REPUBLIQUE DU CAMEROUN
****
Paix-Travail-Patrie
****

REPUBLIC OF CAMEROON
****
Peace-Work-Fatherland
****

****
DEPARTEMENT DE GENIE CIVIL
DEPARTMENT OF CIVIL ENGINEERING
****

****
DEPARTMENT OF CIVIL, ARCHITECTURAL AND ENVIRONMENTAL ENGINEERING
****

COMPARATIVE ANALYSIS BETWEEN SOIL IMPROVEMENT METHODS COUPLED WITH RAFTS AND DEEP FOUNDATIONS FOR CONSTRUCTION ON COMPRESSIBLE SOILS: Case of a tall building in the city of Douala

A thesis submitted for the partial fulfilment of the requirements of the degree of Master of Engineering (MEng) in Civil Engineering

Curriculum: **Geotechnical Engineering**

Presented by:

BOUGHA BOUGHA Steve Blondin

13TP20604

Supervised by:

Pr. Simonetta COLA

Academic year : 2019/2020

DEDICATIONS

*I dedicate this work to my precious and lovely mother Mrs **Ngo BOUGHA Jacqueline**.*

All the words will still not be enough to describe all her efforts, sacrifices and daily support in the accomplishment of this work in particular and to overcome my daily challenges in general.

ACKNOWLEDGMENTS

I would like to take this opportunity to express my profound gratitude to the National Advanced School of Public Works, Yaounde and the University of Padova, Italy for this wonderful program. This document is a result of the combined efforts of several individuals I will like to thank here as well.

It is with a sincere and heartfelt gratitude that I thank;

- The **President of the Jury**, for the honour accepting to preside this Jury.
- The **Examiner of the Jury**, for accepting to examine this work. Thanks for the remarks and suggestions made with regards to this work.
- I would like to extend my deep thanks to my supervisors, **Pr. Simonetta COLA** and **Eng. NJANKA NDIZEU Manuela Paule** for their patient guidance, enthusiastic encouragement and useful commentary on this work.
- **Pr. NKENG George Elambo,** Director of the National Advanced School of Public Works (NASPW), Yaoundé, for his implication in my training.
- **Pr. ESOH ELAME** and **Pr. Carmelo MAJORANA**, of the University of Padova, for their work with establishing this partnership with NASPW for the training of young Cameroonians.
- **Pr. Michel MBESSA**, the head of the civil engineering department in the NASPW, for his dynamic guidance and work in the civil engineering training program.
- **All the administrative staff of NASPW,** who devoted themselves unceasingly to inculcate in us qualitative moral values and knowledge in the civil engineering field and services rendered.
- All my **classmates, friends, academic elders** and **juniors** for the support and the memorable time spent together.
- My **family** for the advice and the support during my training.

Also, I sincerely thank all of those who directly or indirectly supported me all this time.

LIST OF ABBREVIATIONS

ASTM = American Society for Testing and Materials

$\mathbf{V_s}$ = volume of solids

$\mathbf{V_p}$ = Volume of pores

Vt = Total volume of soil

$\mathbf{e_o}$ = initial void ratio

$\boldsymbol{\sigma'_o}$ = initial effective stress

$\mathbf{e_1}$ = final void ratio

$\boldsymbol{\sigma'_1}$ = final effective stress

t = time

$\Delta\mathbf{e} = e_o - e_1$

$\Delta\boldsymbol{\sigma'} = \sigma'_1 - \sigma'_o$

A = Cross-sectional area of soil

h = head difference

$\mathbf{h_0}$ = is the value of the head difference h at time t = 0

L = Length

G = shear modulus,

σ = average normal stress equal to the geostatic condition at a depth of D+B/2

$\boldsymbol{\sigma_z}$ = the total vertical stress

$\boldsymbol{\sigma'_{z0}}$ = overburden pressure

$\boldsymbol{\sigma'_z}$ = the effective vertical

δ = frictional angle at the contact pile/soil.

OCR = Over consolidated ratio

Qs = design load transferred as skin friction

Es = Elastic modulus of soil

B = width of footing

q = pressure at the footing base

$\boldsymbol{\mu_0}$ = coefficient function of D/B

μ_1 = coefficient function of H/B and L/B

H = thickness of foundation layer

D = depth of foundation base

Eu = Young modulus in undrained condition

k_p = Coefficient of pressuremetric bearing capacity

k_c = The coefficient of penetrometric bearing capacity

p_{le}^* = The net equivalent limit pressure

q_{ce} = The equivalent tip resistance

q_s = The shaft resistance

i_δ = The reduction coefficient of bearing capacity dealing with the load inclination

i_β = The reduction coefficient of bearing capacity dealing with the presence of an embankment of inclination

NF= French norm

2D= Two dimensions

3D= Three dimensions

ABSTRACT

The growing scarcity of available land resources is one of the decisive factors affecting the choice of the site of a project. We are therefore sometimes constrained to construct some structures with high loads on soils that are problematic such as compressible soils. A current trend over the world, is the use of ground improvement concepts as a complement to raft and as an alternative to or in complement to deep foundations when dealing with compressible soils. The national market is not an exception to the rule. If the design of foundation piles to transmit the structural loads to the ground is now well established notably with the development of the Eurocodes and their National Annexes, the development and way to design ground improvement concepts still need some improvements eventhough they become numerous and noticeable advances are recorded as time advances. The city of Douala as many other cities in Cameroon contains large areas of compressible soils. The major challenge of this work was to make a comparative analysis between the use of two different types of foundations: piles and raft coupled with a system of reinforced soil, for constructing in the city of Douala. The reinforcement used in the course of this analysis is the use of rigid inclusions. In order to face the challenge, the design of the different foundations used was done both from Menard pressuremeter test results and using finite element method for piles and from finite element method for raft reinforced with rigid inclusions. The different finite element methods are made with the use of PLAXIS 2D V20. A comparison between the different systems of foundations was made base on criteria such as the stability, the requirements for the construction of the foundation, the duration of construction and the cost of the foundations. These comparisons lead to the conclusion that rigid inclusions coupled with raft can be used as an alternative solution to piles for the construction of a tall building on a compressible soil.

Key words: compressible soils, foundation, pile, raft, rigid inclusions, tall building

RESUME

La rareté des ressources foncières disponibles est l'un des facteurs déterminants dans le choix du site d'un projet. On est donc parfois contraint de construire certaines structures à fortes charges sur des sols problématiques tels que les sols compressibles. La tendance actuelle dans le monde entier est l'utilisation de concepts d'amélioration du sol comme complément au radier et comme alternative ou complément aux fondations profondes lorsqu'il s'agit de sols compressibles. Le marché national ne fait pas exception à la règle. Si la conception des pieux pour transmettre les charges structurelles au sol est maintenant bien établie notamment avec le développement des Eurocodes et de leurs Annexes Nationales, le développement et la façon de concevoir les concepts d'amélioration du sol ont encore besoin de quelques améliorations même si elles deviennent nombreuses et que des avancées notables sont enregistrées au fur et à mesure que le temps passe. La ville de Douala, comme beaucoup d'autres villes au Cameroun, contient de grandes zones de sols compressibles. Le défi majeur de ce travail était de faire une analyse comparative entre l'utilisation de deux types de fondations différentes dont les pieux et le radier couplé à un système de renforcement des sols pour une construction dans la ville de Douala. Le renforcement utilisé dans le cadre de cette analyse est l'utilisation d'inclusions rigides. Afin de relever ce défi, la conception des différentes fondations utilisées a été faite à la fois à partir des résultats des essais pressiométriques Menard et en utilisant la méthode des éléments finis pour les pieux et la méthode des éléments finis pour les radiers renforcés par des inclusions rigides. Les différentes méthodes d'éléments finis sont réalisées à l'aide de PLAXIS 2D V20. Une comparaison entre les différents systèmes de fondations a été faite sur la base de critères tels que la stabilité, les exigences pour la construction de la fondation, la durée de la construction et le coût des fondations. Ces comparaisons mènent à la conclusion que les inclusions rigides couplées à un radier peuvent être utilisées comme une solution alternative aux pieux pour la construction d'un bâtiment de grande hauteur sur un sol compressible.

Mots clés: sols compressibles, fondations, pieux, radier, inclusions rigides, batiment de grande hauteur

LIST OF FIGURES

LIST OF TABLES

TABLE OF CONTENTS

GENERAL INTRODUCTION

The constant decrease of resources is one of the major problems on which the world as a whole must find solutions. one of the ways in which this problem manifests is through the continuous reduction in the choice we have when there is the need of the choice of the site of a project. We are therefore sometimes constrained to construct some structures with high loads on soils which are problematic. Consequently, constructions on compressible soils become more and more frequent.

Generally, when dealing with great layers of compressible soils like the one used in this work, the trend is to use pile foundations especially for high loads. This is usually done by making sure that the pile's base is on a firm stratum of soil so as to obtain an adequate bearing capacity. Studies on alternative techniques based on the use of rafts with reinforcements are been done all over the world. Consequently, techniques such as vertical drains, jet grouting, stone columns and rigid inclusions are being used as ways to reinforce rafts.

The city of Douala as many other cities in Cameroon contains large areas of compressible soils. The major challenge of this work was to make a comparison between an alternative method (reinforced raft) and the use of piles in the construction of a tall building in Douala. This reinforced raft must satisfy the technical requirements by assuring that the settlements obtained are acceptable. Moreover, the reinforced raft must also be economically acceptable and affordable since there is no need to bring out a solution which is unrealistic and more expensive than the use of piles.

In order to face the challenge, this study unfolds in three chapters. The first is a literature review on compressible soils, the types of foundations adapted to compressible soils and the different soil improvement methods. Then, chapter 2 describes the methodology used in conducting the site visit, in acquiring the geotechnical and structural data, the methods of design of the different foundations used (both analytically and using finite element method) and at last the different comparison criteria. Finally, the last chapter gives a general presentation of Douala, a presentation of the geotechnical and structural data, results from the design of the different foundations and the comparison between the systems of foundations used.

1. CHAPTER ONE: LITERATURE REVIEW

INTRODUCTION

The growing scarcity of land resources is a major problem on which engineers must also find remedies. This can be done by finding techniques to construct on soils which are considered problematic such as compressible soils is a problem to solve both for the present and for the future, as it helps to establish a better plan for our cities, countries and the world in general. When dealing with compressible soils, especially with high loads, many designers generally prefer to adopt pile foundations for many reasons but an analysis between a reinforced raft and piles even for high loads such as in tall buildings needs more attention. This chapter will therefore be elaborated in three major sections. We will start by dealing with compressible soils, then we will deal with the different types of foundations adapted to construction on compressible soils and finally we will present the different soil improvement techniques.

1.1. Compressible soils

Almost every soil is compressible, that is it will settle when a load is applied but the term compressible soils is used to indicate soils with high compressibility, low permeability and resistance [11]. These types of soils are highly found near mouths of rivers, along the perimeters of bays, and beneath swamps or lagoons but can also be found in other areas [1]. In Cameroon, compressive soils are mostly found in the Northern, Littoral and West regions. Compressible soils are generally identified due to some characteristics.

1.1.1. Characteristics of compressible soils

Compressible soils are widely characterised based on three main characteristics which are their high compressibility which is a function of the load applied and time, very low permeability which vary with soil's deformations and their low resistances which increases generally with depth [11].

1.1.1.1. Compressibility

Compressibility is a measure of the reduction in volume or increase in density when a substance is subjected to an increase of pressure [2]. It is represented by the coefficient of compressibility, the coefficient of volume compressibility and the compression index.

a) **The coefficient of compressibility**

It is defined as the variation of the void ratio (e) against the variation of effective stress (σ’) curve (figure 1.5) as demonstrated in equation 1.1.

$$a_v = \frac{\Delta e}{\Delta \sigma'} \qquad \text{Equation 1.1}$$

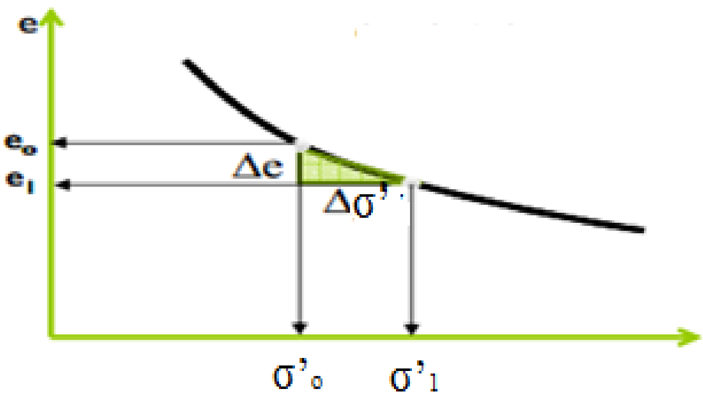


Figure 1.1. Relationship between void ratio and effective stress

b) **The coefficient of volume compressibility**

It is the volume decrease of a unit volume of soil per unit increase of effective stress (effective pressure) during compression (equation 1.2).

$$m_v = \frac{a_v}{1+e_0} \qquad \text{Equation 1.2}$$

Where, e_o is the initial void ratio.

Figure 1.2 shows the initial and final states of soil corresponding to the state before and after compression respectively.

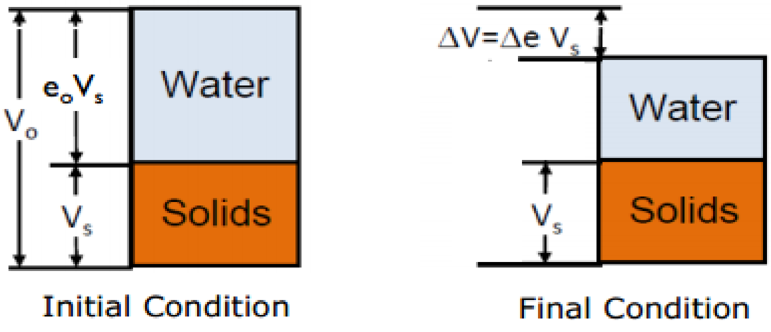


Figure 1.2. Initial and final state (after compression)

The volume change for a given applied stress in compressible soils is very high as compared to other soils.

c) **The compression index**

It is defined as the slope of the straight line portion of the graph of void ratio against the log σ' (figure 1.3) and can be calculated as in equation 1.3.

$$Cc = \frac{\Delta e}{\Delta log\sigma'} \qquad \text{Equation 1.3}$$

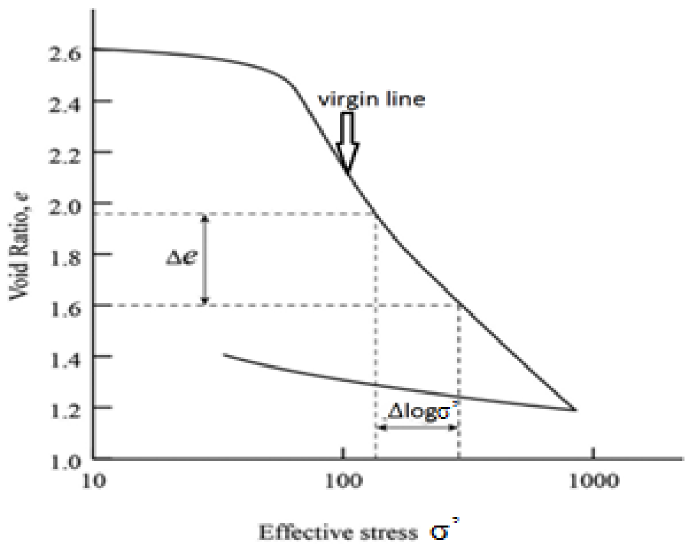


Figure 1.3. Graph of void ratio against effective stress

Since enough time is needed to carry out the consolidation test and cost involved in procuring undisturbed samples, Cc has been correlated extensively with liquid limit, plasticity index, void ratio at liquid limit, activity, plastic limit, natural moisture content, initial in situ void ratio and other similar parameters [13] as presented by table 1.1.

Table 1.1. Relationships between compressibility indices and some parameters (Bowles, 1996; Sridharan and Nagaraj, 2000 [13])

Equation	Reference	Region of applicability
$C_c = 0.007\ (w_L - 7)$	Skempton	Remolded clays
$C_c = 0.01\ w_N$		Chicago clays
$C_c = 1.15\ (e_o - 0.35)$	Nishida	All clays
$C_c = 0.30\ (e_o - 0.27)$	Hough	Inorganic cohesive soil; silt, silty clay, clay
$C_c = 0.0115\ w_N$		Organic soils, peats, organic silt and clay
$C_c = 0.0046\ (w_L - 9)$		Brazilian clays
$C_c = 0.009\ (w_L - 10)$	Terzaghi and Peck	Normally consolidated clays
$C_c = 0.75\ (e_o - 0.50)$		Soils with low plasticity
$C_c = 0.208\ e_o + 0.0083$		Chicago clays
$C_c = 0.156\ e_o + 0.0107$		All clays
Note: e_o = in situ void ratio, w_N = in situ water content; and w_L = liquid limit		

The parameter Cc is widely used in geotechnical engineering. Compressible soils will have higher values of Cc since they have relatively high liquid limits, high water content as presented by the relationships on table 1.1.

Compressibility is lower in coarse grained soils and increases as the proportion of small particles increases and becomes highest in fine-grained soils which contain organic matter [3].

Fine-grained soils which contain at least 50 percent of silt + clay may be listed in three classes of compressibility on the basis of their liquid limit [3]. The first class is low compressibility which is when the liquid limit is less than 30. The second is medium compressibility which is when the liquid limit ranges from 30 to 50 and the third is high compressibility which is when the liquid limit is greater than 50.

1.1.1.2. Permeability

Permeability can be referred to as the ability of a soil to allow a fluid to pass through it under pressure. It is generally represented by a permeability constant which is also known as hydraulic conductivity denoted by k.

The permeabilty constant can be determined both with in-situ tests and in the laboratory. With in-situ tests, the permeability constant can be obtained with Pumping or injection tests from a well (in confined or unconfined acquifers) or with Borehole permeability tests while in the laboratory, it can be determined either by the Constant head permeability test or by the Falling head permeability test.

For soils of low permeability, such as compressible soils, the falling head permeability test is more convenient beacause it has the advantages that very small quantities of flowing water can be measured and takes less time [14].

Considering the falling head permeability test, if the cross sectional area of the glass tube is **a**, the head difference at time **t** is **h**, the hydraulic conductivity k can be obtained by equation 1.4.

$$k = \frac{\mathrm{aL}}{\mathrm{At}}\, ln \frac{\mathrm{ho}}{\mathrm{h}} \qquad \text{Equation 1.4}$$

Where:

h_o is the value of the head difference at time t = 0 and

L is the length of soil sample.

Compressible soils generally have a lower permeability (table 1.4) than other soils.

1.1.1.3. Soil resistance (shear strength)

When designing geotechnical systems, geotechnical engineers must consider both drained and undrained conditions to determine which of these conditions is critical. The decision on what shear strength parameters to use depends on whether you are considering the short-term (undrained) or the long-term (drained) conditions. Due to the high compressibility of compressible soils, it is wise to consider the short term conditions.

The undrained shear strength **Cu** depends on the overburden pressure σ'_{z0}, the soil plasticity and also on OCR. It can be obtained using both laboratory by the triaxial consolidated undrained test and in situ by self-boring pressuremeter test or by the field vane test. The latter is generally preferred since it is easily accessible. However, Bjerrum has presented evidence that undrained strength as measured by the vane test is generally greater than the average strength mobilized along a failure surface in a field situation. The discrepancy was found to be greater the higher the plasticity index of the soil sample and is attributed primarily to the rate effect. In the vane test shear failure occurs within a few minutes, whereas in a field situation the stresses are usually applied over a period of time. For compressible soils, the value of undrained shear strength varies from 10 to 50 kPa [15]. Table 1.2 shows some values of the undrained shear strength for clays and table 1.3 shows some values of elastic constants for various soils.

Table 1.2. Undrained shear strengths against stiffness of clays (Craig [5].)

Stiffness state	*Undrained strength* (kN/m^2)
Hard	>300
Very stiff	150–300
Stiff	75–150
Firm	40–75
Soft	20–40
Very soft	<20

Table 1.3. Soil types and elastic constants

Soil Type	Typical Range of Young's Modulus Values, E_s (tsf)	Poisson's Ratio, ν
Clay:		
Soft sensitive	25-150	0.4-0.5 (undrained)
Medium stiff to stiff	150-500	
Very stiff	500-1,000	
Loess	150-600	0.1-0.3
Silt	20-200	0.3-0.35
Fine Sand:		
Loose	80-120	0.25
Medium dense	120-200	
Dense	200-300	
Sand:		
Loose	100-300	0.20-0.36
Medium dense	300-500	
Dense	500-800	0.30-0.40
Gravel:		
Loose	300-800	0.20-0.35
Medium dense	800-1,000	
Dense	1,000-2,000	0.30-0.40

Other characteristics of compressible soils such as their water content, void ratio, porosity and dry density are presented on table 1.4.

Table 1.4. Characteristics of compressible soils (AMSOIL [15])

CHARACTERISTICS	PEATS	ORGANIC SOILS	VASES	SOFT CLAYS
WATER CONTENT(%)	200-1000	100-200	60-150	30-100
VOID RATIO (e)	3 to 10	2 to 3	1.5 to 3	1.2 to 2
POROSITY n	0.75 to 0.9	0.7 to 0.8	0.6 to 0.75	0.55 to 0.7
COMPRESSIBILITY $Cc / (1+e_0)$	0.4 to 0.8	0.2 to 0.35	0.25 to 0.4	0.15 to 0.3
CREEP INDEX	0.02Cc	0.03 to 0.05Cc		
PERMEABILITY COEFFICIENT k (m/s)	10^{-6} to 10^{-9}	10^{-6} to 10^{-9}	10^{-6} to 10^{-9}	10^{-9} to 10^{-11}
CONSOLIDATION COEFFICIENT Cv (m^2/s)	10^{-6} to 10^{-8}	10^{-6} to 10^{-8}	10^{-6} to 10^{-8}	10^{-6} to 10^{-9}
UNDRAINED SHEAR STRENGTH Cu (kPa)	10 to 50	10 to 50	10 to 50	10 to 50
VARIATION RATE $\lambda cu = \Delta Cu/\Delta\sigma'$	0.5	0.2 to 0.3	0.2 to 0.3	0.2 to 0.3
DRY DENSITY ρd (t/m^3)	0.1 to 0.5	0.5 to 1	0.7 to 1.5	1 to 1.6
PARTICLES DENSITY ρs (t/m^3)	1.4 to 2	2 to 2.6	2.4 to 2.7	2.6 to 2.7

1.1.2. Types of compressible soils

Considering the characteristics common to compressible soils, they can thus be divided into five (05) types that are clays, peats, silts, marls and vases.

1.1.2.1. Clays

Clays are fine-grained sedimentary rocks, smaller than 5µm, composed of a large part of specific minerals, silicates in general, more or less aluminium hydrates, which represent a layered structure that explains their absorption qualities. The main groups of crystalline materials that make up clays are the minerals kaolinite, illite, and montmorillonite [4].

Kaolinite has a structure that consists of one silica sheet and one alumina sheet bonded together into a layer about 0.72 nm thick and stacked repeatedly. A kaolinite particle may consist of over 100 stacks. The layers are held together by hydrogen bonds. Kaolinite is common in clays in humid tropical regions.

Illite consists of repeated layers of one alumina sheet sandwiched by two silica sheets. In the silica sheet there is partial substitution of silicon by aluminium. The layers, each of thickness 0.96 nm, are held together by relatively weak bonding due to non-exchangeable potassium ions.

Montmorillonite has a structure similar to illite, but the layers are held together by weak van der Waals forces. Montmorillonite belongs to the smectite clay family. The space between the combined sheets is occupied by water molecules and exchangeable cations other than potassium, resulting in a very weak bond. Additional water can easily enter the bond and further separate the layers in montmorillonite, causing swelling. Montmorillonite is often called a swelling or expansive clay. They decrease in volume under the effect of drought, up to cracking on the surface and even to a depth of 2m to 4m. Moreover, under the effect of a load a part of the absorbed water contained in the clay grains are driven out, which causes a significant settlement.

