(h_1, h_2, h_3) = 70$cm**

Determining forces

The actions :

Exploitation loads : Q = 613,80 t

Permanent loads on the raft : G = 4834,73 t

Solicitations:

ELU : $N_U = 1,35N_G + 1,5N_Q = 7447,58$ t

ELS: $N_S = N_G + N_Q = 5448,53$ t

Minimum raft surface area :

The surface of the raft must meet the following conditions :

$$\sigma_{soil} = \sigma_{adm} > \frac{N_s}{S}$$

ELS σ_{adm}= 2 bars =20 t/m^2 At the base of the building:

$N_s / S_b = (N_{raft} + N_{building})/S = (G_{raftr} + Q + G_{building})/S$

with :

The total surface area of the building is : $S_b = 480,48$ m²

The total right-of-way with an edge :

d (edge) ≥ max (h / 2 ; 30 cm) = 50 cm

$S = S_{building} + S_{edge} = (33,6 +1).(14,3 +1) = 15,3 \times 34,6 = 529,38$ m²

G_r (raft) = 926,415 t

$G_{total} = 926,415 + 4834,73 = 5761,145$ t

Ns = 5761,145 + 613,80 = 6374,945 t

$Ns / S = (G_b + Q + 2,5\times0,7\times529,38) / S = 10,99$ t/m² < σ_{adm}

ELU : At the base of the building

$Nu / S = (1,35\times G_{raft} + 1,5\times Q + 1,35\times G_{building})/S$

Nu = 5761,145.1,35 + 613,80.1,5 = 8698,246 t

Nu / S = 16,43 t/m² < σ_{adm} Verified condition

Checking under the effect of hydrostatic pressure :

It is necessary to check the raft under the effect of hydrostatic pressure to ensure that the building is not lifted by it. This is done by checking that : $W \geq Fs . \gamma . Z . S$

With

W : Total weight of building at base of raft

$W = W_{raft} + W_{building}$ = 926,415 t + 5448,53 t = 6374,95 t

Fs : Safety coefficient against uplift, Fs = 1,5

γ : Water density (γ = 1t / m^3)

Z : Infrastructure depth (h = 1,5 m)

S : Surface of the raft, (S = 529,38 m)

$Fs . \gamma . Z . S = 1,5\times1\times1,5\times529,38 = 1191,105$ t

So,

W= 6374,95 t ≥ Fs . γ . Z . S = 1191,105 t Verified condition

Checking stability (Tables 7,8 and 9) :

Under the effect of horizontal loads (seismic forces), a reversing moment develops. To this end, the ends of the raft must be checked :

- Tensile stress (uplift) with the combination 0,8G ±E.
- Maximum compressive stress with the combination G + Q + E.

Checking for lifting : 0,8 G ± E

To carry out the verification in question, we need to define the following :

$$\sigma_1 = \frac{N_s}{S} + \frac{MV}{I} ; \qquad \sigma_2 = \frac{N_s}{S} - \frac{MV}{I} ; \qquad \sigma_{moy} = \frac{3\sigma_1 + \sigma_2}{4}$$

Table 7 : Checking the no-lift condition

	0,8G + E		0,8G - E	
	Longitudinal	Transversal	Longitudinal	Transversal
N (t)	3613,07	3611,11	3613,13	3615,09
M (t.m)	2778,57	2323,31	2953,51	8055,40
V (m)	17,3	7,65	17,3	7,65
I (m^4)	10326,88	52812,71	10326,88 $< \sigma_{adm}$	52812,71
σ_1	11,48	7,15	11,77	7,99
σ_2	2,17	6,48	1,87	5,66
σ_{moy}	9,15	6,99	9,3	7,41
σ_{adm}	20	20	20	20
Condition	Verified	Verified	Verified	Verified

Table 8 : Compression check G + Q + E

	G + Q + E	
	Longitudinal	Transversal
N (t)	5087,93	5085,97
M (t.m)	4136,22	965,66
V (m)	17,3	7,65
I (m^4)	10326,88	52812,71
σ_1	7,16	16,54
σ_2	6,48	2,68
σ_{moy}	6,99	13,08
σ_{adm}	20	20
Condition	Verified	Verified

Rollover stability check : 0, 8G ± E

It is insured if : $e = \frac{M}{N} \leq \frac{L}{4}$ (RPA99 V2003 Art. 10.1.5)

The results are displayed in the following Table (Table 9):

Table 9 : Checking rollover stability

	0,8G+E		0,8G-E	
	Longitudinal	Transversal	Longitudinal	Transversal
N (t)	3613,07	3611,11	3613,13	3615,09
M (t.m)	2778,57	2323,31	2953,51	8055,40
e = M/N	0,77	0,64	0,82	2,23
L/4	1,35	1,09	1,35	1,09
Condition	Verified	Verified	Verified	Verified

Reinforcement of the raft :

The raft acts like an inverted floor, supported by columns and ribs, and is subjected to uniform pressure from the structure's own weight and from surcharges.