1.1.2.2. Peats

Peats consist predominantly of plant remains, usually dark brown or black in colour and with a distinctive odour [5]. If the plant remains are recognizable and retain some strength the peat is described as fibrous. If the plant remains are recognizable but their strength has been lost they are pseudo-fibrous and if recognizable plant remains are absent, the peat is described as amorphous [5]. They have a high organic matter content, a very high water content and a very high degree of saturation and are composed of decomposed vegetable fibers which constitutes an anisotropic structure that influences the mechanical resistance. The pressure of preconsolidation is generally difficult to determine, although they are most likely normally consolidated soils. The consolidation phase is generally very short and difficult to define. Secondary compression is often predominant. The compression indices determined with the oedometer are strong (greater than 1). The permeability generally has a much stronger horizontal component than the vertical. This permeability decreases significantly during compaction.

1.1.2.3. Silts

They have a skeleton which is siliceous to silica-lime skeleton with fine grain. Their sizes are located between that of sands and that of clays that is smaller than 0.075 mm and larger than 0.002 mm [6]. The silts are less permeable and constitute fertile land. Their seat being poor, they are therefore to be avoided for the foundations.

1.1.2.4. Marls

Marl (marlstone) is a mud cemented by calcium carbonate or lime [4]. Marls are both clayey and calcareous. We consider, according to their composition, three major categories. Firstly, clayey marls which contain 5 to 35% carbonate of calcium. Then, the pure marls (relatively pure) and the calcareous marl with rates respectively from 35 to 65% and 65 to 95% of calcium carbonate. Similar to clays, clayey marls have the particular disadvantage of cracking to some depth in drought. Marl has often been the subject of underground quarrying to produce lime. In general, marls constitute good foundations, particularly in the absence of gypsum, with some risks indeed. Marls are relatively soft rocks, they undergo a very active geodynamic on their surface and their fragility makes them very vulnerable to the aggressiveness of nature and humans. A combination of natural and anthropogenic factors can cause intense water erosion which will be noticed in soil degradation.

1.1.2.5. Vases

They are deposits formed in fresh or salt water, made up of generally very fine grains (less than 200 µm with a high percentage of particles smaller than 2 µm) of variable mineralogical nature, arranged in flakes. The proportion of water retained is quite high, the particles adhere to each other not according to the arrangement giving the greatest compactness, but according to the directions in which they came into contact. They generally contain a certain proportion of organic matter (the most often less than 10%). They can be peaty if the presence of certain microorganisms promotes the formation of peat. In coastal areas, the presence of sodium prevents the proliferation of these microorganisms and therefore vases deposited are not peaty. As they consolidate, they lose part of their water, the structure is destroyed and it transforms into clay or marl the less soft when the consolidation is important.

We can thus conclude that the evolution of fine soils is due to the presence of minerals clayey in soils such as marls, clays, etc., which show great sensitivity air (shrinkage, cracking, gradual disintegration of soil layers) and a strong affinity for water (with the classic consequences of humidification, including swelling, deconsolidation and loss of mechanical characteristics).

1.1.3. General problems on compressible soils

Compressible soils are problematic soils due to the numerous disadvantages they exhibit considering their characteristics. Some of these disadvantages include their low ultimate bearing capacity, their high settlements and the instabilities on excavations and embankments.

1.1.3.1. Ultimate bearing capacity

The ultimate bearing capacity is defined as the pressure which would cause shear failure of the supporting soil immediately below and adjacent to a foundation [7]. For a strip footing, general, local and punching shear failure have been identified [4].

a) **General shear failure**

In the case of general shear failure, a rigid wedge under the foundation penetrates into the soil and continuous failure surfaces develop between the edges of the footing and the ground surface, a state of plastic equilibrium is reached initially in the soil around the edges of the footing, which subsequently spreads downwards and outwards (figure 1.4). Heaving of the ground surface occurs on both sides of the footing, although the final slip movement would occur only on one side, accompanied by tilting of the footing. This mode of failure is typical of soils of low compressibility and the ultimate bearing capacity can clearly be defined.

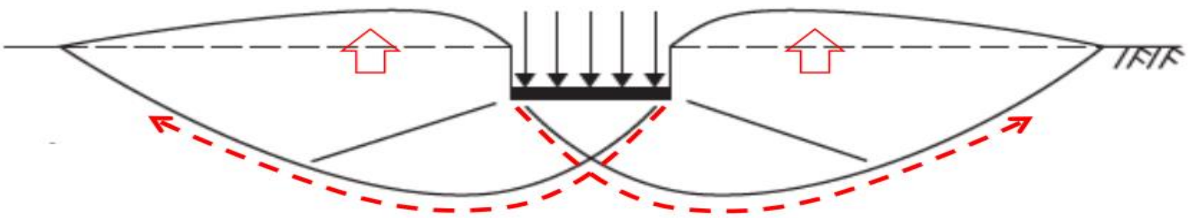


Figure 1.4. General shear failure mechanism

b) **Local shear failure**

Local shear failure is observed in relatively high compressible soils. Here, there is significant compression of the soil under the footing and only partial development of the state of plastic equilibrium (figure 1.5). It is also characterized by the occurrence of relatively large settlements which would be unacceptable in practice. Tilting of the foundation is generally not expected and the ultimate bearing capacity is not clearly defined.

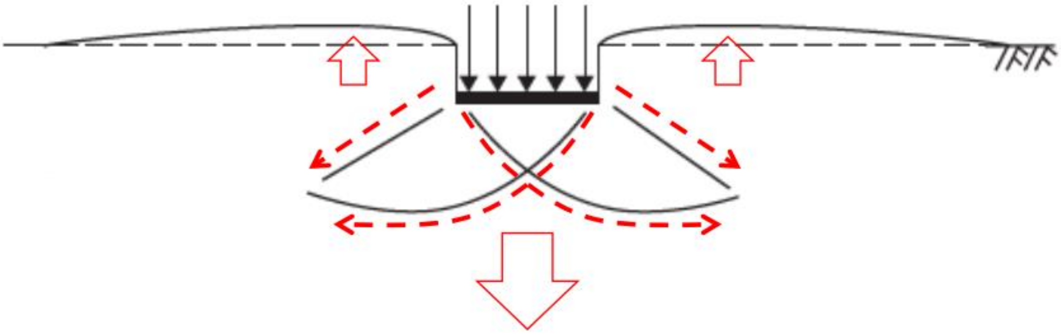


Figure 1.5. Local shear failure mechanism

c) **Punching shear failure**

This type of failure will mostly occur in highly compressible soils but can also occur in a low compressible soil if the foundation is located at considerable depth [5]. This mechanism occurs when there is relatively high compression of the soil under the footing, accompanied by shearing in the vertical direction around the edges of the footing (figure 1.6). Large settlements will be developed with no heaving of the ground surface beside the edges, and no tilting of the footing. The ultimate bearing capacity is not clearly defined.

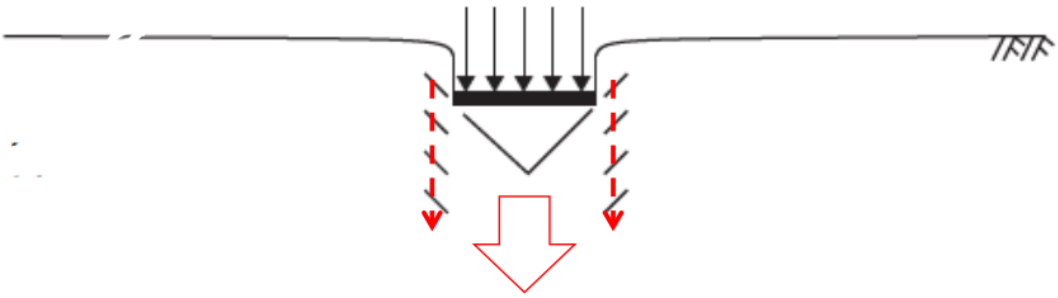


Figure 1.6. Punching shear failure mechanism

Generally, the mode of failure of a soil depends on both the compressibility of the soil and the depth of the foundation relative to its breadth.

1.1.3.2. Settlements

The total settlement is the sum of immediate settlement due to the elastic response of the soil without change in water content, primary consolidation settlement which takes place in clayey soil mainly due to the expulsion of the pore water in the soil and secondary consolidation (creep) settlement which takes place over long periods due to viscous resistance of soil under constant compression [9].

a) **Immediate settlement**

The immediate settlement of a strip footing can be gotten using an elasto-plastic response of soil to boundary conditions. It can be calculated assuming elastic model and integrating the vertical strain within the reference depth. For a layer of soft soil (as compressible soil) above a hard soil, the immediate settlement is evaluated in undrained conditions. Researchers Christian & Carter (1978) suggested equation 1.5.

$$Si = \mu_0 \mu_1 \frac{qB}{Eu} \qquad \text{Equation 1.5}$$

Where:

B, L are width and length of footing

q is pressure at the footing base

μ_0 is a coefficient function of D/B

μ_1 is a coefficient function of H/B and L/B

Charts which give the relationships between μ_1, μ_0 and the foundation dimensions are presented in annexe 9 and 10.

The elastic settlement of a single pile depends on the relative stiffness of the pile and the soil, the length-to-diameter ratio of the pile, and the distribution of elastic modulus of the soil along the pile length [8]. The elastic settlement of a single pile on soft soils tends to have elastic moduli that vary linearly with depth [5] and can be calculated with equation 1.6.

$$Si = \frac{Qs}{m\,L^2} Is \qquad \text{Equation 1.6}$$

Where m is a constant, Qs is the design load transferred as skin friction and Is is an influence factor given by equation 1.7.

$$Is = 2.0 \log \frac{L}{D} \qquad \text{Equation 1.7}$$

b) **Consolidation settlement**

The consolidation settlement takes place in clayey soil mainly due to the expulsion of the pore water in the soil [9]. The consolidation settlement (Sc) may be calculated with the oedometer method, based on the results of oedometer tests if the load area is very large or the thickness of the compressible layer is very small (B>>H) by using the compressibility and reloading indices Cc and Cr as shown in equations 1.8, 1.9 and 1.10.

$$Sc = \frac{Ho}{1+e0} Cc \log\frac{\sigma'z0 + \Delta\sigma'z}{\sigma'z0} \quad \text{if OCR=1} \qquad \text{Equation 1.8}$$

$$Sc = \frac{Ho}{1+e0} \left(Cr \log\frac{\sigma'zc}{\sigma'z0} + Cc \log\frac{\sigma'z0 + \Delta\sigma'z}{\sigma'zc} \right) \text{ if OCR>1 and } \sigma'z0 + \Delta\sigma'z > \sigma'zc \qquad \text{Equation 1.9}$$

$$Sc = \frac{Ho}{1+e0} Cc \log\frac{\sigma'z0 + \Delta\sigma'z}{\sigma'z0} \quad \text{if OCR>1 and } \sigma'z0 + \Delta\sigma'z < \sigma'zc \qquad \text{Equation 1.10}$$

Sometimes, a pile group may be embedded above a soft clay layer and transfer sufficient load to it (soft clay) to cause consolidation settlement. To estimate the consolidation settlement, the full design load is assumed to act at a depth of 2/3 L and is then distributed in the ratio of 2:1 (vertical: horizontal). The increase in vertical stress at a depth z in the soft clay layer shown in Figure 1.7.

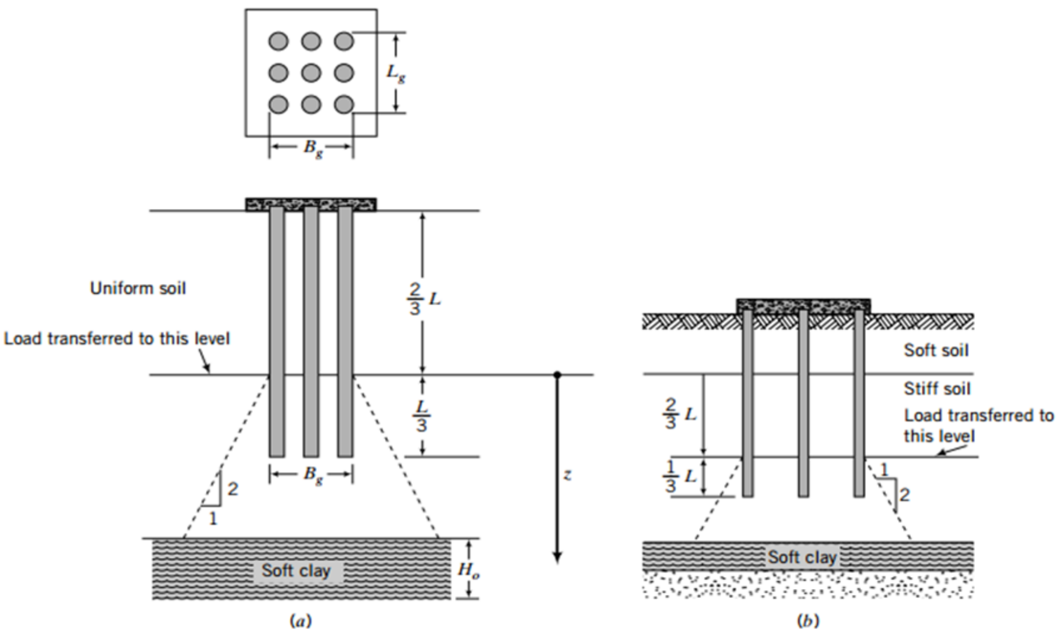


Figure 1.7. Increase in vertical stress of pile foundation on soft clay (BUDHU [4]).

c) Secondary consolidation settlement

Secondary consolidation (creep) settlement (ΔS) which takes place over long periods due to viscous resistance of soil under constant compression [9]. It is calculated on the assumption that the secondary compression index is a constant and can be determined from a consolidation test as shown in figure 1.8. It can be obtained from equation 1.11.

$$\Delta S = \left(\frac{\Delta e}{1+e0}\right)H \qquad \text{Equation 1.11}$$

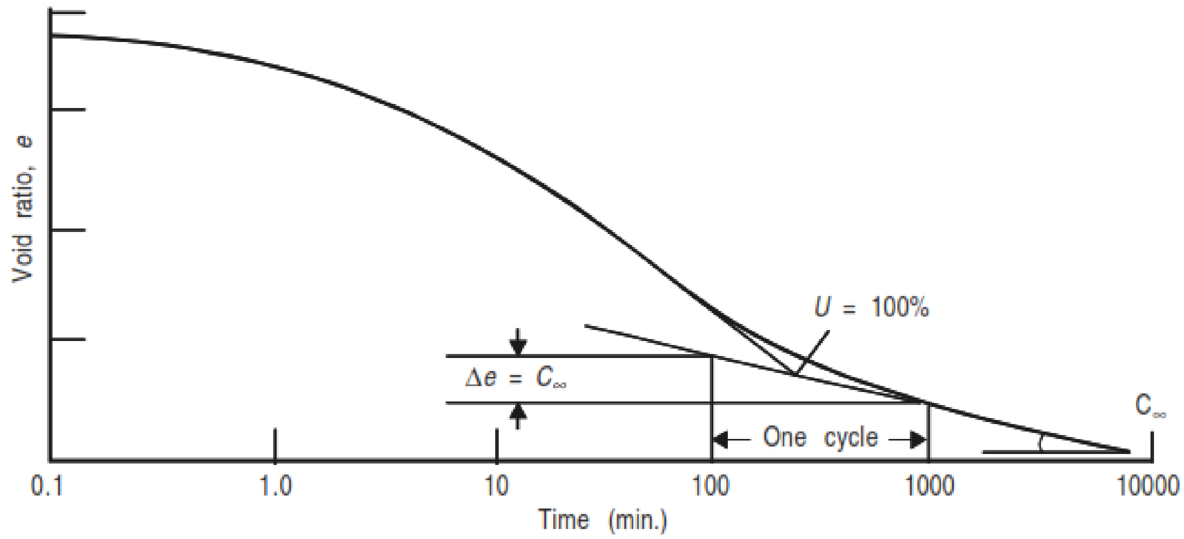


Figure 1.8. Determination of secondary consolidation in clays (P.C. VARGHESE [9]).

This shows that the settlements are directly proportional to the compressibility coefficients of soils. Thus compressible soils will show high settlements for stresses applied to them.

1.1.3.3. Instability on embankment and excavation forming

When constructing an embankment or when making an excavation on compressible soils, many problems arise due to the problematic characteristics of these soils leading to instabilities.

a) **Instability on embankment**

Embankment constructions which are on compressible soils, face two types of instabilities which are due to punching (Figure 1.9) and due to a failure or slip surface (Figure1.10).

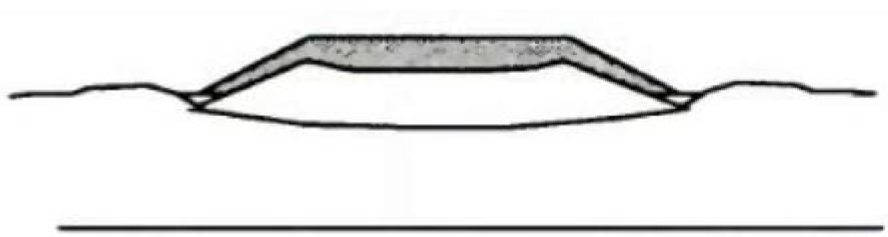


Figure 1.9. Punching under an embankment (Dadouche [23])

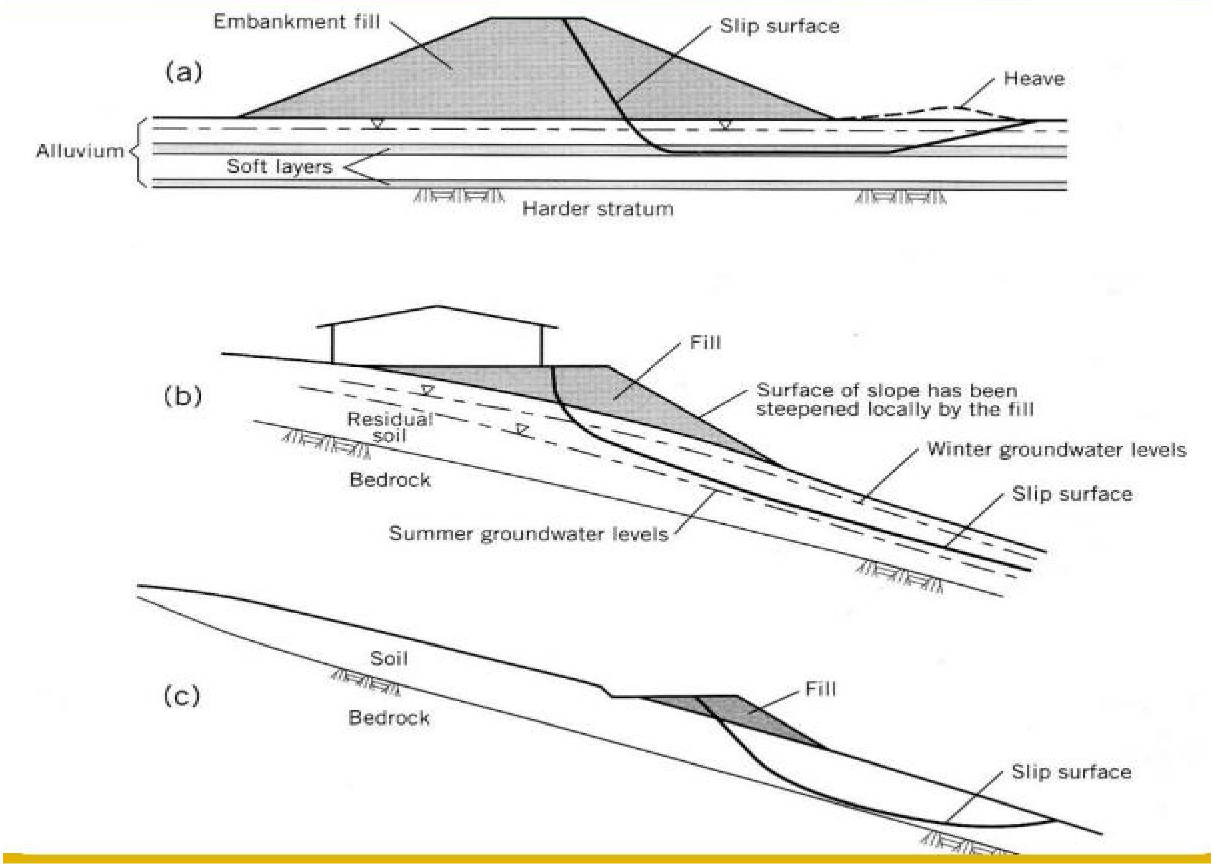


Figure 1.10. Instability due to failure or slip surface [24]

b) **Instability on excavation**

Instability on excavations can occur due to the settlement of the ground surface adjacent to the excavation (1), lateral movement of the vertical supports (2), and heave of the base of the excavation (3) [7] as illustrated by figure 1.11.

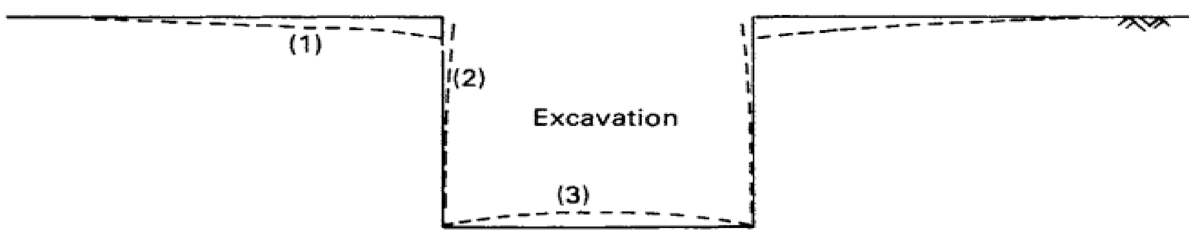


Figure 1.11. Instabilities on an excavation [24]

The stability of an excavation slope depends on the strength of the natural soil, its unit weight, the slope height, the slope angle and pore pressures generated by the excavation [12]. When excavating a part of a compressible layer of soil or another soil which lies on a compressible layer of soil, landslides generally occur due to the instability of compressible soils and a shear failure can therefore be developed due to undercutting as seen in figure 1.12.

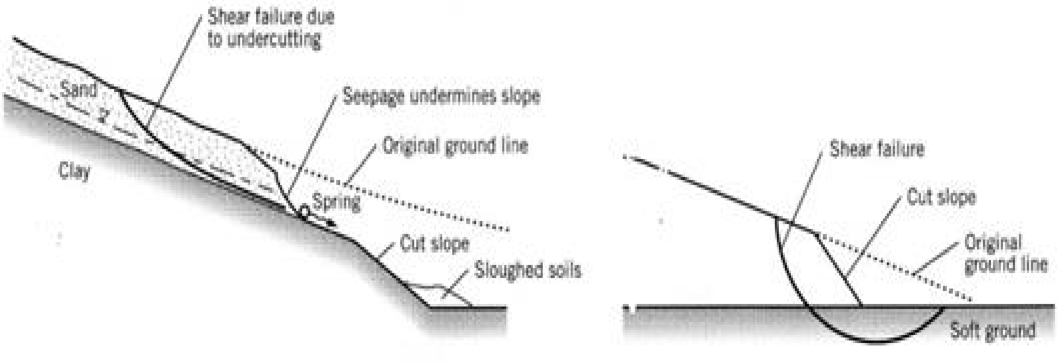


Figure 1.12. Instability due to undercutting [24]

The problem of instability on compressible soils generally leads to high settlements (due to punching) and soil movements as landslides. Thus, when coming in contact with these soils, great care must be taken to avoid long term and short term disasters.

1.2. Types of foundations adapted to construction on compressible soils

Due to the low bearing capacity, high settlements and low resistances of compressible soils, the choice and design of foundations when dealing with them must be done with great care. Foundations such as raft foundations which may sometimes be coupled with reinforcements, pile foundations or a combination of raft and pile foundations can be used among many others.

1.2.1. Raft foundation

A raft foundation consists of a relatively thin reinforced concrete slab cast integrally with reinforced concrete beams either above or below the slab in both directions. On compressible soils, raft foundations are usually used in order to redistribute the building load over the entire building area [10].

1.2.1.1. Types of raft foundations

Different types of raft foundations exist and are generally selected based on the structural system and the loads to be supported. They include solid slab raft, slab beam raft, cellular raft and piled raft foundations.

a) **Solid slab rafts**

These types of rafts generally have a slab and they include flat rafts, wide toe rafts, slip plane rafts and blanket rafts.

i) Flat raft

The flat raft is a reinforced concrete slab of uniform thickness over the whole bearing area. As reinforcements, two steel meshes are generally used with one at the bottom and another at the top of the slab.

ii) Wide toe raft

This type of solid slab raft is used to take the load at the external leaf of the cavity walls with their reinforced concrete toe which extends as a base. The shape of the extended toe allows a wider manoeuvre as the external brick outer leaf of the cavity wall can be finished below the ground.

iii) Slip plane raft

It generally involves a slip plane layer usually made of sand which is located between the sub-stratum and the raft. The slip plane layer extends beyond the raft and it should be of sufficient thickness to resist tensile or compressive ground strains as well as frost heave.

iv) Blanket raft

It consists of a concrete raft poured on a blanket. The blanket layer (generally made of stones) is built from the reduced sub-strata level. Compensation of the weak areas is done by the interaction between the raft and the blanket.

b) Slab beam raft

It generally consists of reinforced concrete beams place on (figure 1.14) or under (figure 1.15) a slab to reinforce the slab's rigidity. This type of raft is mostly used when the loads arriving on the ground are unequally distributed.

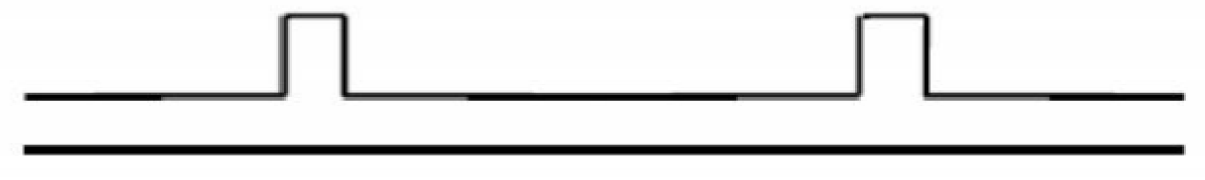


Figure 1.13. Beam on slab raft

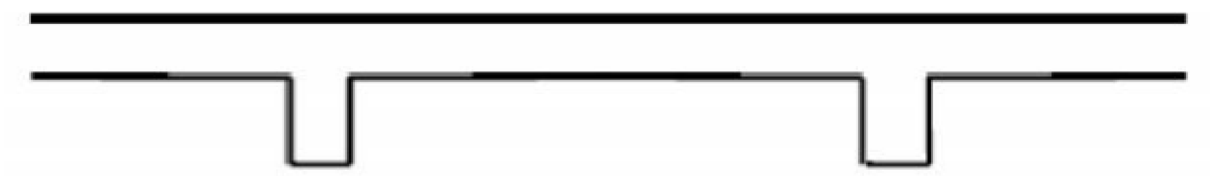


Figure 1.14. Beam under slab raft

c) **Cellular raft foundation**

This is a very rigid raft which consist of two slabs with two way interlocking ground beams. The upper slab and the lower slab are usually incorporated within the beams to form I sections with voids between them as illustrated on figure 1.16. Their rigidity make them suitable for heavy loads or loosed soils that can be subjected to uneven settlement.

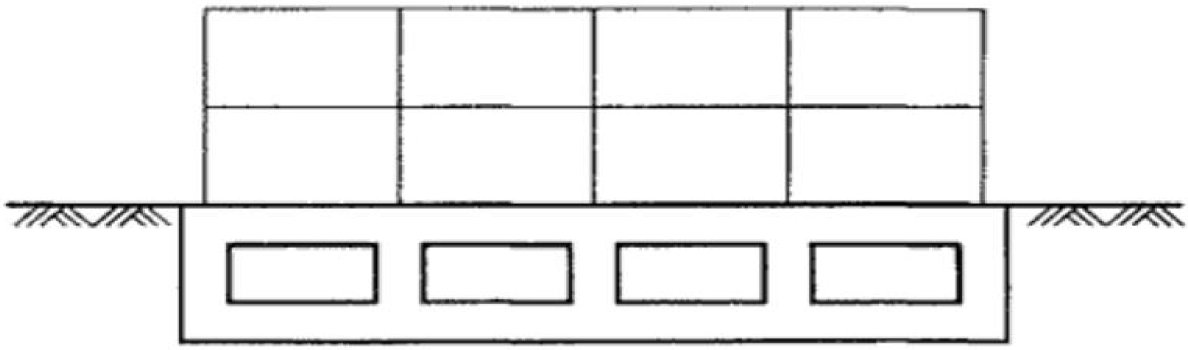


Figure 1.15. Cellular raft foundation

d) **Piled raft foundation**

This type of raft foundation is supported by piles as illustrated by figure 1.17. It is used when the soil at a shallow depth is highly compressible and the water table is high. Piles tend to improve the performance of raft foundations by reducing the amount of settlements and increasing the ultimate bearing capacity.

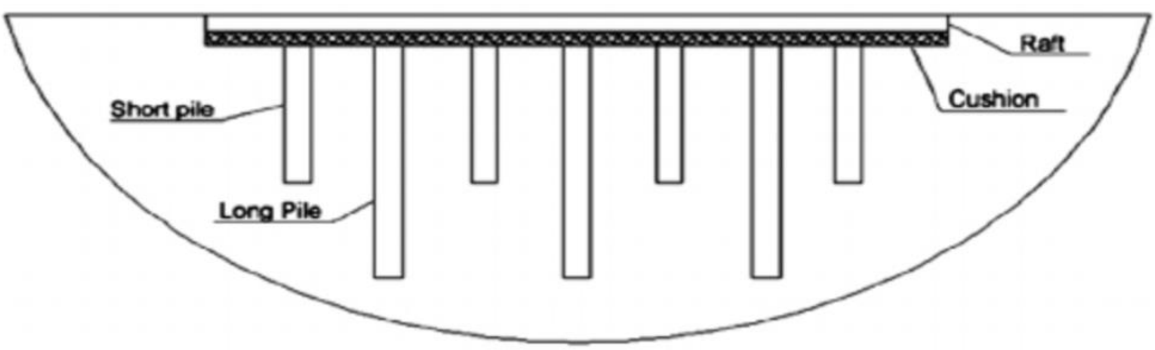


Figure 1.16. Piled raft foundation (V.J. Sharma et. al. 2015 [3])

1.2.1.2. Methods of design of raft foundations

In practice, rafts foundations can be designed both from in situ and from laboratory tests. Though the design from the laboratory tests are more precise than from in situ tests, the latter are generally cheaper and faster.

a) From in-situ tests

Design of rafts can be made from several in situ tests but in the purpose of this work, only the design from Ménard pressuremeter test and from the penetrometer test.

i) From Ménard pressuremeter test

The Ménard pressuremeter test is defined by NF EN ISO 22476-4. It is performed by the radial expansion of a tricell probe placed in the ground. This expansion is measured as a function of time and pressure.

When coupled with results of investigations from ISO 22475-1 or at least with identification and description of the ground according to ISO 14688-1 and ISO 14689-1 we get the get Ménard modulus (E_M), the Ménard limit pressure (P_{LM}) and the Ménard creep pressure (P_{fM}).

According to section D.2.1 of NF P 94-261, the net bearing capacity can be calculated from equation 1.12.

$$q_{net} = k_p \, p_{le}^* \, i_\delta \, i_\beta \qquad \text{Equation 1.12}$$

The settlement, according to section H.2 of NF P 94-261, is the sum of the deviatoric settlement (S_d) due to shear strains and consolidation settlement (S_c) as given by equation 1.13.

$$S = S_d + S_c \qquad \text{Equation 1.13}$$

Deviatoric and consolidation are calculated depending on whether the soil is homogenous or not as prescribed in section H.2.1.1 and H.2.1.2 of NF P 94-261.

ii) From penetrometer test

With penetrometer tests, we generally use the tip resistance (q_c) and the local fictional resistance between the sleeve and the ground to design raft foundations.

According to section E.2.1 of NF P 94-261, the net bearing capacity can be calculated from equation 1.14.

$$q_{net} = k_c\, q_{ce}^*\, i_\delta\, i_\beta \qquad \text{Equation 1.14}$$

b) **From laboratory tests**

Design of shallow foundations with a laboratory test can be performed through the oedometer test and the triaxial consolidated undrained tests.

The oedometer test enables to determine the compressibility parameters, the permeability constant and the over consolidation ratio which is used to calculate settlement. Settlement is then calculated with equations 1.5, 1.8, 1.9, 1.10 and 1.11.

The triaxial consolidated undrained test enables to determine the friction angle, the pore pressure and the undrained shear strength which is used to calculate the bearing capacity. This test also provides the elastic moduli which are necessary in the computation of settlement. The bearing capacity can therefore be calculated from equation 1.15 proposed by Vesic which corrects punching using the corrected coefficients ψ_γ, ψ_q and ψ_c.

$$q_{lim} = \psi_c\, cN_c + \psi_q\, q\, N_q + ½\, B\, \psi_\gamma\, N_\gamma \qquad \text{Equation 1.15}$$

Since we are in undrained condition, $c = c_u$, $\phi = \phi_u = 0$, $N_q = 1$, $q = \gamma_{sat}\, D$, $N_c = 5.14$ and $N_\gamma = 0$ thus equation 1.15 becomes equation1.16.

$$q_{lim} = 5.14\, \psi_c\, c_u + \psi_q\, \gamma_{sat}\, D \qquad \text{Equation 1.16}$$

1.2.1.3. Factors influencing the choice of raft foundations

Several factors are generally to be considered when there is a need to choose among raft foundations and other types of foundations. These reasons can either be technical, economical or even environmental.

A raft foundation can be chosen if the bearing capacity of the soil is so low that the total surface area necessary for other types of shallow foundations is greater than or equal to half the surface area of the building on the soil [4].

Moreover, when the bearing soil is heterogeneous enough and can cause high amounts of differential settlements, we can adopt raft foundations to neutralise these differential settlements [9].

When the last layer of the sub soil bearing the structure is located below the surface of the water table, the choice of raft foundation must be studied since it will be a good way to provide a waterproofed foundation.

In situations where there are soft deposits below hard layers, individual footings should be preferred over rafts [9].

1.2.2. Pile foundations

A pile is a long, slender structural element made of concrete, steel, timber, or polymer used to support structural loads [12]. Piles are suitable when dealing with compressible soils because it permits to transfer the loads to deep ground with a better bearing capacity.

1.2.2.1. Types of piles

Piles are generally classified according to the material on which they are made, the diameter and the technology used to construct the pile.

a) **Classification of piles based on the material**

Considering the material of constitution, piles can be classified as wooden piles, steel piles, concrete precast piles and concrete cast in place piles.

b) **Classification of piles based on the diameter**

Piles can be classified based on their diameters as micropiles ($d<25$ cm), medium diameter piles ($25\leq d\leq 80$ cm) and large diameter piles ($d>80$ cm)

c) **Classification of piles based on the technology**

Based on the technology, piles can be classified as driven piles (that is installation without soil removal), drilled piles, continuous-flight auger piles and full displacement piles.

1.2.2.2. Design of pile foundations

There are several methods to estimate the bearing capacity of a pile among which the design from in-situ tests and from laboratory tests.

a) **From in-situ tests**

Generally, the bearing capacity of piles in compression is the sum of the contributions from the base bearing capacity (Q_b) and that of the shaft resistance (Q_s) as given by equation

1.17 while for that of piles in traction the bearing capacity is given only by the contribution of shaft resistance [5].

$Q_{lim} = Q_b + Q_s$ Equation 1.17

The contribution of the bearing capacity given by the base of the pile is given by the product of the base resistance (q_b) and the base area (A_b) as given by equation 1.18.

$Q_b = q_b A_b$ Equation 1.18

The base resistance can be obtained from equation 1.19 and 1.20 from Ménard pressuremeter test and from penetrometer test respectively as proposed by NF P94-262.

$q_b = k_p p_{le}*$ Equation 1.19

$q_b = k_c q_{ce}*$ Equation 1.20

With the effective vertical stress (σ'_z), the friction angle at contact pile/soil (δ) and an empirical coefficient of horizontal stress (k), the shaft resistance is obtained using equation 1.21.

$q_s = k\ \sigma'_z \tan\delta$ Equation 1.21

Knowing the pile length under the ground (D) with the depth (z), the perimeter of the pile (Ps) and the shaft resistance (q_s), the contribution of the bearing capacity by the shaft of the pile can be obtained from equation 1.22.

$Q_s = Ps \int_0^D qs\,(z)dz$ Equation 1.22

b) **From laboratory tests**

This method is based on static bearing capacity equations. The bearing capacity equation is given by equation 1.17. The skin resistance q_s and base resistance q_b depend on strength parameters and can be gotten either by α-method based on total stress analysis or β-method based on effective stress analysis.

i) The α-method

The α-method is normally used to estimate short term load capacity of piles embedded in fine grained soils. In the α-method, a coefficient α is used to relate the undrained shear strength c_u to the lateral resistance q_s along the pile shaft (equation 1.23). The value of this coefficient α can be obtained from laboratory tests on model piles installed in a uniform deposit of soil or from table 1.5 derived from statistical correlations from CPT and SPT results.

$$q_s = \alpha c_u \qquad \text{Equation 1.23}$$

Table 1.5 Some values of α for driven and drilled piles

Pile	c_u	α
Driven	$c_u \leq 25$ $25 < c_u < 70$ $c_u \geq 70$	1.0 $1-0.011(c_u-25)$ 0.5
Drilled	$c_u \leq 25$ $25 < c_u < 70$ $c_u \geq 70$	0.7 $0.7-0.008(c_u-25)$ 0.35

The tip resistance is gotten from equation 1.24, found by analogy with conventional failure mode of shallow foundations, but without considering the term relative to the weight of the soil below the foundation since its contribution is often negligible.

$$q_b = \sigma_{vl} N_q + c N_c = 9c_u + \sigma_{vl} \qquad \text{Equation 1.24}$$

Where:

σ: total vertical pressure at the pile tip

, N_c: bearing capacity factors for deep foundations

ii) The β-method

The β-method is used to estimate short term and long-term pile load capacities in all soil types. The skin resistance is found using coulomb's friction law and the tip resistance still by analogy of the conventional failure mode of shallow foundations as given by equations (1.11) and (1.12), respectively.

$$q_s = k\sigma_{vl}'\tan\delta \qquad \text{Equation 1.25}$$

$$q_b = \sigma_{vl}' N_q + c' N_c \qquad \text{Equation 1.26}$$

Where:

k: empirical coefficient of horizontal stress

σ_{vl}': effective vertical stress

δ : frictional angle at the contact pile/soil

With the contribution of the shaft to bearing capacity known, the settlement of pile can be estimated using equation 1.6.

1.2.2.3. Factors influencing the choice of pile foundation

The choice of pile foundations instead of other foundations can be based on criteria that can be economic, technical or even environmental. A compromise must therefore be found before a decision is taken.

Economically, the cost of using all the different types of foundations must be evaluated and if pile foundations seem to be advantageous, they can be adopted.

Technically, pile foundations are chosen for many reasons. This can be the case when the top strata have a very low bearing capacity and a layer of soil having a good one is found underground, pile foundations become more reliable since they will be supported by the good layer of soil [9]. It can also be chosen if the differential settlements evaluated cannot be tolerated [4].

With compressible soils the use of rafts and/or piles with or without reinforcements as techniques to deal with the low bearing capacity, high settlements and low resistances they exhibit seems more efficient and must be designed with great care.

1.3. Soil improvement methods

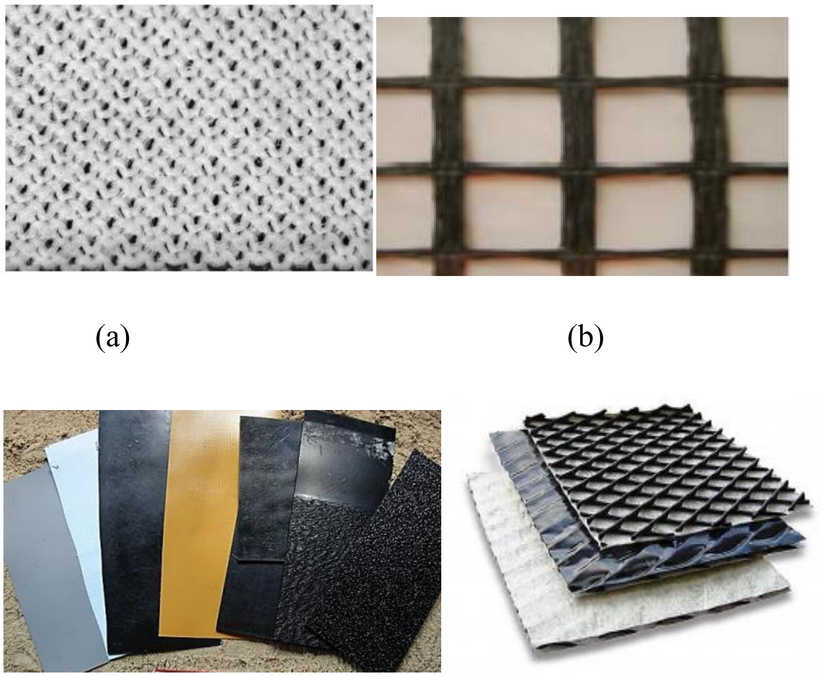
Several situations (such as large settlements, long consolidation time or instabilities) encountered with soils can motivate the need of a soil improvement method. Soil improvement methods are numerous and can be regrouped into surface improvement methods and deep improvement methods.

1.3.1. Surface improvement methods

Surface improvements methods include the use of geotextiles, soil nailing in case of excavations and embankments, chemical improvements and mechanical improvements such as power hammers, vibrating plates and rollers.

1.3.1.1. The use of geosynthetics

Geosynthetics are human-made materials, made from various types of polymers and used to enhance environmental, transportation and geotechnical engineering construction projects and make possible cost effective. They include geotextiles, geogrids, geonets, geocells, geomembranes, geocomposites and geosynthetic clay liners. They are often used for separation, reinforcement, filtration, drainage or as a liquid barrier.



(a) (b)

(c) (d)

Figure 1.17. Some types of geosynthetics: (a) Geotextile; (b) Biaxial geogrid; (c) Geomembranes and (d) Geocomposites (Sanja K [25])

When used as separations, they avoid the mixture of two soils in contact wheras as reinforcements, they absorb tension like steel reinforcement in the concrete thus increasing the overall resistance. As for their filtration role, they permit water passage avoiding the soil particle passage. Furthermore, they can be used for drainage by permitting water passage along the element with low energy dissipation and also as a liquid barrier by avoiding the passage of a liquid (general pollutant) across a section. Finally, they serve as a protection by avoiding erosion on a slope due to the overland runoff of water.

Geosynthetic inclusions within a soil mass can provide a reinforcement function by developing tensile forces that contribute to the stability of the geosynthetic-soil composite. According to several researchers [8] geosynthetic reinforcements are used to increase the bearing capacity of the soil and their efficiency depends on the depth of the first reinforcement, the vertical interspace between the reinforcement layers, the reinforcement width, the reinforcement number, the total reinforcement thickness, the strength and the stiffness of the reinforcements.

1.3.1.2. Chemical improvements

In geotechnical work quicklime (dry mixing) is used so as to take advantages from hydration which dries the soil, especially in soft clay.

Hydration: $CaO + H_2O \rightarrow Ca(OH)_2$ + 65.3 kJ/mol.

The lime adsorbs water = 32% of the lime weight and it increases its volume of about $\Delta V/V$= 100%. At the same time, it provides heat producing an increase of temperature. The hydrated lime or slaked lime $Ca(OH)_2$, is used for wet mixing. When lime reacts with soil, some reactions occur which include flocculation, cementation and carbonation.

Flocculation is the process of cation exchange between lime and soil (the clay adsorbs Ca and releases Na, K and other cations). The consequences are the aggregation of clayey minerals with the formation of flakes, the reduction of plasticity and an increase of strength and stiffness.

Cementation is the process of hydration of the clayey silicate which form the crystals typical of the cement. The increase of strength is proportional to the availability of silica in the clay, and the maximum improvement is reached when the lime combines with the all amount of silica: a larger amount of lime doesn't improve more the strength of the final product. The mixture hardening evolves as the cement gels develop (very fast at the beginning and then gradually lower with totally 28 days for reaching the final resistance).

In carbonation the lime reacts slowly with the carbon anhydride present in air or in the soil pores, forming $CaCO_3$ that is a binder very stable in time.

Tables on typical cement requirements for various soil types and typical average properties of soil-cement and soil-lime mixtures are shown on annexes 11 and 12 respectively**.**

1.3.1.3. Mechanical improvements

Surface mechanical improvements of compressible soils are mainly done by compaction with vertical drains since they have a low permeability. They include techniques such as heavy tamping, the use of power hammers, vibrating plates, rollers and explosives. These techniques are mainly used to reduce the settlements and increase the bearing capacity. The soil mass is compacted in layers called lifts. Coarse grained soils are compacted in lifts between 250 mm and 300 mm while fine-grained soils are compacted in lifts ranging between 100 mm and 150 mm [4]. The stresses imparted by compactors, especially static compactors, decrease with lift depth but a lower lift thickness is then preferable for uniform compaction. A comparison of various types of field compactors and the type of soils they are suitable for is shown in table 1.6.

Table 1.6. Comparison of field compactors for various types of soils (BUDHU [4])

		Compaction type				
		Static		Dynamic		
		Pressure with kneading	Kneading with pressure	Vibration	Impact	
Material	Lift thickness (mm)	Static sheeps-foot grid roller; scraper	Scraper; rubber-tired roller; loader; grid roller	Vibrating plate compactor; vibrating roller; vibrating sheepsfoot roller	Vibrating sheepsfoot rammer	Compactability
Gravel	300+	Not applicable	Very good	Good	Poor	Very easy
Sand	250±	Not applicable	Good	Excellent	Poor	Easy
Silt	150±	Good	Excellent	Poor	Good	Difficult
Clay	150±	Very good	Good	No	Excellent	Very difficult

1.3.2. Deep improvement methods

Improving compressible soils using deep improvement methods include techniques such as vertical drains, stone columns, vertical inclusions and jet grouting.

1.3.2.1. Vertical drains

These are highly permeable elements which are across the layer to be consolidated and permit to reduce the consolidation time by reducing the drainage path and/or by creating a path with a major permeability ($k_h > k_v$).

Some arrays of vertical drains are installed and a load is applied on the top of the drains. The vertical drains accelerate the settlement rate by reducing the drainage path the water must travel to escape from the compressible soil layer to half the horizontal distance between drains [7] as shown in figure 1.18. Since the consolidation time is proportional to the square of the length of the longest drainage path, when the latter is shortened by 50%, the consolidation time is reduced by a factor of four.