Therefore, we can refer to the methods given by reinforced concrete.

Calculation method :

The raft comprises slab panels supported on 4 sides subjected to a uniformly distributed load.

The moments in the slabs are calculated for a strip of unit width and have the following values :

- In the short-span direction : $M_x=\mu_x.q.L_x^2$
- In the long-span direction : $M_y=\mu_y.M_x$

The values of μ_x , μ_y depend on ($\alpha = L_x/L_y$)

For the calculation, it is assumed that the panels are partially embedded at the support levels, from which the span and support moments are deduced.

- If the panel in question is continuous beyond the supports (intermediate panel)
  - Moment in span : ($M_{tx}=0,75.M_x$; $M_{ty}=0,75.M_y$)
  - Moment on support : ($M_{ax}=0,5.M_x$; $M_{ay}=0,5.M_y$)
- If the panel in question is an edge panel
  - Moment in span : ($M_{tx}=0,85.M_x$; $M_{ty}=0,85.M_y$)
  - Moment on support : ($M_{tx}=0,3.M_x$; $M_{ty}=0,3.M_y$)

The calculation will be made for the most stressed panel only. This has the dimensions shown in the attached diagram, where L_x is the smallest dimension.

The ratio of the smallest dimension of the panel to the largest dimension must be greater than 0.40.

Evaluation of loads and excess loads on the most heavily loaded panel :

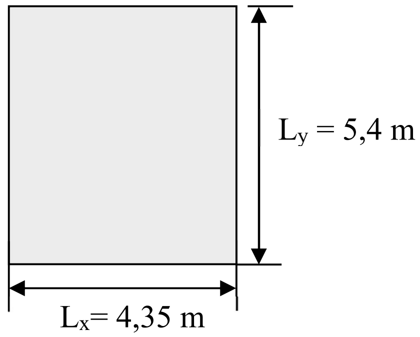


Figure 46 : The busiest panel

<u>ELU</u>

$$q_u = \frac{N_U}{S_{rod}} = \frac{8698,246}{529,38} = 16,431 \text{ t/m}^2$$

$q_u = 164,31\text{kPa}$

<u>ELS</u>

$q_{ser} = 120,42 \text{ kPa}$

The work is per linear metre of length.

$$\alpha = \frac{L_x}{L_y} = \frac{4,35}{5,4} = 0,805$$

$\alpha > 0,4$

ELU :

Direction x-x

$\mu_x = 0,0615$

$M_x = \mu_x \times q \times l_x^2 = 191,21$ kN.m

$M_{tx} = 0,75 \times M_x = 143,41$ kN.m

$M_{ax} = 0,5 \times M_x = 95,61$ kN.m

Direction y-y

$\mu_y = 0,684$

$M_y = \mu_y \times M_x = 130,79$ kN.m

$M_{ty} = 0,75 \times M_y = 98,09$ kN.m

$M_{ay} = 0,5 \times M_y = 65,39$ kN.m

ELS :

Direction x-x

$\mu_x = 0,0615$

$M_x = \mu_x \times q \times l_x^2 = 140,14$ kN.m

$M_{tx} = 0,75 \times M_x = 105,10$ kN.m

$M_{ax} = 0,5 \times M_x = 70,07$ kN.m

Direction y-y

$\mu_y = 0,684$

$M_y = \mu_y \times M_x = 95,85$ kN.m

$M_{ty} = 0,75 \times M_y = 71,89$ kN.m

$M_{ay} = 0,5 \times M_y = 47,93$ kN.m

Calculation of forces and reinforcement at ELU and ELS are summarized in the following tables (Tables 10 and 11).