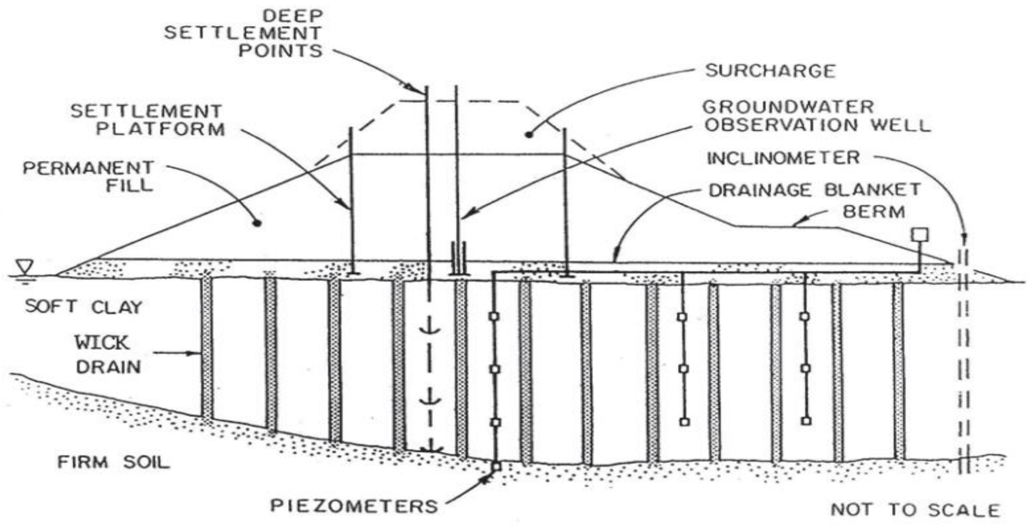


Figure 1.18. Use of vertical drains to accelerate settlement (NCHRP, [26]).

Vertical drains such as sand drains which are basically holes drilled in a cohesive soil and filled with sand are sometimes used. Sand has larger particle size thus its permeability is much higher, so water can flows easily. Prefabricated vertical drains are another option which are relatively cheap, provide higher conductivity and can easily be installed at close spacing, thus shortening the path of pore water in the impermeable soil and expediting the consolidation process. Figure 1.19 shows a prefabricated vertical drain and the stages to install it.

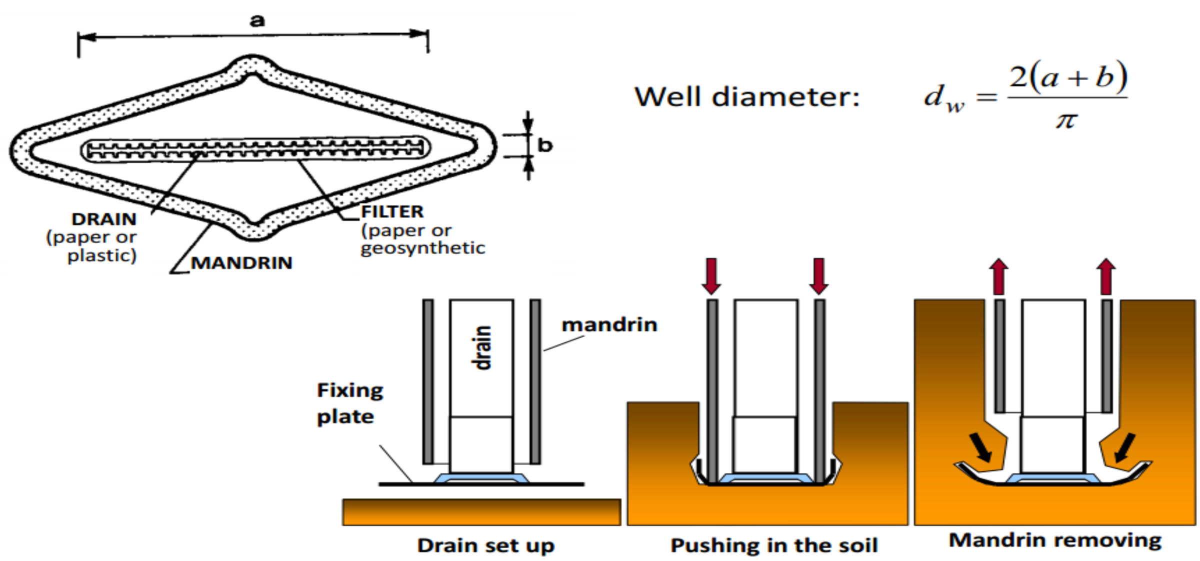


Figure 1.19. Prefabricated vertical drain (Miura et. al. [22])

1.3.2.2. Stone columns

Stone column ground improvement involves adding vertical columns of stone into the ground to a depth of at least 4m below the ground surface. A layer of compacted gravel can then be put over the top of the columns, ready for the construction of new house foundations. Stone columns may sometimes provide the soil with an increased drainage path to help reduce excess pore water pressure. They are realised with coarse materials (sand or gravel with 5mm <D<150mm) put in site by vibroflotting (deep vibrator), casing pile installed by vibration or casing pile installed by a screw. They are used to reduce the consolidation time (are like sand drains), reduce the settlement entity and increase the overall resistance of the system and are suitable in soft soil with c_u<50 kPa. Figure 1.20 and figure 1.21 respectively show a vibroflotting stone column and a casing pile stone column made by dry bottom feed method.

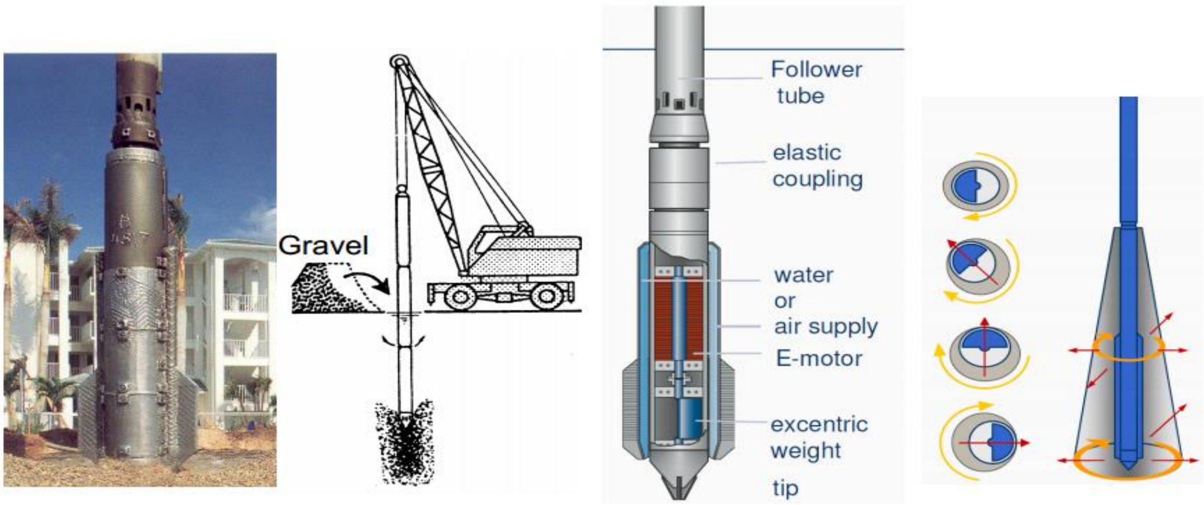


Figure 1.20. Vibroflotting stone column

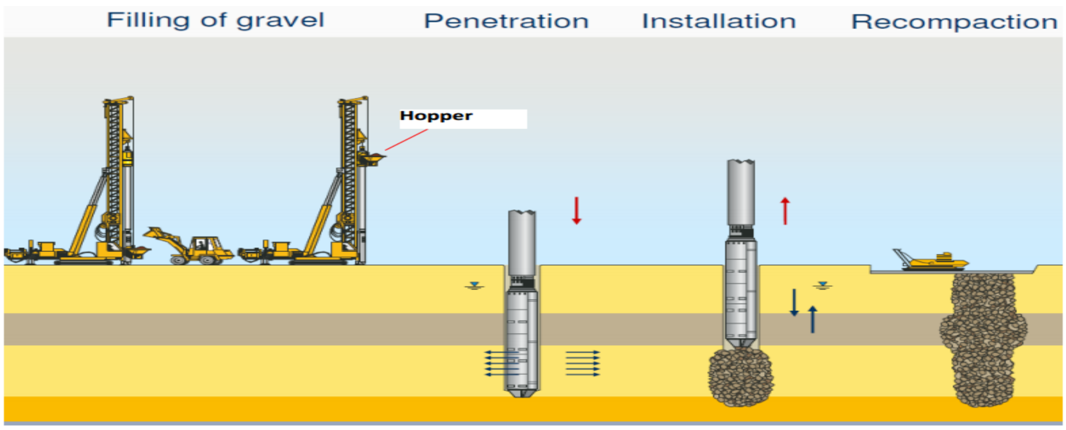


Figure 1.21. Casing pile stone column (Krishna et. al. [27])

1.3.2.3. Jet grouting

Jet grouting is a grouting technique that creates in situ geometries of soil crete (grouted soil), using a grouting monitor attached to the end of a drill stem. The jet grout monitor is advanced to the maximum treatment depth, at which time high velocity grout jets (and sometimes water and air) are initiated from ports in the side of the monitor. The jets erode and mix the in situ soil as the drill stem and jet grout monitor are rotated and raised. Figure 1.18 illustrates the typical procedures for jet grouting.

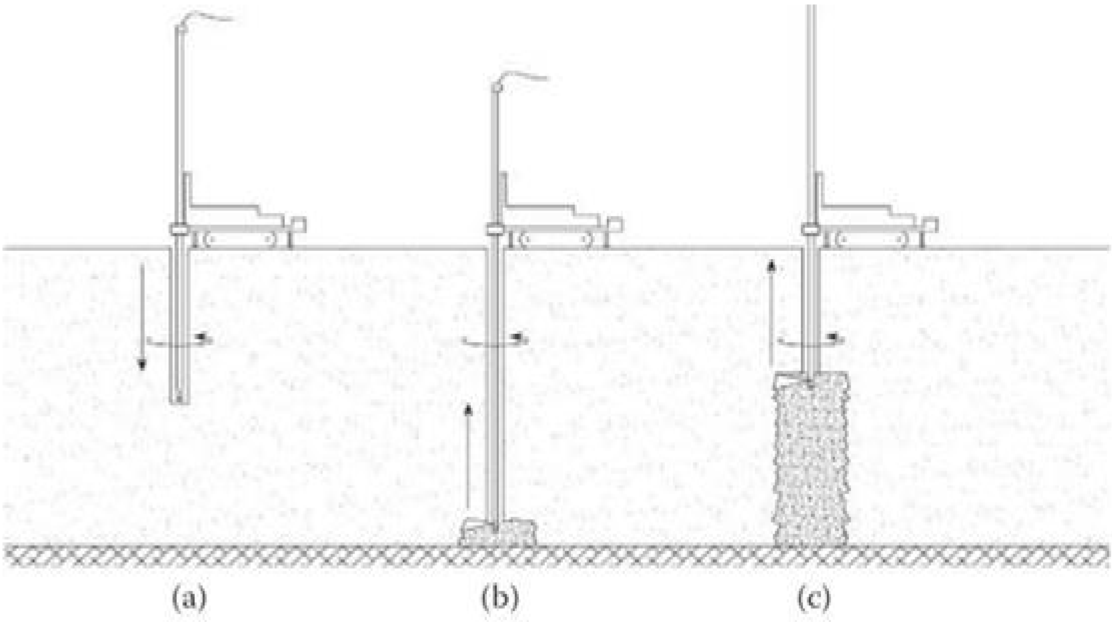


Figure 1.22. Typical jet grouting procedure: (a) drilling; (b and c) jet column formation (Croce et. al. [28]).

The jet grouting technology is based on the high-velocity injection of one or more fluids (grout, air, water) into the subsoil. The fluids are injected through small-diameter nozzles placed on a pipe that, in its usual application, is first drilled into the soil and is then raised towards the ground surface during jetting. The injected water-cement (W-C) grout cures underground, eventually producing a body made of cemented soil. Drilling is executed, up to the maximum desired depth of treatment, by using a rotating or rotary-percussive direct drilling system. Drilling can be performed with air, water, grouts or foams as flushing media. In general, the direct circulation of the drilling fluid, which flows downhole inside the hollow rods and up hole along the outer annular space, allows carrying of the drill cuttings to the surface and may also help in stabilizing the borehole walls.

Jet grouting can be regrouped into three types which are the mono-fluid or Cement Column Pile (CCP), the bi-fluid or Jumbo Special Pile (JSP) and the tri-fluid or Column Jet Grout (CJG or Kajima method) as illustrated in table 1.7.

Table 1.7. Characteristics of some jet grouting systems [24]

System	Fluid	Pressure at the pump (MPa)	Jet rate (m/s)	Column diameter (cm)
Monofluid C.C.P.	Slurry of cement	20-40	100-250	40-60
Bifluid J.S.P.	Slurry of cement	25-40	100-200	80-160
	Air	0,7-1	>330	
Trifluid C.J.G. (Kaijma)	Water	40-60	350-500	80-250
	Air	0,7-1,7	>330	
	Slurry of cement	2-6	50-80	

1.3.2.4. Ground improvement by rigid inclusions

Rigid inclusions refer to the use of semi-rigid or rigid integrated columns or bodies in soft ground to improve the ground performance globally so as to decrease settlement and increase the bearing capacity of the ground [29].

Piles and rigid inclusions are different in that, in rigid inclusions, the loads sustained by the soft soil is reduced (usually between 60 and 90%) in order to reduce the global and differential settlements [31]. The soft soil plays a role in rigid inclusions, and supports part of the load whereas in the pile foundation concept the soft soil is used for skin friction considerations.

With rigid inclusions, a Load Transfer Platform (LTP) usually made up of single or multiple layers of geosynthetics horizontally placed in compacted granular material, is often used with a thickness generally ranging between 40 and 80 cm [31]. Previously studied by Combarieu [30], the concept of rigid inclusions applied with a load transfer platform has more recently been the subject of an extensive French national research programme called ASIRI (*Améliorations des Sols par Inclusions Rigides*, which translates to *Ground Improvement by Rigid Inclusions*) (IREX, 2012).

The load transfer platform helps in the decrease of the bending moments and the shear stresses in the foundation slab of the structure to be supported while also helping in the transfer of the structural loads to the head of the rigid inclusions (figure 1.23) by means of an arching effect (developing in the granular layer) caused by the differential settlement arising between the soft soil and the heads of the rigid inclusions at the base of the load transfer platform, which also results in the emergence of a negative skin friction along the rigid inclusions at shallow depth [31].

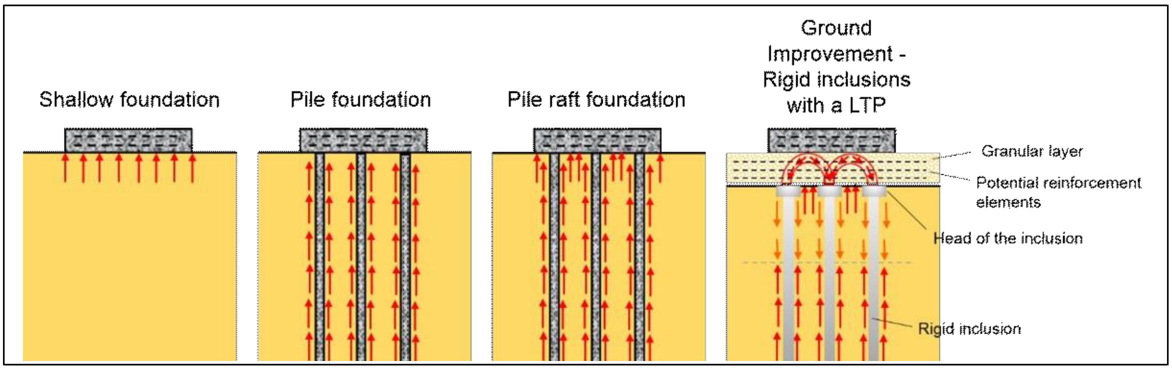


Figure 1.23. Type of load transfer in the different usual foundation concepts

Soils are generally improved because some conditions are not satisfied. Some of these conditions are the stability, settlement, consolidation time and piping. Table 1.8 presents some strategies to remedy situations where the mentioned conditions are unsatisfied.

Table 1.8. Strategies to remedy some foundation problems [24]

Requirement not satisfied	*STRATEGY*
Stability	Modification of embankment profile (heigth reduction, base enlargement, lateral banks)
	Step construction
	Use of light materials
	Reinforcement of soil (stone columns, piles)
	Reinforcement of embankement base or corp (geosynthetics)
	Partial o complete substitution of soil fondation
	Preconsolidation (Elettroconsolidation, Soil baking, Thermal precompression, Vacuum)
Too large settlement	Heigth reduction
	Use of light materials
	Reinforcement of soil (stone columns, jet-grounting, inection)
	Preload
	Partial o complete substitution of soil fondation
Too long consolidation time	Partial o complete substitution of soil fondation
	Preconsolidation (Elettroconsolidation, Soil baking, Thermal precompression, Vacuum)
	Drainage system (trench, vertical drains, stone columns)
Piping in foundation	Drainage system,
	Barrier constitured by concrete, Jet-grouting, Deep-mixing, Iniection

A classification of ground improvement methods and their principles for different ground categories is presented on table 1.9.

Table 1.9. Classification of ground improvement methods of the ISSMGE TC211 [29]

Category	Method	Principle
A. Ground improvement without admixtures in non-cohesive soils or fill materials	A1. Dynamic compaction	Densification of granular soil by dropping a heavy weight from air onto ground.
	A2. Vibrocompaction	Densification of granular soil using a vibratory probe inserted into ground.
	A3. Explosive compaction	Shock waves and vibrations are generated by blasting to cause granular soil ground to settle through liquefaction or compaction.
	A4. Electric pulse compaction	Densification of granular soil using the shock waves and energy generated by electric pulse under ultra-high voltage.
	A5. Surface compaction (including rapid	Compaction of fill or ground at the surface or shallow depth using a variety of compaction machines.
B. Ground improvement without admixtures in cohesive soils	B1. Replacement/displacement (including load reduction using lightweight materials)	Remove bad soil by excavation or displacement and replace it by good soil or rocks. Some lightweight materials may be used as backfill to reduce the load or earth pressure.
	B2. Preloading using fill (including the use of vertical drains)	Fill is applied and removed to pre-consolidate compressible soil so that its compressibility will be much reduced when future loads are applied.
	B3. Preloading using vacuum (including combined fill and vacuum)	Vacuum pressure of up to 90 kPa is used to pre-consolidate compressible soil so that its compressibility will be much reduced when future loads are applied.
	B4. Dynamic consolidation with enhanced drainage (including the use of vacuum)	Similar to dynamic compaction except vertical or horizontal drains (or together with vacuum) are used to dissipate pore pressures generated in soil during compaction.
	B5. Electro-osmosis or electro-kinetic consolidation	DC current causes water in soil or solutions to flow from anodes to cathodes which are installed in soil.
	B6. Thermal stabilisation using heating or freezing	Change the physical or mechanical properties of soil permanently or temporarily by heating or freezing the soil.
	B7. Hydro-blasting compaction	Collapsible soil (loess) is compacted by a combined wetting and deep explosion action along a borehole.
C. Ground improvement with admixtures or inclusions	C1. Vibro replacement or stone columns	Hole jetted into soft, fine-grained soil and back filled with densely compacted gravel or sand to form columns.
	C2. Dynamic replacement	Aggregates are driven into soil by high energy dynamic impact to form columns. The backfill can be either sand, gravel, stones or demolition debris.
	C3. Sand compaction piles	Sand is fed into ground through a casing pipe and compacted by either vibration, dynamic impact, or static excitation to form columns.
	C4. Geotextile confined columns	Sand is fed into a closed bottom geotextile lined cylindrical hole to form a column.
	C5. **Rigid inclusions**	Use of piles, rigid or semi-rigid bodies or columns which are either premade or formed *in-situ* to strengthen soft ground.
	C6. Geosynthetic reinforced column or pile supported embankment	Use of piles, rigid or semi-rigid columns/inclusions and geosynthetic grids to enhance the stability and reduce the settlement of embankments.
	C7. Microbial methods	Use of microbial materials to modify soil to increase its permeability strength or reduce its permeability.
	C8 Other methods	Unconventional methods, such as formation of sand piles using blasting and the use of bamboo, timber and other natural products.
D. Ground improvement with grouting type admixtures	D1. Particulate grouting	Grout granular soil or cavities or fissures in soil or rock by injecting cement or other particulate grouts to either increase the strength or reduce the permeability of soil or ground.
	D2. Chemical grouting	Solutions of two or more chemicals react in soil pores to form a gel or a solid precipitate to either increase the strength or reduce the permeability of soil or ground.
	D3. Mixing methods (including premixing or **deep mixing**)	Treat the weak soil by mixing it with cement, lime, or other binders *in-situ* using a mixing machine or before placement
	D4. Jet grouting	High speed jets at depth erode the soil and inject grout to form columns or panels
	D5. Compaction grouting	Very stiff, mortar-like grout is injected into discrete soil zones and remains in a homogenous mass so as to densify loose soil or lift settled ground.
	D6. Compensation grouting	Medium to high viscosity particulate suspensions is injected into the ground between a subsurface excavation and a structure in order to negate or reduce settlement of the structure due to ongoing excavation.
E. Earth reinforcement	E1. Geosynthetics or mechanically stabilised earth (MSE)	Use of the tensile strength of various steel or geosynthetic materials to enhance the shear strength of soil and stability of roads, foundations, embankments, slopes, or retaining walls.
	E2. Ground anchors or soil nails	Use of the tensile strength of embedded nails or anchors to enhance the stability of slopes or retaining walls.

CONCLUSION

All through this chapter, we have presented compressible soils, their common characteristics, their types, the problems we faced when dealing with them, the various types of foundations when constructing on them and the different soil improvement methods. From this analysis we are able to conclude that there is a need to compare between soil improvement methods coupled with rafts and deep foundations when we have high loads such as tall buildings on compressible soils. This comparative analysis will make the subject of the two next chapters.

2. CHAPTER TWO: METHODOLOGY

INTRODUCTION

Following the literature review which enabled us to have a broad knowledge on compressible soils, the different types of foundations adapted to constructions on them and the soil improvement techniques necessary to make them suitable, the objective of this chapter is to present the methods used to achieve our main objective clearly stated in the thesis topic. The achievement of this objective is done following a methodical and methodological approach which is mainly composed of recognition of the site, site visit, collection of data (geotechnical and structural data), design methods (analytical and finite element method using PLAXIS 3D) for the pile and raft foundations coupled with stone columns and finally the criteria comparison.

2.1. Site recognition

Recognition of the site was done through documentary research in order to know on one hand the general physical characteristics (geographical location, relief, climate, hydrography and geology) and on the other hand socio-economic characteristics.

2.2. Site visit

The site visit consisted mainly of inspecting the town of Douala precisely the boundaries of the Wouri river. This was done in one phase through observations.

2.3. Data collection

The data collected for the purpose of this research are of two main types. These are geotechnical and structural data.

2.3.1. Geotechnical data

The geotechnical data were collected from in-situ, laboratory test and from literature research.

2.3.1.1. In-situ tests*

The geotechnical data collected from in-situ tests were done from the standard penetrometer and Menard pressuremeter tests. From these tests we get the penetrometric and pressuremetric resistances and the Menard elastic modulus which enable us to establish the soil stratigraphy and to calculate the bearing capacity and settlement.

2.3.1.2. Laboratory tests

The geotechnical data collected from laboratory tests were done to obtain grain size from the sieve analysis, limits of Atterberg, density, natural water content, organic matter content, calcium carbonates content and uniaxial compression test.

2.3.2. Collection of structural data

The structural data used for the purpose of this work has been obtained from a thesis on the structural analysis and design of tall building [20]. It is a 22 storey building and the concrete structure is the case chosen. The base of the footing with the highest solicitations is the subject of this work. Solicitations are considered with an area subgrade reaction of 20000 kN/m^2.

2.4. Methods of design of foundations

The foundation design will be presented into two majors sections which are the design of pile using the analytical method and finite element method (using PLAXIS 3D) for the design of pile and raft coupled with reinforcements.

2.4.1. The design of pile foundation with empirical methods

In the course of this work we have computed the bearing capacity and settlement of a circular bored pile of diameter 1.0 metre working in compression under the most loaded column. The model used is the ground model.

2.4.1.1. Determination of the bearing capacity

The bearing capacity is the sum of the contributions from the base bearing capacity (Q_b) and that of the shaft bearing capacity (Q_s) as given by equation 2.1.

$$Q_{lim} = Q_b + Q_s \qquad \text{Equation 2.1}$$

The contribution of the bearing capacity given by the base of the pile is given by the product of the base resistance (q_b) and the base area (A_b) as given by equation 2.2.

$$Q_b = q_b A_b \qquad \text{Equation 2.2}$$

The base resistance is obtained from equation 2.3 from Ménard pressuremeter test as proposed by NF P94-262.

$$q_b = k_p p_{le}^* \qquad \text{Equation 2.3}$$

Where:

B: pile diameter

k_p: coefficient of pressuremetric bearing capacity

p_{le}*: net equivalent limit pressure

The net equivalent limit pressure is obtained from equation 2.4 taking into consideration individual limit pressures under the interval ranging from D+3a and D-b.

$$p_{le}^* = \frac{1}{b+3a} \int_{D-b}^{D+3a} p_l^*(z)dz \qquad \text{Equation 2.4}$$

Where:

D: depth of the pile in the ground

a: a factor obtained by max {B/2 ; 0.5 m}

b: a factor obtained by min {a ; h} with h the height of the foundation embedded in bearing substratum.

The coefficient of pressuremetric bearing capacity k_p depends on the pile class obtained from annexe 4 on the NF 94 262, the conventional category of the ground gotten from table B.2.1 of annexe B of the NF 94 262 and the effective embedded depth D_{ef};

The effective embedded depth is obtained from equation 2.5 and used to obtain k_p which in turn is compared to k_{pmax} obtained from table F.4.2.1 of NF 94-262.

$$D_{ef} = \frac{1}{p_{le}^*} \int_{D-hD}^{D} p_l^*(z)dz \qquad \text{Equation 2.5}$$

Where

h_D : the minimum between 10B and D

The coefficient of pressuremetric bearing capacity k_p is then given by equation 2.6 or 2.7 depending on the ratio D_{ef} / B.

$k_p = k_{pmax}$ when D_{ef} / B is greater than 5 Equation 2.6

$k_p = 1 + (k_{pmax} - 1) (D_{ef} / 5B)$ when D_{ef} / B is less than 5 Equation 2.7

The contribution of the bearing capacity by the shaft of the pile is obtained from equation 2.8.

$$Q_s = Ps \int_0^D qs\,(z)dz$$ Equation 2.8

Where:

Ps is the perimeter of the shaft given by πB

qs is the shaft resistance obtained from equation 2.9

$$qs\,(z) = \alpha_{pile\text{-}soil}\, f_{soil}\,(p_l^*\,(z)) \leq q_{s,max}$$ Equation 2.9

Where:

$\alpha_{pile\text{-}soil}$: a parameter obtained from table F.5.2.1 of NF 94-262

f_{soil} : a parameter depending on the soil and obtained from equation 2.10

$$f_{soil}\,(p_l^*) = (a\, p_l^* + b)\,(1 - e^{-c}\, p_l^*)$$ Equation 2.10

Where a, b and c depend only on the category of ground and are obtained from table 2.1.

Table 2.1. Values of parameters a, b, and c to determine f_{soil} (table F.5.2.2 of NF 94-262)

Ground category	Clay %CaCO3 < 30% Silts Intermediary soil	Intermediary soil Sand Gravel	Chalk	Marls and Calcareous marls	Altered and fragmented rock
Choice of curve	Q1	Q2	Q3	Q4	Q5
a	0.003	0.01	0.007	0.008	0.01
b	0.04	0.06	0.07	0.08	0.08
c	3.5	1.2	1.3	3	3

The shaft resistance qs is then compared to maximum shaft resistance ($q_{s,max}$) given by table F.5.2.3 of NF 94-262.

The characteristic values of the base and shaft resistance are then obtained from equation 2.11 and 2.12 respectively.

$$q_{b;i;k} = \frac{q_b}{\gamma_{R;d1}\,\gamma_{R;d2}} \qquad \text{Equation 2.11}$$

$$q_{s;i;k} = \frac{q_{s;i}}{\gamma_{R;d1}\,\gamma_{R;d2}} \qquad \text{Equation 2.12}$$

Where:

$q_{s;i}$ the shaft resistance of each layer.

$\gamma_{R;d1}$ and $\gamma_{R;d2}$ are coefficients gotten from table 2.2.

Table 2.2. Values of coefficients for the pressuremetric method (table F.2.1 of NF 94-262)

	Procedure of "model pile" (use of coefficient ξ or of annex D of NF EN 1990) Procedure of "ground model"		Procedure of "ground model"	
	$\gamma_{R;d1}$ Compression	$\gamma_{R;d1}$ Traction	$\gamma_{R;d2}$ Compression	$\gamma_{R;d2}$ Traction
Piles of class 1 to 7 not embedded in chalk excluding piles of category 10 and 15.	1.15	1.4	1.1	
Piles of class 1 to 7 embedded in chalk excluding piles of category 10, 15, 17, 18, 19 and 20.	1.4	1.7	1.1	
Piles of category 10, 15, 17, 18, 19 and 20.	2.0	2.0	1.1	

Using equations 2.11 and 2.12, the characteristic values of the bearing capacity of the base and shaft are obtained from equation 2.13 and 2.14 respectively with their sum giving the characteristic value of the bearing capacity of the pile given by 2.15.

$$Q_{b\,;\,k} = q_{b\,,\,k}\ A_b \qquad \text{Equation 2.13}$$

$$Q_{s\,;\,k} = \textstyle\sum_{i}.\ q_{s\,;\,i\,;\,k}\ A_{s\,;\,i} \qquad \text{Equation 2.14}$$

$Q_{c\,;\,k} = Q_{b\,;\,k} + Q_{s\,;\,k}$ Equation 2.15

With $A_{s;i}$: lateral section of the shaft (product of height and perimeter)

For the pile set in place without soil push-back we calculate load due to creeping with equation 2.16.

$Q_{c\,;\,cr\,;\,k} = 0.5\ Q_{b\,;\,k} + 0.7\ Q_{s\,;\,k}$ Equation 2.16

We therefore calculate the design value of the bearing capacity and load due to creeping of the pile using equation 2.17 and 2.18 respectively taking the parameters γt and γcr from table 2.3 obtained from tables C.2.3.1, C.2.3.2, 14.2.1.1 and 14.2.1.2 of NF 94-262.

$Q_{c\,;\,d} = Q_{c\,;\,k} / \ \gamma t$ Equation 2.17

$Q_{c\,;\,cr\,;\,d} = Q_{c\,;\,cr\,;\,k} / \ \gamma cr$ Equation 2.18

Table 2.3. Partial factors for design values of bored piles working in compression

ULS		SLS	
γt		γcr	
Persistent and transient situations	Accidental situations	Characteristics situations and	Quasi-permanent situations
1.1	1.0	0.9	1.1

Finally, we verify the two conditions given by equation 2.19 at ULS (for the persistent and transient situations) and equation 2.20 at SLS (for the quasi-permanent situations) with $F_{c\,;\,d}$ and F_d the design values of axial load at compression at ULS and SLS respectively.

$F_{c\,;\,d} \leq Q_{c\,;\,d}$ Equation 2.19

$F_d \leq Q_{c\,;\,cr\,;\,d}$ Equation 2.20

2.4.1.2. Settlement computation

Different methods have been established and are proposed by Eurocode NF 94-262. These methods are either based on experience or lump methods and for important cases, the method of Frank and Zhao.

From section L.2 of the NF P94-262 for load due to creeping (Q_{cr}) applied indefinitely, settlement is evaluated using equation 2.21 while Frank and Zhao proposed a method which gives the correlation with axial and based resistances using the pressuremetric modulus (E_M) as illustrated with equation 2.22 and equation 2.23.

$$S_{cr\,;\,v} = kB/100 + e_{iv} \qquad \text{Equation 2.21}$$

Where:

e_{iv} : shortening of the part of the pile out of the ground

k : an empirical factor usually taken as 2.

$$K_\tau = 2.0E_M \;/\; B \qquad \text{Equation 2.22}$$

$$K_q = 11.0E_M \;/\; B \qquad \text{Equation 2.23}$$

The model proposed by Frank and Zhao valid for loads less than $0.7Q_{cr}$ and for piles of width between 0.8 and 1.2 m.

2.4.2. Method of design of foundations from finite element

The finite element method in the course of this work is used both for the design of pile and that of reinforced raft using PLAXIS 2D.

2.4.2.1. Presentation of PLAXIS 2D

PLAXIS 2D is program, developed for the analysis of deformation, stability and groundwater flow in geotechnical engineering.

a) Historical background on the creation of PLAXIS 2D

PLAXIS is a suite of finite element programs that is used worldwide for geotechnical engineering and design. The development of PLAXIS began in 1987 at Delft University of Technology as an initiative of the Dutch Ministry of Public Works and Water Management. In 1993, because of continuous growing activities, the PLAXIS company (Plaxis bv) was formed and in 1998 the first PLAXIS 2D for windows was released. In the meantime, a calculation kernel for 3D finite element calculations was developed.

b) PLAXIS 2D sub programs

PLAXIS 2D is a full 2D program composed of an input and an output subprogram which combine an easy-to-use interface with full 2D modelling facilities.

i) PLAXIS 2D Input program

The Input program is a pre-processor, which is used to define the problem geometry, to create the finite element mesh and to define calculation phases. In this program are created the 2D geometry model composed of points, lines, x-y plane and other components and specify the material properties and boundary conditions. This is done in the first two tabsheets (Geometry modes) of the Input sub program. The last three tabsheets (Calculation modes) of this program are used for the mesh generation and the definition of the calculation phases. Figure 2.1 shows the layout of the Input program.

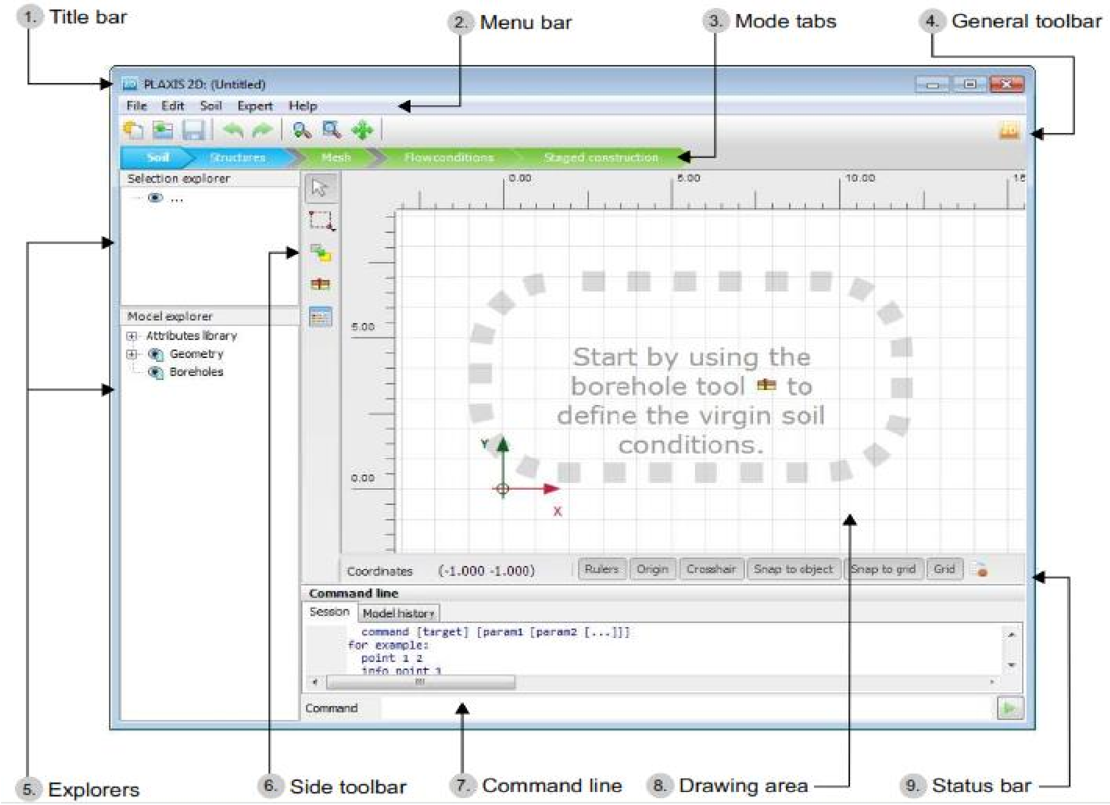


Figure 2.1. Layout of the Input program [21]

ii) PLAXIS 2D Output program

The output program is a post-processor, which is used to inspect the results of calculations in a two dimensional view or in cross sections and to plot graphs(curves) of output quantities of selected geometry points. The main output quantities of a finite element calculation are the displacements and the stresses. Moreover, when a finite element model involves structural elements, the structural forces in these elements are calculated. An extensive range of facilities exists within this program to display results of a finite element analysis. The main window of the output program is presented in figure 2.2.

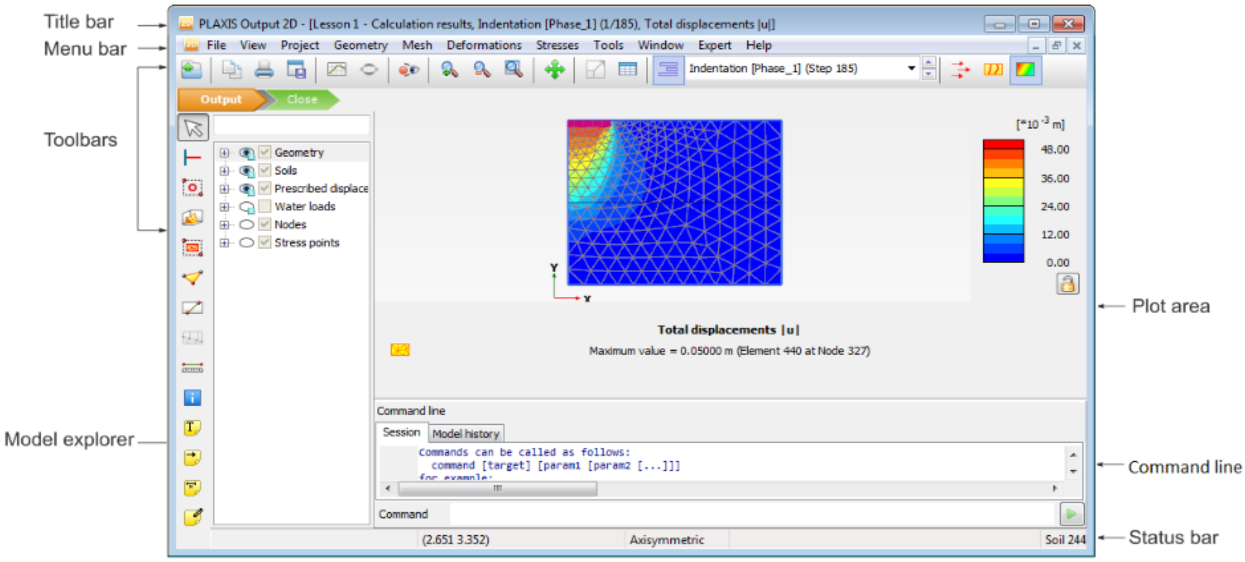


Figure 2.2. Main window of the Output program [21]

2.4.2.2. Design of pile from finite element method using PLAXIS 2D

Design in Plaxis is generally made in the input sub program in 7 steps. This include the definition of the project properties, the definition of soils, the definition of the different structures, meshing, the definition of the ground water conditions, the definition of the staged construction parameters and finally the calculations are launched.

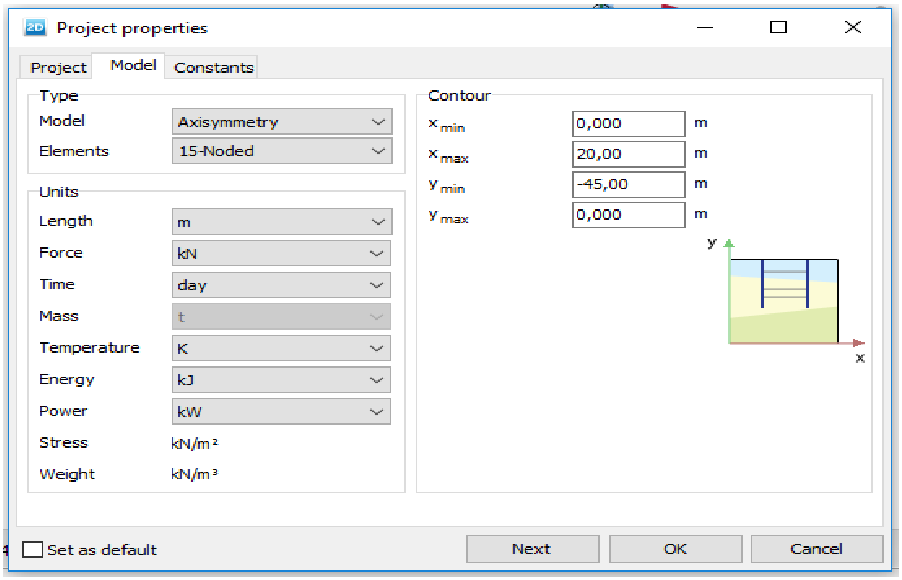


Figure 2.3. Model, units and contour values chosen for pile design

Defining the project properties includes setting up the project name, the contour values, project units (such as length, force and time), model type, number of elements and defining constants (such as gravity and volumetric weight of water). Figure 2.3 shows the chosen model, units and contour values. Constants used in the course of this design are the default values of constants.

Table 2.4. Soils and piles parameters for the finite element design

Material	Model	Material type	Young's modulus	Poisson's ratio	Unit weigths		Friction angle	cohesion	Layer depth
			E (kN/m2)	v'	γsat (kN/m3)	γunsat (kN/m3)	φ (°)	c' (kN/m2)	d (m)
Soils	Mohr-Coulomb	Drained	82920	0,20	25,41	24,41	22,00	0,20	0-1,5
			26940	0,20	25,51	24,51	22,00	0,20	1,5-3,0
			28920	0,30	22,46	22,00	25,00	0,25	3,0-5,0
			48300	0,30	16,58	16,00	25,00	0,25	5,0-7,0
			23880	0,20	24,43	23,43	22,00	0,20	7,0-9,0
			31620	0,20	23,94	22,94	22,00	0,20	9,0-11,0
			23820	0,25	23,45	22,45	25,00	0,30	11,0-11,5
			44220	0,25	23,74	23,00	25,00	0,25	11,5-13,0
			14310	0,20	23,64	22,64	22,00	0,20	13,0-16,0
			39360	0,20	25,02	24,02	22,00	0,20	16,0-17,5
			36150	0,20	25,11	24,11	22,00	0,20	17,5-19,0
			30330	0,20	25,02	24,02	22,00	0,20	19,0-20,5
			31680	0,25	25,02	24,50	25,00	0,5	20,5-22,0
			68310	0,30	24,43	23,43	22,00	0,20	22,0-23,5
			54060	0,30	20,80	19,80	22,00	0,20	23,5-25,0
			41700	0,20	25,31	24,31	22,00	0,20	25,0-26,5
			46890	0,25	26,00	25,50	30,00	0,50	26,5-28,0
			37680	0,20	25,70	24,70	22,00	0,20	28,0-29,5
			75000	0,25	25,21	23,00	25,00	0,40	29,5-31,0
			64770	0,25	25,60	25,00	22,00	0,20	31,0-34,0
			116685	0,4	24,82	24,50	25,00	0,70	34,0-36,0
			53955	0,4	26,19	26,00	25,00	0,70	36,0-37,5
			210900	0,35	25,51	25,00	25,00	0,50	37,5-39,0
			131085	0,1	25,60	25,00	15,00	0,50	39,0-44,0
Pile	Linear elastic	Non-porous	30000000	0,2	25	25	-	-	0-30,0

The next step involves the definition of the different soil types with their characteristics to define the soil stratigraphy using the borehole option. The different soil materials are defined in the material sets menu. Mohr's model was used for the different layers of soils. The soils' parameters used are summarised in table 2.4. The Young's moduli were obtained from correlations between the Menard pressuremetric moduli and a rheological factor obtained from table H.2.1.1.1 of NF P 94-261. Poisson's ratios, friction angles and cohesion are obtained from bibliographic researches. The dilatancy angles were taken as zero for all soils.

The structure was then defined using the linear elastic material model for pile and the different pile parameters (as presented in table 2.4) inserted. The interfaces were inserted to assure the boundaries between the different components and materials. Furthermore, the line loads were inserted.

The overall components are meshed, laying emphasis (more refinement) on areas near the structure and the ground water conditions are defined.

The calculation was done on 11 phases. The first phase represents the initial conditions (only the soil stratigraphy) while the second represents the pile construction. After the second phase, the displacements have been set back to zero. The remaining 9 phases are the incremental loading of the pile. This has been done using an increment factor of 0.2 as presented on table 2.5 and the calculation launched.

Table 2.5. Incremental load distribution

Loads (KN)	Load increments (KN)	Line loads (KN/m/m)
3735,89	0,00	0,00
	747,18	951,34
	1494,36	1902,67
	2241,53	2854,01
	2988,71	3805,35
	3735,89	4756,68
	4483,07	5708,02
	5230,25	6659,36
	5977,43	7610,69

2.4.2.3. Design of raft reinforced with rigid inclusions with PLAXIS 2D

The project properties and soil parameters used for this design are the same as those of pile design in section 2.4.2.2.

The foundation in this case consists of a raft, a load transfer platform (LTP) and rigid inclusions. The characteristics of these constituents are presented on table 2.6.

Table 2.6. Characteristics of the raft, load transfer platform and rigid inclusions

Material	Units	Raft	Load transfer platform	Rigid inclusion
Model	-	-	Mohr-Coulomb	Linear elastic
Material type	-	Plate	Drained	Non-porous
Young's modulus (E)	(kN/m2)	-	50000	11000000
Poisson's ratio (v')	-	-	0.2	0.2
Unit weigths	γsat (kN/m3)	-	22.0	20.0
	γunsat (kN/m3)	-	20.0	20.0
Friction angle (φ)	(°)	-	38.0	-
cohesion (c')	kN/m2	-	0.0	-
Layer depth	m	-	0.15-0.95	0.15-8.15
Diameter	m	-	-	0.5
Normal stiffness (EA)	kN/m	840.0e6	-	-
Flexural rigidity (EI)	kN/m2/m	44.80e6	-	-
Plate's weight	kN/m/m	0.0	-	-

To define the geometry and load applied on the foundation structure, the area of influence corresponding to the panel on the pile designed in sections 2.4.1 and 2.4.2.2 is considered. This panel is a 5m by 5m panel and the parameters are as shown by figure 2.4.