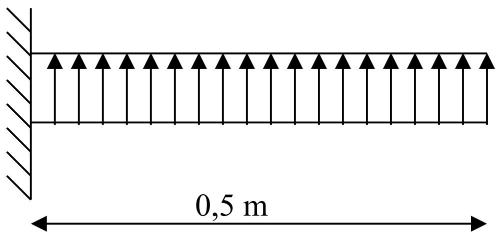
Table 10 : Summary of reinforcement chosen for ELU

	Transverse direction		Longitudinal direction	
	In span	On support	In span	On support
M_u (MN.m)	0,143	0,096	0,098	0,065
μ	0,016	0,011	0,011	0,008
μ < 0.186	Yes	Yes	Yes	Yes
A_s (cm^2/ml)	4,95	3,29	3,37	2,24
A_{smin} (cm^2/ml)	7,60	7,60	7,60	7,60
Choice of bars	6HA14	6HA14	6HA14	6HA14
As adopted	9,24	9,24	9,24	9,24

Table 11 : Summary of reinforcement chosen for ELS

	Transverse direction		Longitudinal direction	
	In span	On support	In span	On support
M_{ser} (MN.m)	0,105	0,07	0,072	0,048
X	0,36	0,36	0,36	0,36
Z	0,51	0,51	0,51	0,51
$\overline{M1}$	1,377	1,377	1,377	1,377
$\overline{M1} > \text{Mser}$	Yes	Yes	Yes	Yes
A_s (cm^2/ml)	17,57	11,767	12,052	8,099
A_{smin} (cm^2/ml)	7,60	7,60	7,60	7,60
Choice of bars	6HA20	6HA16	6HA16	6HA14
A_s adopted	18,85	12 ,06	12,06	9,24

Reinforcement of the edge (Tables 12 and 13) :



$b = 1\text{m}$

$h = 70 \text{ cm}$

$d = 63 \text{ cm}$

$L = 0,5 \text{ m}$

$q_u = 164,31 \text{ kN/m}^2 \times 1\text{ml}$

$q_{ser} = 120,42 \text{ kN/m}^2 \times 1\text{ml}$

$f_{bc} = 14,17\text{MPa}$

ELU : $M_u = \frac{q_u L^2}{2} \Rightarrow M_u = 20{,}539\ kN.m$

Table 12 : Calculated reinforcement of the edge at ELU

M_u (kN.m)	d (m)	μ	Z	A_s (cm^2)
20,54	0,63	0,003	0,629	0,8

ELS : $M_{ser} = \frac{q_{ser} L^2}{2} = 15{,}05\ kN.m$

Table 13 : Calculated reinforcement of the edge at ELS

M_s	D (m)	$\overline{M_1}$	A_s (cm^2)
15,05	0,63	1,102	2,50

Checks :

Non-fragility condition

$A_s \geq A_s^{min}$

$A_s^{min} = 0.23bd\frac{f_{tj}}{f_e}$

$A_s^{min} = 3{,}8cm^2$

Choice of edge bars summarized in Table 14.

Table 14 : Reinforcement adopted for the edge

$A_s(ELU)$ (cm^2)	$A_s(ELS)$ (cm^2)	A_s^{min} (cm^2)	Choice of bars	Spac (cm)
0,8	2,50	3,8	4HA12 $A_s = 4{,}52\ cm^2$	15

Design and reinforcement of the rib :

Rib height : $h_n \geq \frac{L_{max}}{10}$ With L_{max} = 5,4 m

$h_n \geq\ 0{,}54\ m$

Elastic length condition(rigidity) :

$L_e = \left[\frac{4EI}{Kb}\right]^{\frac{1}{4}} \geq \frac{2L_{MAX}}{\pi}$

With :

L_e : Elastic length.

E : Elastic modulus of concrete

E =32164195 kN/m^2.

b : Width of the raft (1 metre strip).

K : Coefficient of soil stiffness per unit area for an average soil.

K=40000 kN/m^3.

where : $hn \geq [3K \times (2L_{Max}/\pi)^4/E]^{\frac{1}{3}}$. $h_n \geq 78{,}23$ cm. We take : $h_n = 80$ cm

For the rib calculation, it is assumed that the foundation is sufficiently rigid to ensure that the stresses vary linearly along the foundation. In this case, the ribs are considered to be supported at the level of the load-bearing elements of the superstructure and loaded below by the reactions of the soil.

The three-moment method is used for continuous beams.

The transmission of loads to the panels follows the triangular and trapezoidal rule (because in this case the centre-to-centre distances are very large i.e., $\rho > 0{,}4$).

Exercise 9

A closed tubular pile 0.3 m in diameter and 10 mm thick is driven by ramming into silty sand to a depth of Z = 18 m. The sand has a density of 18 kN/m^2. Table 15 of the penetration tests (SPT) gives the following values of N (blows/0.3 m).

Table 15 : N values for penetration tests (SPT)

Depth (m)	0	2	4	6	8	10	12	14	16	18	20	22	24
N (blows)		12	15	17	19	23	25	27	30	32	34	36	37

Determine the ultimate bearing capacity of the pile and the ultimate bearing capacity of a group of 4 piles arranged in a portal effect.