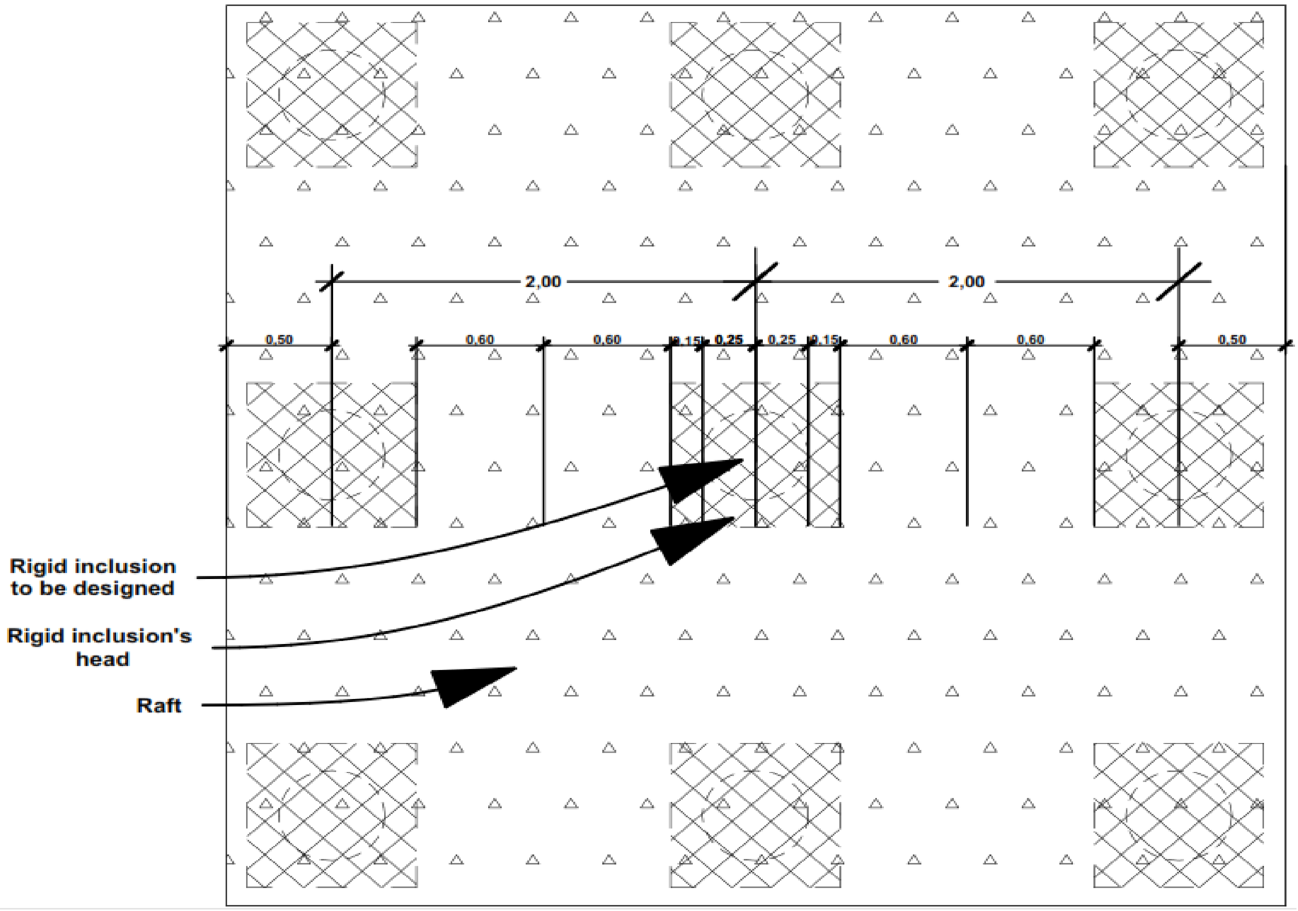


Figure 2.4. Details of the geometry of the 5m by 5m panel of raft

Since the model used is axisymmetry, the rigid inclusion used for design is the central one with the geometry characteristics shown in figure 2.5.

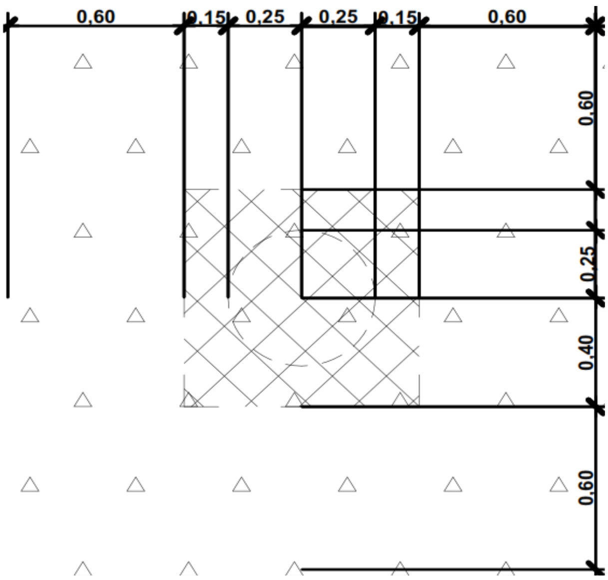


Figure 2.5. Geometric characteristics of the rigid inclusion to be designed

The cross section of the model used for the reinforced raft design, with the various thicknesses are presented on figure 2.6.

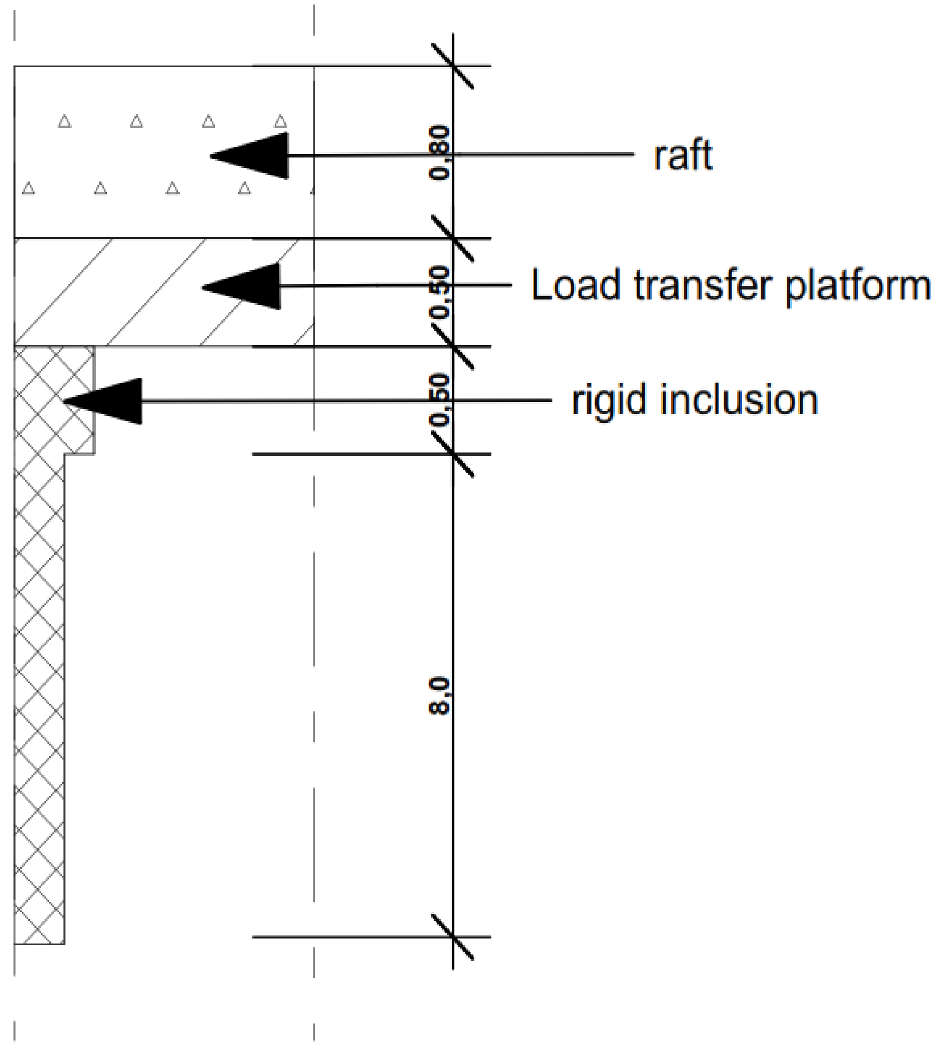


Figure 2.6. Model used for design

The structure was drawn based on figure 2.5 and the interfaces were inserted to assure the boundaries between the different components and materials. Furthermore, the line loads were inserted.

The overall components are meshed, laying emphasis (more refinement) on areas near the structure and the ground water conditions are defined.

The calculation was done on 11 phases. The first phase contains only the different soil stratigraphy while in the second (construction phase) the soil has been drilled and the structures (rigid inclusion, load transfer platform and raft) inserted. After this phase, the displacements have been set back to zero. The remaining 9 phases are the incremental loading of the reinforced raft. This has been done using an increment factor of 0.2 as presented on table 2.8 and the calculation launched.

Table 2.7. Loads on raft

Loads (KN)	Load increments (KN)	Line loads increment (KN/m/m)
3735,89	0,00	0,00
	747,18	29,89
	1494,36	59,77
	2241,53	89,66
	2988,71	119,55
	3735,89	149,44
	4483,07	179,32
	5230,25	209,21
	5977,43	239,10

2.5. Comparison criteria

Generally, three comparison criteria can be sorted out. These criteria are the stability, the cost and the duration of the construction of the foundation.

2.5.1. The stability

The stability is a factor which is generally considered as the most important factor since the structure lies on the foundation and the most important issue for an engineer is to avoid the collapse of the structure. The stability of the different foundations is evaluated in the course of this work, using the settlement analysis for various loads.

2.5.2. Requirements for the construction

This is one of the major factors since in some cases it is the determining factor for the choice of the type of foundations to be constructed. This will be evaluated in terms of the quality and quantity of equipments and human resources required for the construction of the different foundations.

2.5.3. The duration of the construction of the foundation

The duration of the foundation's construction generally depends on many factors which are material and human resources. In the course of this comparative analysis, the aforementioned factors are going to be assumed.

2.5.4. The cost of the foundation

Cost is a criterion which cannot be neglected since it constitutes one of the major task of an engineer who needs to find the best compromise between the different criteria. Since the costs of transports and logistics are quite complicated to evaluate, the costs of construction of foundations are evaluated in the course of this work based on the cost of excavations and cost of reinforced concrete used for the different foundations and the unit prices are those practised in the national market (obtained from the 2020 version of a document called "Mercurial").

CONCLUSION

The objective of this chapter was to present the methods used for the comparative analysis between the different foundations. This objective has been achieved through five (05) major steps. Firstly, was made the site recognition which was done using research from available documents. Secondly, was conducted the site visit. Collection of data was done at the third step and included the collection of geotechnical and structural data. Furthermore, the different methods of design of the foundations were presented and finally, the different comparison criteria.

3. CHAPTER THREE: RESULTS AND INTERPRETATIONS

INTRODUCTION

This chapter presents and interprets the results of the methodology outlined in chapter 2. This last chapter provides the actual design results and the comparison elements for the two foundation modes. Thus, we will start with the general presentation of the project site followed by the presentation of the project data: structural and geotechnical. Then, the presentation of results obtained from the design of the pile and reinforced raft foundations. Finally, the comparison of the two foundation modes according to the comparison criteria mentioned in the previous chapter.

3.1. General presentation of the city of Douala

In a general way, the city of Douala can be presented through the town's geographic location, relief, climate, hydrography, geology, demography and economic activities.

3.1.1. Geographic location

Douala is situated in the littoral region of Cameroon between latitudes 3°8 and 5°8 N and longitudes 9°8 and 11°8 E. It is the largest city in Cameroon (210 km^2), the economic capital of Cameroon, the headquarter of the littoral region and the economic capital of the entire CEMAC region comprising; Gabon, Congo, Chad, Equatorial Guinea, Central African Republic and Cameroon [16]. The city is located on the banks of the Wouri River, the two sides linked in the first place by the Bonaberi Bridge and eventually the second bridge. It is also the host of Central Africa's largest port and has a major international airport, Douala International Airport Figure 3.1 presents the location of Douala.

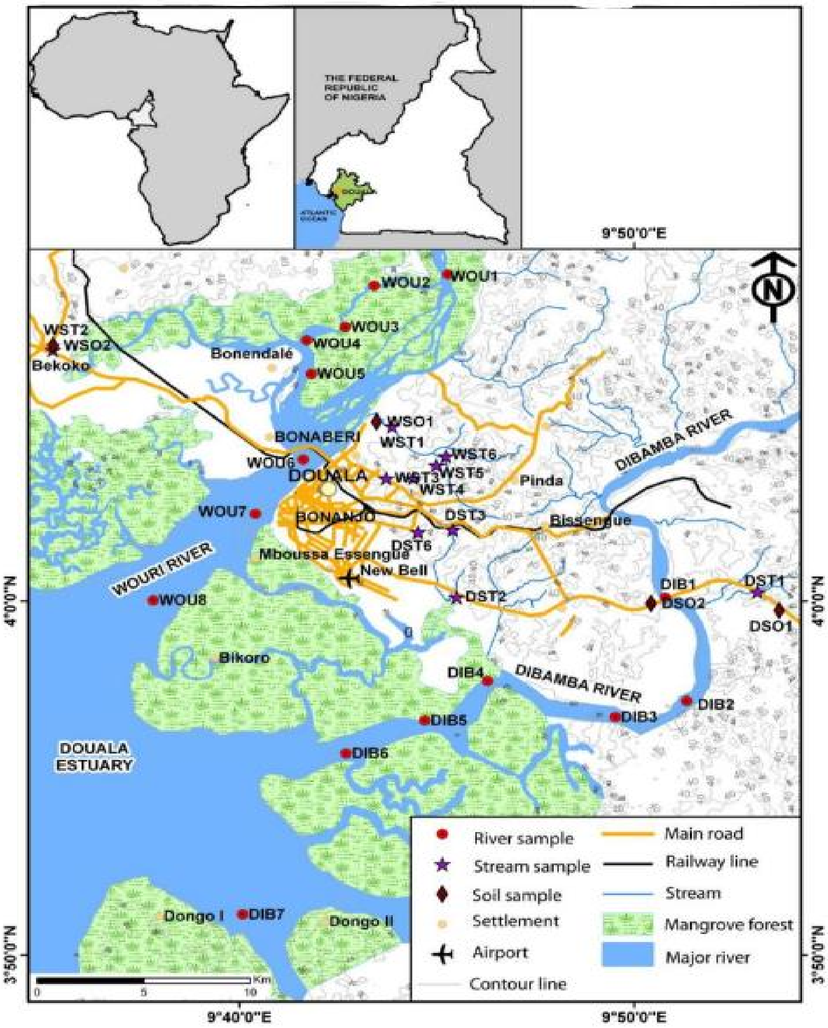


Figure 3.1. Location of Douala

3.1.2. Relief

The city of Douala is located on a sandy plateau. Its elevation ranges from 0m to about 22m above mean sea level (amsl) and on an average could be said to be 13m amsl and has a morphology which evolves from the coasts to the interior of the territory and becomes more and more rugged as one moves away from the shore. This relief consists of a set of valleys, mostly flat-bottomed.

3.1.3. Climate

Douala's location permits the existence of a tropical monsoon climate with dry season (four months) and heavy monsoon the rest of the year and no cold season. According to the Holdridge life zones system of bioclimatic classification Douala is situated in or near the subtropical wet forest biome. It has a mean annual temperature of 26.7°C which varies by 3.2°C and an average humidity of 83%. Douala sees high amounts of rainfall during the course of the year, experiencing on average roughly 3,600 millimetres of precipitation per year. Its driest month is December, when on average of 28 millimetres of precipitation falls, while its wettest month is August, when on average nearly 700 millimetres of rain falls [17]. Figure 3.2 gives an overview of the climate of Douala.

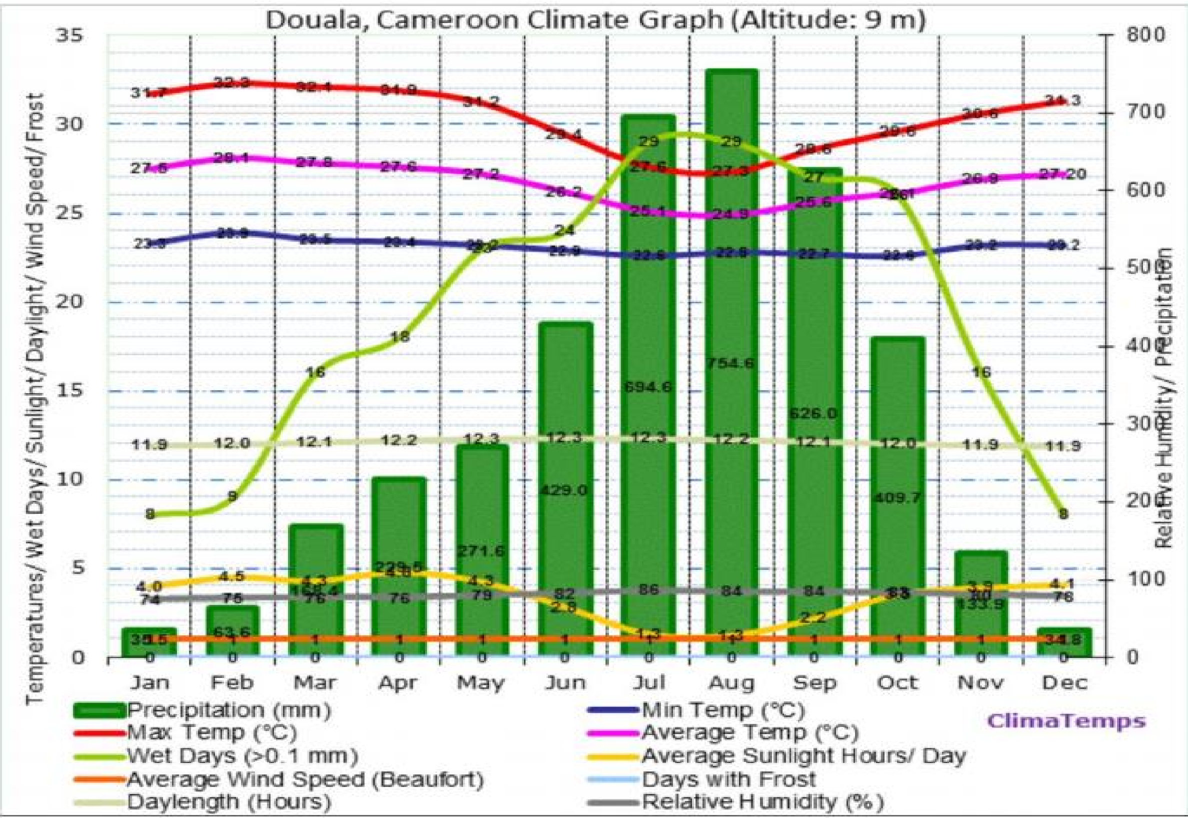


Figure 3.2. Douala climate graph (climatemps.com, 2009-2017)

3.1.4. Hydrography

The river system in the city of Douala is dense. It consists of a main river, the Wouri, framed by the Sanaga, the Dibamba, the Moungo and Nyong. The city is divided into several watersheds: Good races, Epolo, Mbanya MbopiBologo, Ngoua, Lonmayagui, Kambo, TongoBassa and Beseke.

3.1.5. Geology

The Douala Basin has a roughly triangular shape. The basin is monoclonal, with no apparent tectonics. The series visible in outcrop is about 2,400 metres thick, but the isobaths of the magnetic basement indicate nearly 8,000 metres of sediment in Kwa-Kwa Trench. This series extends from the Albo-Aptian to the Quaternary, the only notable gap being that of the upper Eocene. Rather continental and detrital in the near swimming of the outcrops, the facies become frankly clayey and marines out to sea [18]. Figure 3.3 shows the stratigraphy of the Douala basin.

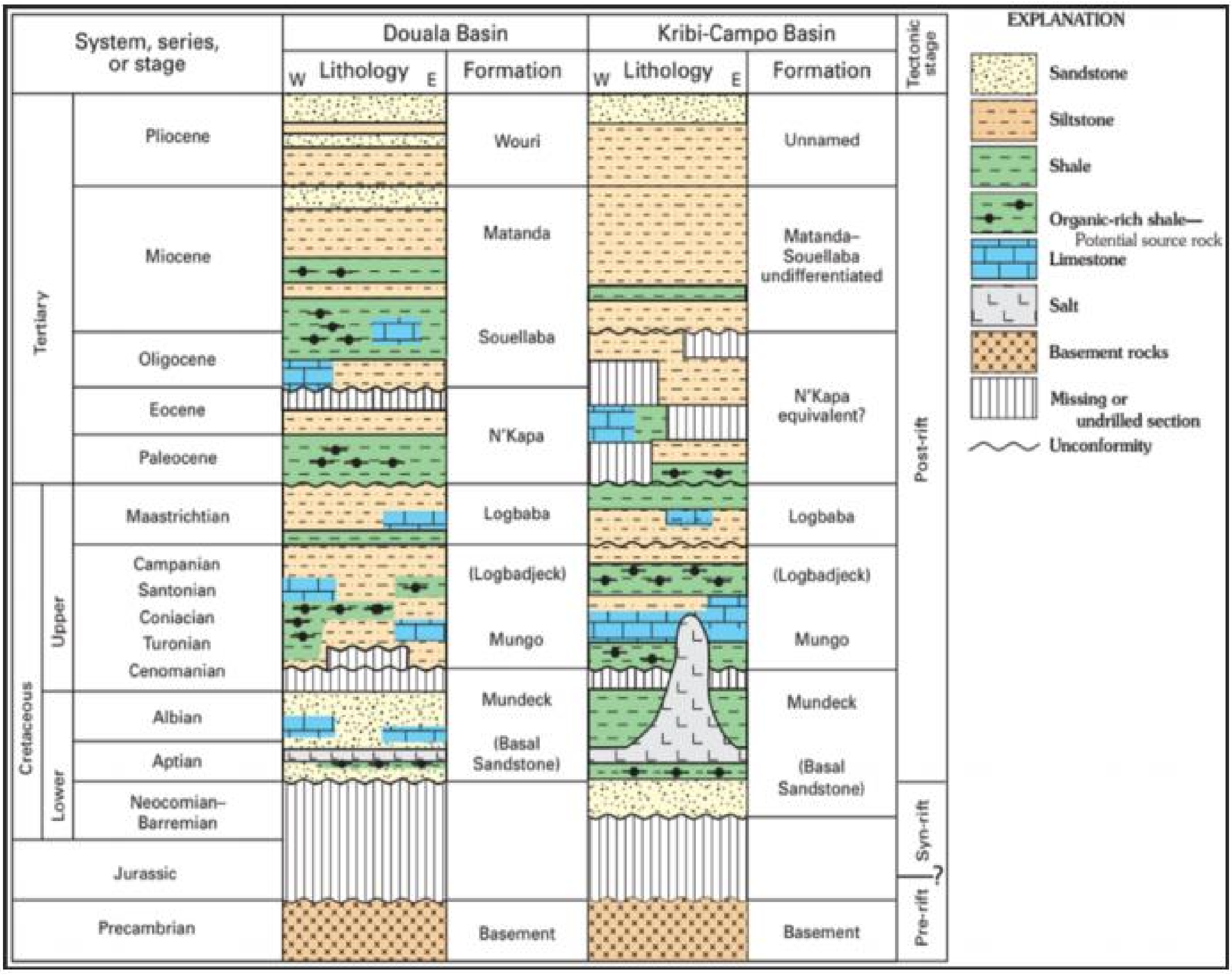


Figure 3.3. Stratigraphy of the Douala basin (link.springer.com).

3.1.6. Economically

Douala is a city with a modest oil resource but is in excellent agricultural condition, therefore it has one of best economies in Africa. However, it also faces some problems like other cities of underdeveloped countries such as heavy civil service and bad climate (flood, tornado, and storm) that affect business activities. The main economic parameters are:

- GDP: $42.2 billion (2006 EST.)
- GDP growth rate: 4.1% (2006 EST.)
- Exports-partners: Spain 17.3%, Italy 13.8%, France 9.5%, South Korea 8.1%, UK 8.1%,

Netherlands 7.9%, Belgium 4.9%, US 4.3% (2005)

- Imports - partners: France 21%, Nigeria 15%, Belgium 6.3%, China 5.6%, US 5.1%, Thailand 4.5%, Germany 4.2% (2005)

Even though Douala is the economic centre of Cameroon, a large percentage of its inhabitants live below the poverty line. Recent data shows that about thirty percent of the population lives in poverty [19] while the aforementioned percentage is doubled for rural regions. Nevertheless, about 80% of industries in Cameroon are found in this area making it a highly industrialized town.

3.2. Presentation of data

The data presented here are both the geotechnical data (gotten from in situ and laboratory tests) and structural data (obtained from research of from documents).

3.2.1. Presentation of geotechnical data

Geotechnical data are results obtained from in situ and laboratory tests coupled with some data obtained from research from the literature. The results obtained from geotechnical investigations are presented on annexe 13. From these results, we notice a stratigraphy composed mainly of three (03) main types of soils. These soils are sands, silts and clays with a predominance of sands and clays.

On the 80 metres (depth of investigations), clays and silts cover about 62% while the rest is occupied by sands. The data used in this work is up to a depth of 45 metres. Table 3.1 shows a summary of the results from the pressuremetric test for sands (up to about 33 metres) while table 3.2 shows those for clays (up to 45 metres).

Table 3.1 Data description from pressumeter test results for sands

Data	Minimum value	Maximum value	Arithmetic mean value	Standard deviation value
Limit pressure Pl* [MPa]	0,22	3,37	1,06	0,75
Pressuremetric modulus Em [MPa]	4,77	27,64	14,46	6,52

Table 3.2. Data description from pressumeter test results for clays

Data	Minimum value	Maximum value	Arithmetic mean value	Standard deviation value
Limit pressure Pl* [MPa]	1,59	9,72	5,34	2,87
Pressuremetric modulus Em [MPa]	11,99	216,48	109,62	64,92

3.2.2. Presentation of structural data

The structure is a 22 storey concrete building with the 2D distribution of its load bearing structure presented in figure 3.4 and its structural 3D framing presented on figure 3.5. The structure covers an effective area of 544 square metres with 48 columns.

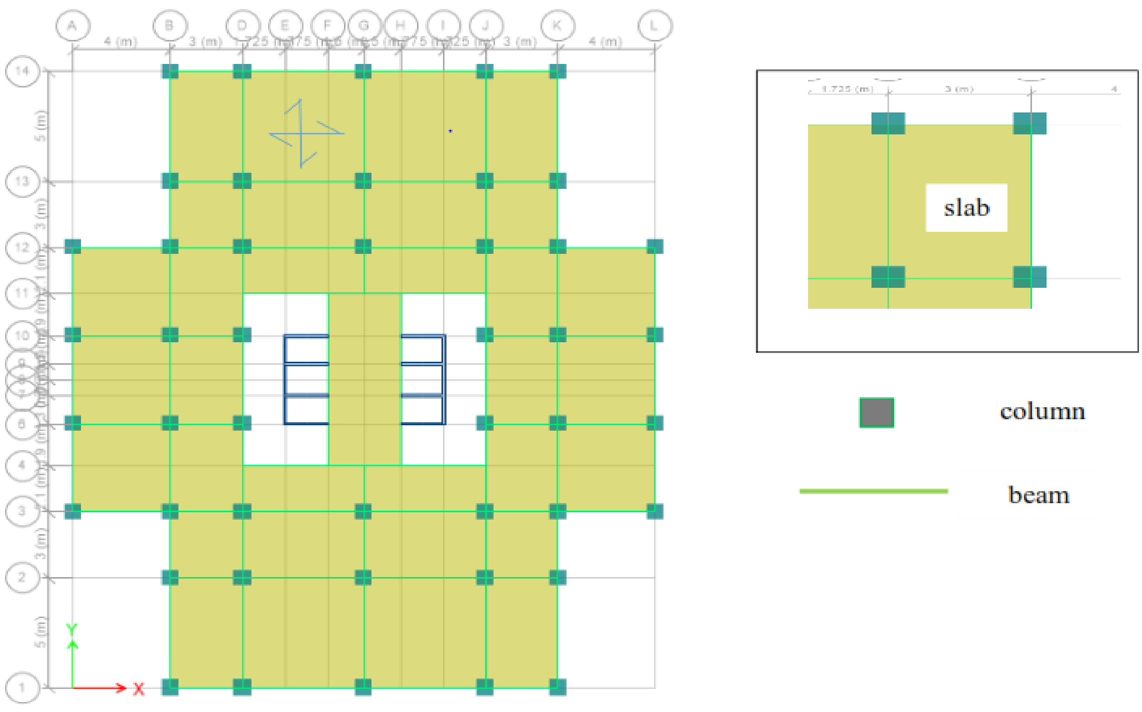


Figure 3.4 Structural concrete frame [20]

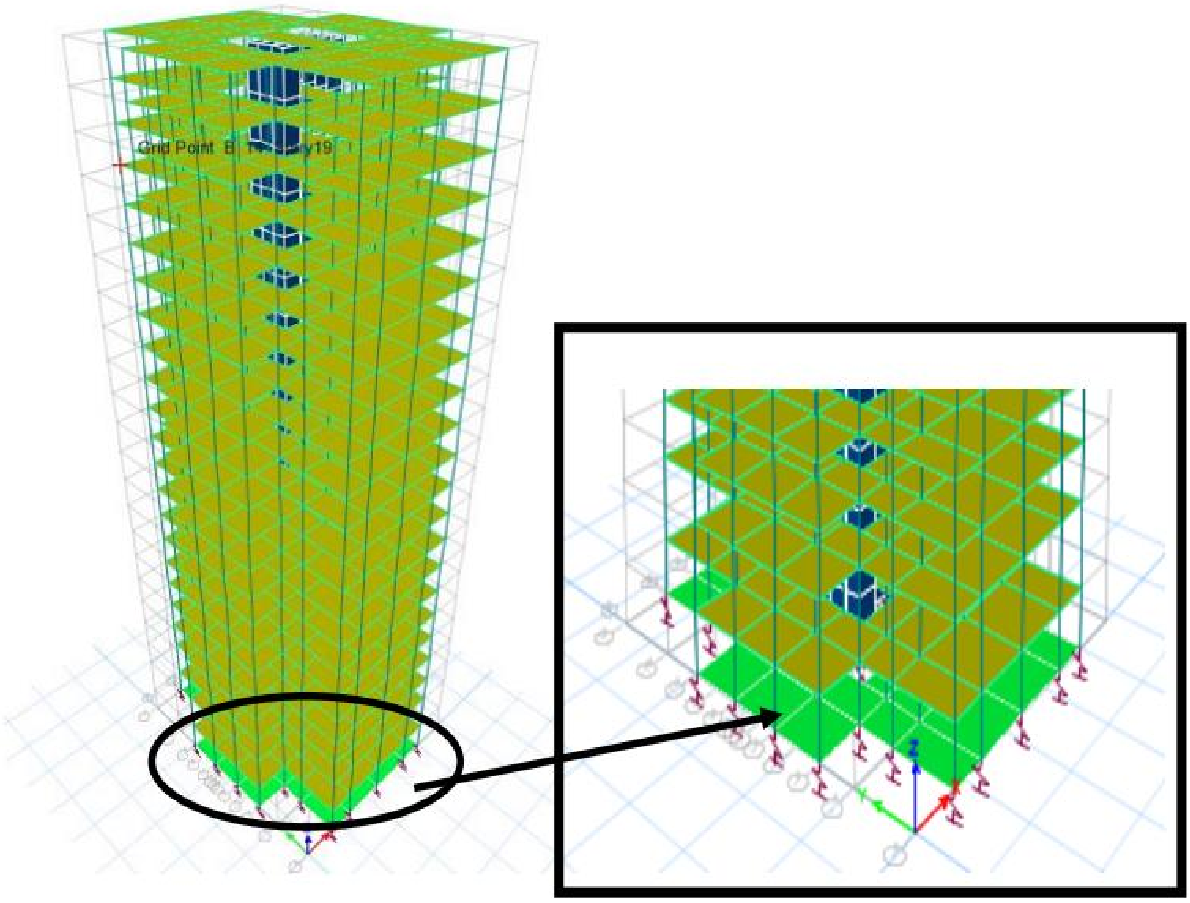


Figure 3.5. Structural 3D framing with subgrade reaction of soils [20]

Solicitations on the ground floor has been summarised on table 3.2 while those of the subbase floor on table 3.3. In

Table 3.3. Design Axial Force & Biaxial Moment for NEd MEd2 MEd3 Interaction for subbase floor [20].

Column End	**Design N_{Ed} [kN]**	**Design M_{Ed2} [kN-m]**	**Design M_{Ed3} [kN-m]**	**Station Loc [mm]**	**Controlling Combo**
Top	6202	459	577	3450	1.35G+1.5Wy+1.05Q
Bottom	6260	459	578	0	1.35G+1.5Wy+1.05Q

3.3. Results from design of foundations

The design of foundations has been made analytically from Menard pressuremeter test results for piles and with the finite element method for piles and reinforced raft using PLAXIS 2D.

3.3.1. Results from analytical design of piles

Results of the analytical design of pile gives a 31 metres pile of diameter 1.0 metre. These results have been summarised on table 3.3, 3.4 and 3.5. Table 3.3 presents the summary of results of the contribution of the base of the pile to the bearing capacity.

Table 3.4. Summary of results for the contribution of base to bearing capacity

Diameter (B)	a	b	Net equivalent limit pressure (p_{le}*)	h_D	Effectif embedded depth (D_{ef})	Coefficient of pressuremetric bearing capacity (k_p)	Base resistance (q_b)	Base bearing capacity (Q_b)
[m]	[m]	[m]	[MPa]	[m]	[m]	[ ]	[kPa]	[kN]
1.00	0.50	0.50	5.76	10	2,68	1,04	6064,75	4763,24

The results of the contribution of the shaft of the pile to bearing capacity is summarised on table 3.4.

Table 3.5. Summary of results for the contribution of shaft to bearing capacity

Depth (D)	Diameter (B)	Shaft perimeter (Ps)	Shaft bearing capacity (Q_s)
[m]	[m]	[m]	[kN]
31,00	1,00	3,14	4024,10

The summary of all the different verifications made both at ULS and SLS during the pile design from the Menard pressuremeter test is presented on table 3.5.

Table 3.6. Results and verifications from pile design

	ULS	SLS
	Persistent and transient situations	Quasi-permanent situations
Verification value	$F_{c\,;\,d}$ (kN)	F_d (kN)
	6260	3700
Designed value	$Q_{c\,;\,d}$ (kN)	$Q_{c\,;\,cr\,;\,d}$ (kN)
	6315,02	3735,89
Appreciations	Ok	Ok

3.3.2. Results from finite element design

These results have been grouped in two sections. Firstly, the results from the design of piles and next, the results from the design of reinforced raft.

3.3.2.1. Results from finite element for pile design

The output subprogram displaces all the results obtained. The generated mesh produced is presented on figure 3.6.

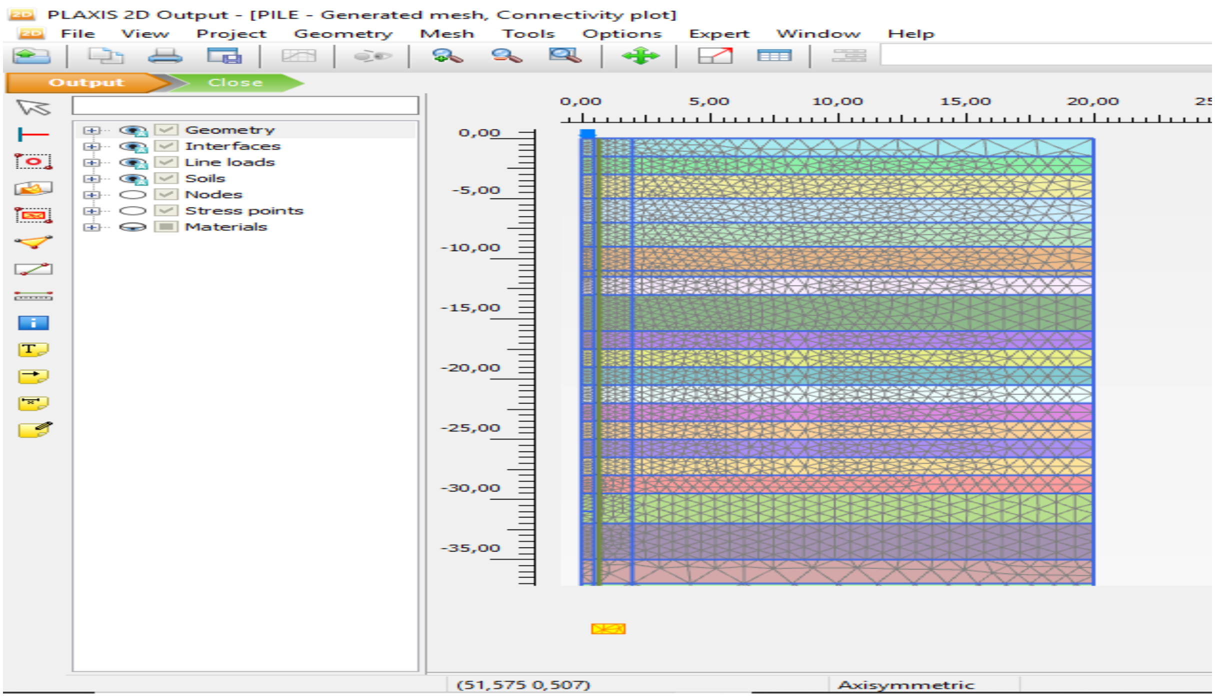


Figure 3.6. Mesh generated-connectivity plot

The initial phase defines the different soil stratigraphy with the phreatic level. In this phase, there is neither the pile nor the loads transmitted by the structure as shown by figure 3.7.

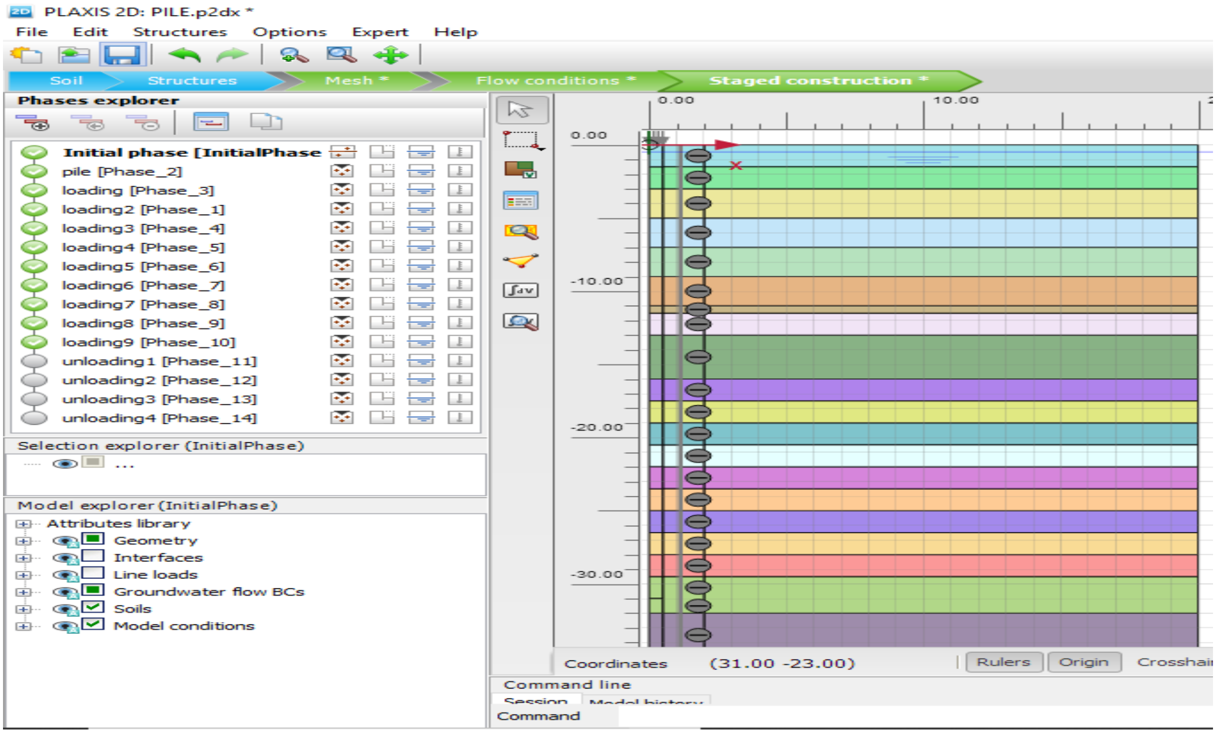


Figure 3.7. Initial phase showing soil stratigraphy

The next phase is the pile construction phase. This phase represents the pile installed in the ground as shown by figure 3.8.

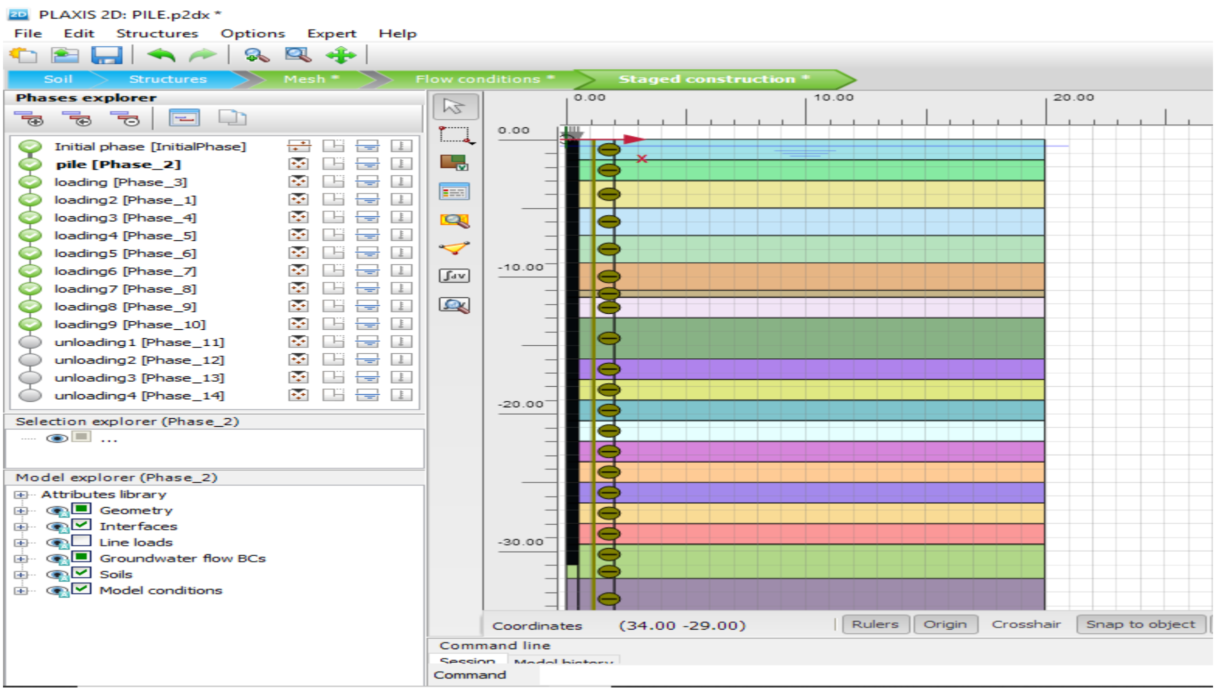


Figure 3.8. Pile phase showing pile in soil

After pile installation, the different line loads were added producing the respective settlements presented on table 3.6 corresponding to loading phases.

Table 3.7. Loads against displacement for pile head

Load (kN)	Load increments (kN)	Line loads (kN/m/m)	displacement (mm)
3735,89	0,00	0,00	0,00
	747,18	951,34	-1,40
	1494,36	1902,67	-2,85
	2241,53	2854,01	-4,44
	2988,71	3805,35	-6,08
	3735,89	4756,68	-7,88
	4483,07	5708,02	-9,85
	5230,25	6659,36	-12,04
	5977,43	7610,69	-19,09

These results are also plotted on figure 3.9. We notice that these results are widely acceptable since it gives satisfactory settlements.

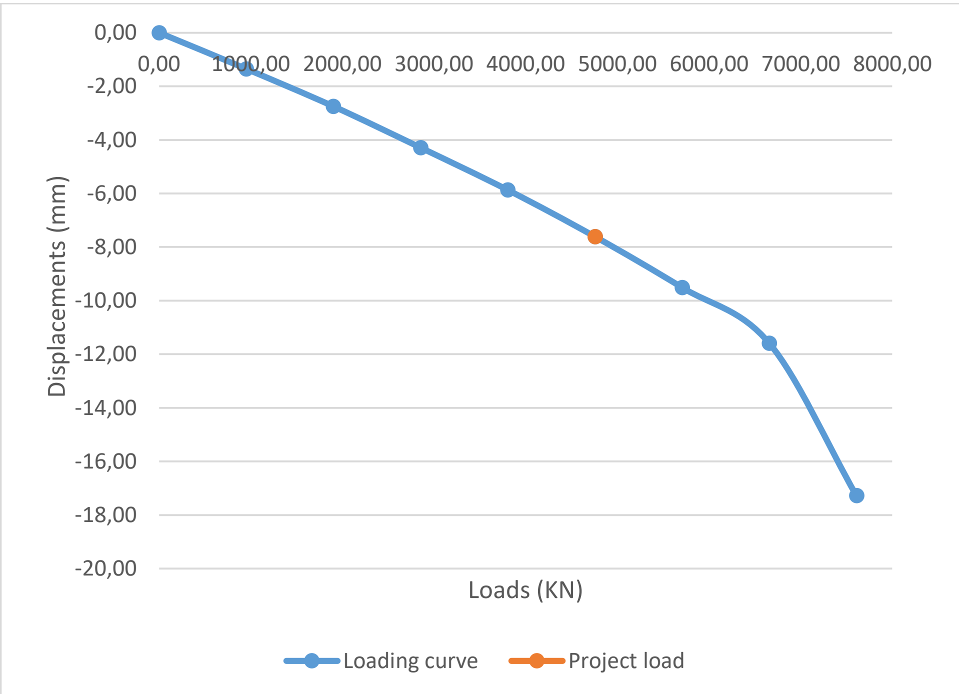


Figure 3.9. Load-displacement curve

The variation of the axial load with depth is shown on figure 3.10. This enables to determine the proportion of the applied load consumed by the shaft and that used up by the base of the pile. The load used for this curve is that at SLS.

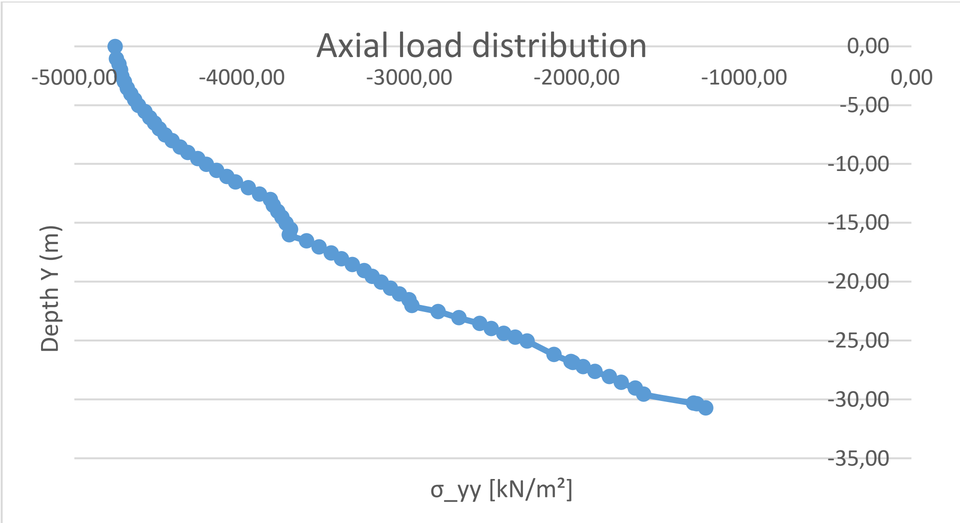


Figure 3.10. Axial load distribution on pile

3.3.2.2. Results from design of raft reinforced with rigid inclusions

Following the calculations, all the results obtained were displaced in the output subprogram. The generated mesh produced is presented on figure 3.11.