Determine the total possible settlement of the group of 4 piles.

Solution

At a peak depth of 18 m N = 32

Beaten pile : m = 400

$A_p = \pi\left(\frac{0{,}3}{2}\right)^2 = 0{,}0707\ m^2$

Bearing capacity of the pile :

$Q_{pl} = m.N.A_p = 905$ kN

Beaten pile : n = 2

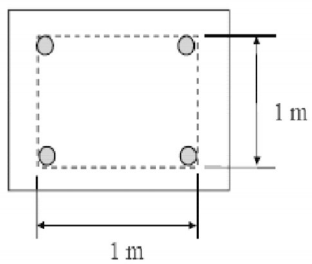
$N_{moy} = (12 + 15 + 17 + 23 + 25 + 27 + 30 + 32)/\ 9 = 22$

$A_s = 2\pi.\left(\frac{0{,}3}{2}\right).1 = 0{,}94\ m^2$

$Q_{sl} = n.N_{moy}.A_s.z = 744{,}5$ kN

$Q_l = Q_{pl} + Q_{sl} = 1649{,}5$ kN

$Q_{adm} = \frac{Q_l}{4} = 412$ kN



Maximum load-bearing capacity of the group :

spacing / diameter = $\frac{1}{0,3}$ = 3,333 < 3,5

$Q_{adm\ (group)}$ = 412 . 4 = 1648 kN

Pile settlement:

$S_{1\ pile} = \frac{D}{100} + 100\ Q_{adm}\frac{Z}{AE}$ = 0,706 cm = 7,06 mm

Pile group settlement :

$S_{group} = S_{1\ pile}\sqrt{\frac{B}{b}}$ = 12,89 mm

Exercise 10

A steel pile 12 m long and 0.2 m in diameter is driven into a clay deposit.

The soil properties are as shown in Figure 47

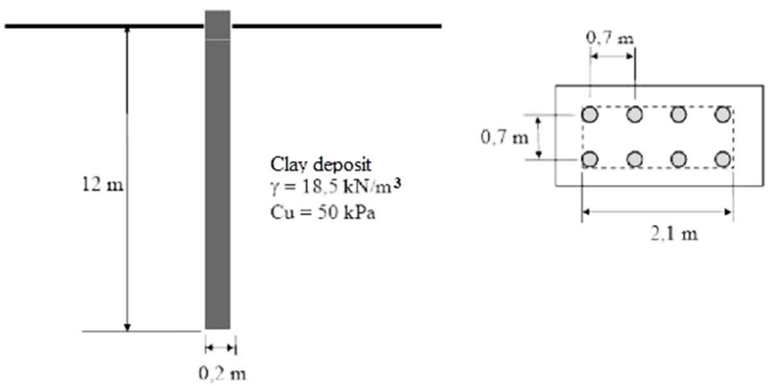


Figure 47 : Elevation and plan section of piles

Determine the ultimate bearing capacity of the pile and the ultimate bearing capacity of a group of 8 piles arranged as shown in Figure 47.

Solution

At its peak, at a depth of 12 m, D = 0,2 m < 0,5 m, so $N_c = 9$

C_U = 50 kPa

$A_p = \pi\left(\frac{0,2}{2}\right)^2$ = 0,0314 m^2

$Q_{pl} = C_U.\ N_c.A_p$ = 14,13 kN

According to the abacus, for a steel pile with $C_U = 50$ kPa :

$\alpha\ .\ C_U = 31$ kPa

$A_s = 2\pi.\left(\frac{0,2}{2}\right).12$ = 7,54 m^2

$Q_{sl} = \alpha\ .\ C_U.A_s$ = 233,74 kN

$Q_l = Q_{pl} + Q_{sl} = 247{,}87$ kN

$Q_{adm} = \frac{Q_l}{3} = 82{,}62$ kN

Group load-bearing capacity :

$C_U = 50$ kPa < 100 kPa

This group of piles consists of 8 piles with :

$Q_{l\,(group)} = 0{,}7.\ 8\ .\ Q_{l\,(pile)} = 1388{,}07$ kN

$Q_{adm\,(group)} = 462{,}69$ kN

Conclusion

The purpose of this handbook of lessons and application exercises is to serve as a guide for students of the master's degree in civil engineering at Abdelhamid Ibn Badis University in Mostaganem and for anyone wishing to gain an overview of surface and deep foundations. Bibliographical references are cited below to enable the reader to delve deeper into the subjects covered in this handout.

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