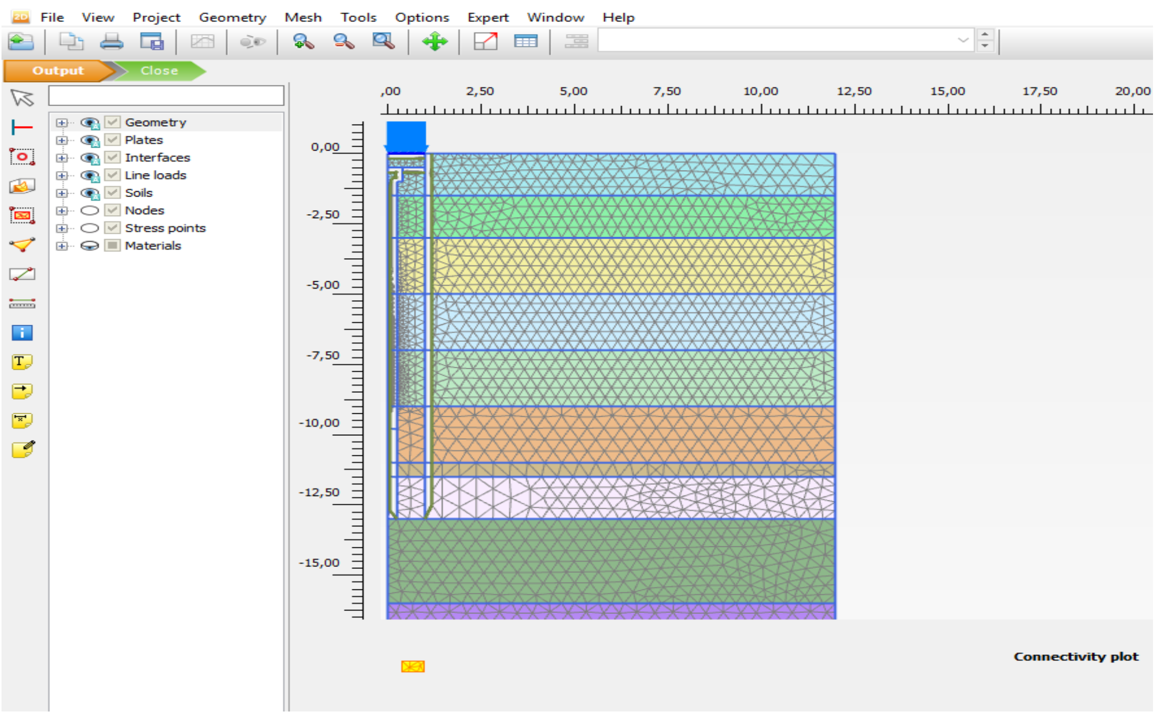


Figure 3.11. Generated mesh for design (connectivity plot)

The initial phase is the same as that of pile design presented on figure 3.7. The next phase is the construction phase which involves the construction of the rigid inclusion, the load transfer platform and the raft. This phase (construction phase) is followed by the addition of the different line loads (figure 3.11), the respective settlements presented on table 3.7 corresponding to the incremental loading phases were obtained.

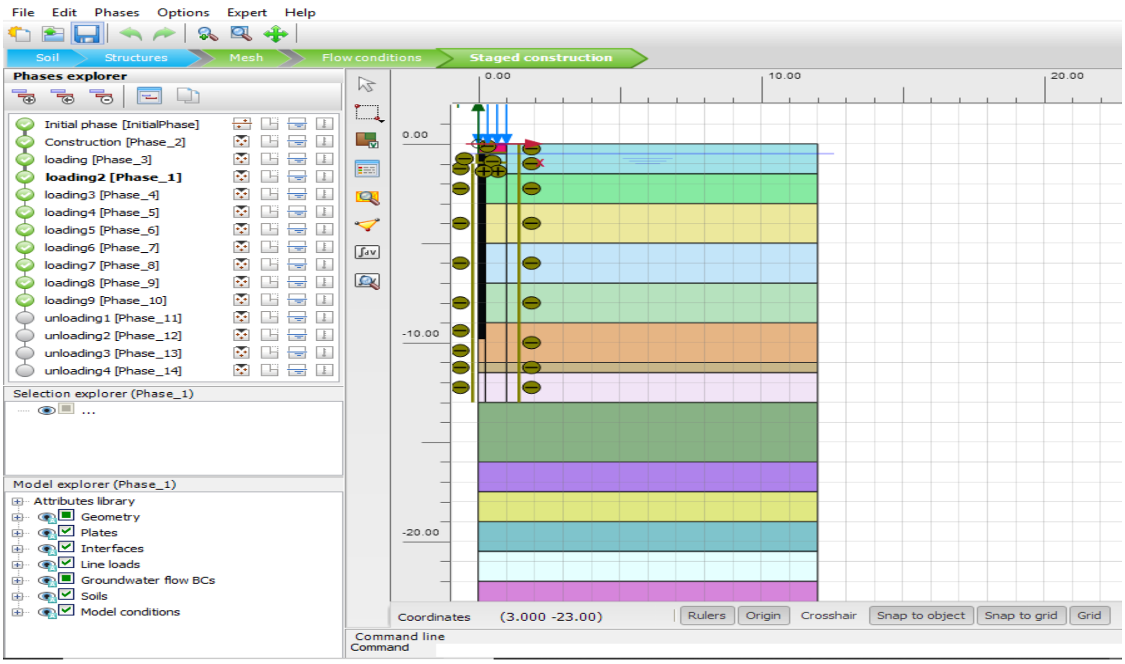


Figure 3.12. Phases of design of reinforced raft

Table 3.8. Loads against displacements for reinforced raft

Loads (kN)	Load increments (kN)	Line loads (kN/m/m)	Displacement (mm)
	0,00	0,00	0,00
	747,18	29,89	-0,91
	1494,36	59,77	-2,16
	2241,53	89,66	-3,74
3735,89	2988,71	119,55	-5,49
	3735,89	149,44	-7,67
	4483,07	179,32	-10,99
	5230,25	209,21	-15,76
	5977,43	239,10	-22,76

These results are also plotted on figure 3.13. We notice that these results are widely acceptable since it gives satisfactory settlements (less than 25 mm [32]).

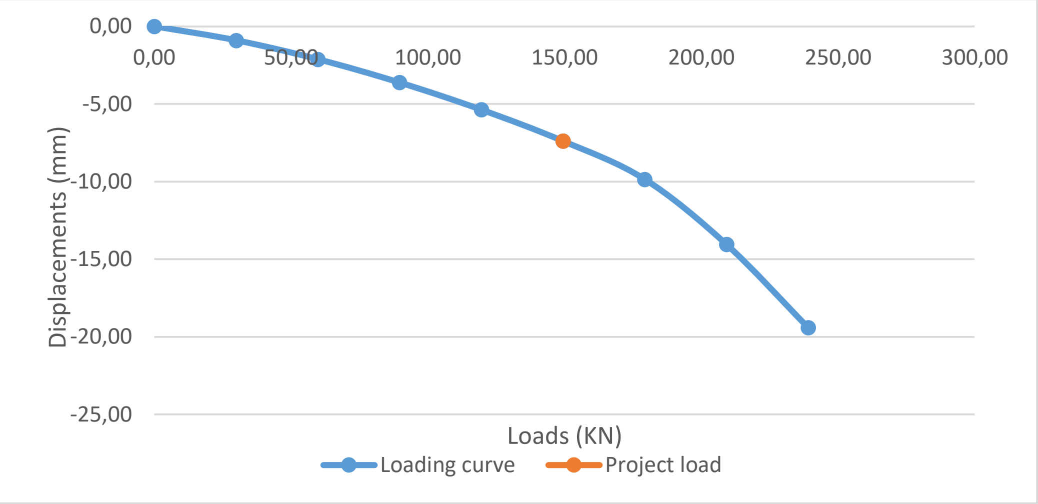


Figure 3.13. Loads-displacement curve for reinforced raft

3.4. Comparative analysis

The comparative analysis is done based on the comparison criteria; stability, cost and duration of the construction of the foundation.

3.4.1. Stability

The stability of the different foundations is evaluated in the course of this work, using the settlement analysis for various loads. The results from this criteria of comparison are well displaced on table 3.8 and figure 3.14.

Table 3.9. Loads against displacements of pile and reinforced raft

Loads (KN)	Load increments (KN)	Pile displacement (mm)	Raft displacement (mm)
3735,89	0,00	0,00	0,00
	747,18	-1,40	-0,91
	1494,36	-2,85	-2,16
	2241,53	-4,44	-3,74
	2988,71	-6,08	-5,49
	3735,89	-7,88	-7,67
	4483,07	-9,85	-10,99
	5230,25	-12,04	-15,76
	5977,43	-19,09	-22,76

We also observe that both piles and reinforced raft show satisfactory results when talking about settlements since for all the loading procedures, the settlements produced are satisfactory (less than 25 mm which is the minimum of the allowable differential settlements [32]).

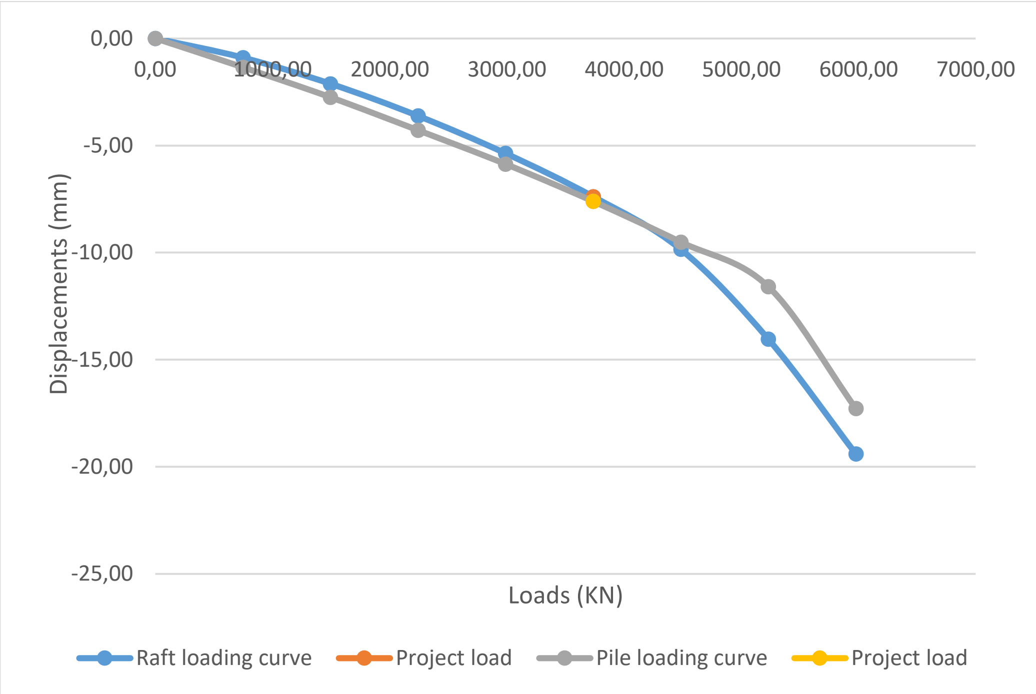


Figure 3.14. Loads against displacements of pile and reinforced raft

From figure 3.14, we conclude that the reinforced raft is more stable than pile for loads less than or equal to the actual load at SLS while as loads become greater than the actual load at SLS, pile becomes more stable.

3.4.2. Requirements for the construction

The construction of piles is known to be more complicated than that of reinforced rafts. This can be explained by the fact that, drilling to very high depths (at least 31 metres in this case) needs very performant drilling instruments which are not only rare in Cameroon, but are also very expensive and needs the presence of a specialist permanently for its construction. Consequently, the adaptability of raft foundation is an advantage since it needs lesser requirements.

3.4.3. Duration of foundation construction

Assuming the availability of financial, material and human resources and the efficient use of those resources, the duration of construction of piles is generally longer since it needs the mobilisation of more material resources. Drilling to high depths generally needs more time than excavating lower depths making it more time consuming to construct piles than reinforced rafts.

3.4.4. The cost of the foundations

Since the costs of transports and logistics are quite complicated to evaluate, the costs of construction of foundations are evaluated in the course of this work based on the cost of excavations and cost of reinforced concrete used for the different foundations. These unit costs are those used in the national market since they are obtained from ''mercurial 2020''. Table 3.9 presents the cost of construction of piles while table 3.10 presents the cost of production of the reinforced raft.

Table 3.10. Cost of pile foundations

Piles		
Diameter	m	1,00
Area	m2	0,79
Heigth	m	31,00
Unit volume	m3	24,35
Number	-	48,00
Total volume	m3	1164,67
Unit cost of excavation	FCFA/m3	4500,00
Unit cost of reinforced concrete	FCFA/m3	220000,00
Total cost of foundation	**FCFA**	**262366968,90**

Table 3.11. Cost of reinforced raft foundation

	Units	Raft	Load Transfer platform (LTP)	Rigid inclusion
Diameter	m	-	-	0,50
Thickness (heigth)	m	0,80	0,50	8,00
Length (Lx)	m	29,00	30,60	-
Width (Ly)	m	25,00	26,60	-
Area	m2	725,00	813,96	-
Area of influence	m2	597,00	813,96	-
Volume of rigid inclusion head	m3	-	-	0,32
Unit volume	m3	477,60	406,98	1,89
Number of rigid inclusions	-	-	-	294,00
Total volume	m3	477,60	406,98	555,89
Volumetric weigth	kN/m3	-	20,00	-
Mass	kg	-	829724,77	-
Unit cost of excavation	FCFA/m3	4500,00	4500,00	4500,00
Unit cost of reinforced concrete	FCFA/m3	220000,00	-	220000,00
Unit cost of stones	FCFA/ton	-	5000,00	-
Total cost	FCFA	107221200,00	5064328,85	124798230
Total cost of foundation	**FCFA**	**237083758,80**		

We can observe that reinforced raft foundation is cheaper than pile foundation, even more since the logistic for piles is more important than that used for reinforced raft.

CONCLUSION

The main objective of this chapter was to present the results obtained from the analysis of the different foundation modes and to used them for a comparative analysis. This objective has been achieved through five (05) major steps. The first was the general presentation of the project site followed by the presentation of the project data (structural and geotechnical). Then, the presentation of results obtained from the design of the pile and reinforced raft foundations. Finally, the comparison of the two foundation modes according to the stability, requirements for the construction of the different foundations, the duration of construction and the cost as comparison criteria.

General conclusion

The main objective of this work was to show by the use of a comparative analysis that an alternative method (satisfying both the economic and technical standards) to the use of piles in the construction of a tall building in Douala can be adopted. In order to face the challenge, an analysis was made in three chapters.

The first chapter is a literature review on compressible soils, the types of foundations adapted to compressible soils and the different soil improvement methods. The second chapter describes the methodology used in conducting the site visit, in acquiring the geotechnical and structural data. The data used in the course of this work has been obtained from documentary research, Menard pressuremeter test and some laboratory tests. The methods used to design the different foundation modes are the analytical method and the use of the finite element method using PLAXIS 2D V20. Moreover, in chapter 2, the different comparison criteria were outlined. These criteria were the stability, the requirements for the construction of the different foundation modes, the duration of construction and the cost of the foundations. On the last chapter a general presentation of Douala, a presentation of the geotechnical and structural data, results from the design of the different foundations and the comparisons between the systems of foundations were made based on the aforementioned criteria.

Following the different analysis, it was found that both the piles and the raft coupled with rigid inclusions are quite stable. Secondly, the construction of piles is more difficult than that of the reinforced raft. Moreover, it was observed that more time is needed to construct piles than to construct the reinforced raft. The cost of piles is also greater than that of rigid inclusions coupled with rafts. These comparisons lead to the conclusion that rigid inclusions coupled with raft can be used as an alternative solution to piles for the construction of a tall building on a compressible soil.

However, this study could be made better if more geotechnical tests, in particular laboratory tests were made. In order to make further analysis, it will be interesting to:

- To make further analysis on the results when designing piles as a group in order to have an idea on the depth of influence.
- An analysis to evaluate the liquefaction susceptibility due to the high level of the phreatic level and the presence of high thicknesses of depths.

- An analysis in 3 dimensions (eventhough it requires more time for modelling and computations) to obtain more information on the different behaviours of the foundation modes.

ANNEXES

ANNEXE 1: Base resistance in sand from CPT

Base resistance in sand from CPT

Pile type	c_p	*Notes*	*Source*
Displacement	$0.35 \div 0.5$	Database of high quality pile load tests	Chow (1997)
	$0.20 \div 0.35$	Computed	Lee and Salgado (1999)
	$0.32 \div 0.47$	Test data	
	0.4	Reinterpretation of the Chow (1997) data	Randolph (2003)
	0.4 for Franki piles 0.57 for precast concrete piles	Data from pile load test; Q_{lim} by Van der Veen's criterion	Aoki and Velloso (1975)
Replacement	0.2	Load tests on drilled shafts	Franke (1989)
	0.13 ± 0.02	Calibration chamber load tests	Ghionna *et al.* (1994)
	$0.23 \exp(-0.0066 D_R)$	FEM analyses and calibration chamber tests	Salgado (2006)
	$0.20 \div 0.26$	Test data	Lee and Salgado (1999)

ANNEXE 2 : Base resistance in clay from CPT

Base resistance in clay from CPT

Pile type	c_p	*Notes*	*Source*
Displacement	$0.9 \div 1.0$	Soft to lightly OC clays	State of the art
	0.35 for driven piles 0.30 for jacked piles	Stiff clays	Price and Wardle (1982)
Replacement	0.47 for pure clay 0.52 for silty clay 0.78 for silty clay with sand 0.71 for sandy clay with silt 0.83 for sandy clay	Medium to stiff clays	Aoki and Velloso (1975) Aoki *et al.* (1978)
	0.34 for pure clay and silty clay 0.41 for silty clay with sand and sandy clay with silt 0.66 for sandy clay	Medium to stiff clays	Lopes and Laprovitera (1988)

ANNEXE 3 : Shaft resistance in sand from CPT

Shaft resistance in sand from CPT

c_s	*Source*
0.008 for open ended steel pipe piles 0.012 for precast concrete and closed-ended steel pipe piles	Schmertmann (1978)
0.004 ÷ 0.006 per $D_R \leq 50\%$ 0.004 ÷ 0.007 per $50\% < D_R \leq 70\%$ 0.004 ÷ 0.009 per $70\% < D_R \leq 90\%$ Closed-ended pipe piles	Lee *et al.* (2003)
0.0040 for clean sand 0.0057 for silty sand 0.0069 for silty sand with clay 0.0080 for clayey sand with silt 0.0086 for clayey sand Driven piles: for Franki piles: multiply number above by 0.7 For drilled shafts: multiply number above by 0.5	Aoki and Velloso (1975) Aoki *et al.* (1978)
0.0027 for clean sand 0.0037 for silty sand 0.0046 for silty sand with clay 0.0054 for clayey sand with silt 0.0058 for clayey sand Replacement piles	Lopes and Laprovitera (1988)
0.0034 ÷ 0.006 This method uses a corrected value of cone resistance $q_c - u$, where u is the pore pressure at the depth considered	Eslami and Fellenius (1997)

ANNEXE 4 : Shaft resistance in clay from CPT

Shaft resistance in clay from CPT

c_s	*Source*
0.074 ÷ 0.086 for sensitive clay 0.046 ÷ 0.056 for soft clay 0.021 ÷ 0.028 for silty clay or stiff clay Driven piles This method uses a corrected value of cone resistance $q_c - u$, where u is the pore pressure at the depth considered	Eslami and Fellenius (1997)
0.025 Displacement piles	Thorburn and MacVicar (1971)
0.017 for pure clay 0.011 for silty clay 0.0086 for silty clay with sand 0.0080 for sandy clay with silt 0.0069 for sandy clay Driven piles: for Franki piles: multiply number above by 0.7 For drilled shafts: multiply number above by 0.5	Aoki and Velloso (1975) Aoki *et al.* (1978)
0.012 for pure clay 0.011 for silty clay 0.010 for silty clay with sand 0.0087 for sandy clay with silt 0.0077 for sandy clay Non displacement piles	Lopes and Laprovitera (1988)

ANNEXE 5 : Base resistance in sand from SPT

Base resistance in sand from SPT

Pile type	n_P	*Source*
Displacement	4 4.8 for clean sand 3.8 for silty sand 3.3 for silty sand with clay 2.4 for clayey sand with silt 2.9 for clayey sand For Franki piles: multiply numbers above by 0.7	Meyerhof (1983) Aoki and Velloso (1975)
	3.25 for sand 2.05 for sandy silt 1.65 for clayey silt 1.00 for clay	Decourt (1995)
Replacement	0.82 for clean sand 0.72 for sand with silt or clay	Lopes and Laprovitera (1988)
	0.6 ($p/p_A \leq 45$)	Reese and O'Neill (1989)
	1.9 for CFA piles 1.2 < for drilled shafts	Neely (1991)
	1.65 for sand 1.15 for sandy silt 1.00 for clayey silt 0.080 for clay	Decourt (1995)

ANNEXE 6 : Base resistance in clay from SPT

Base resistance in clay from SPT

Pile type	n_P	*Source*
Displacement	0.95 for pure clay	Aoki and Velloso (1975)
	1.05 for silty clay 1.57 for silty clay with sand 1.43 for sandy clay with silt 1.67 for sandy clay For Franki piles: multiply numbers above by 0.7	Aoki *et al.* (1978)
Replacement	0.47 for pure clay 0.52 for silty clay 0.78 for silty clay with sand 0.71 for sandy clay with silt 0.83 for sandy clay	Aoki and Velloso (1975) Aoki *et al.* (1978)
	0.34 for pure clay and silty clay 0.41 for silty clay with sand and sandy clay with silt 0.66 for sandy clay	Lopes and Laprovitera (1988)

ANNEXE 7 : Shaft resistance in sand from SPT

Shaft resistance in sand from SPT

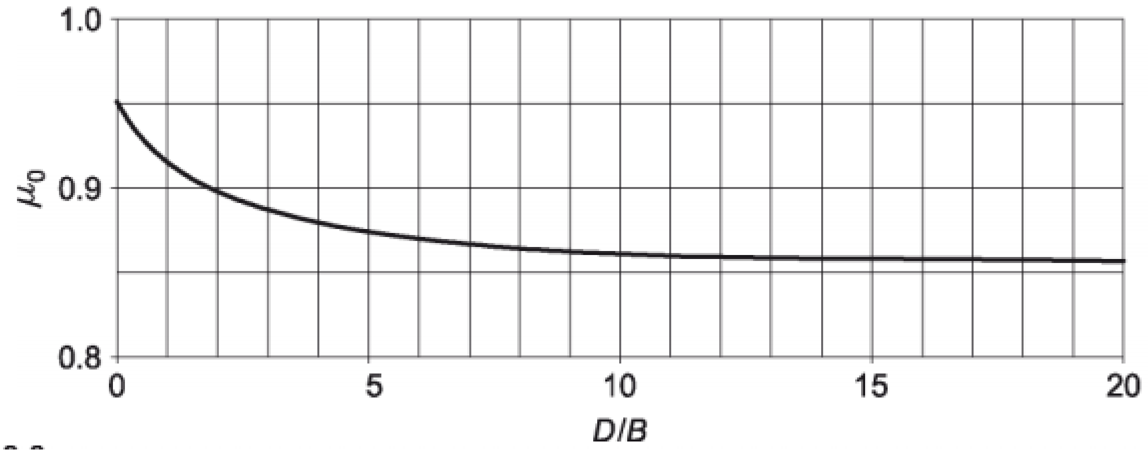
Pile type	n_s	*Source*
Displacement	0.02 ($s \leq 100$ kPa)	Meyerhof (1976) Thorburn and MacVicar (1971)
	0.02 for full displacement piles 0.01 for H piles	Meyerhof (1976, 1983)
	0.033 for sand 0.038 for silty sand 0.040 for silty sand with clay 0.033 for clayey sand with silt 0.043 for clayey sand For Franki piles: multiply numbers above by 0.7	Aoki and Velloso (1975) Aoki *et al.* (1978)
Replacement	0.01 ($s \leq 50$ kPa)	Meyerhof (1976)
	0.016 for sand 0.019 for silty sand 0.020 for silty sand with clay 0.016 fror clayey sand with silt 0.021 for clayey sand	Aoki and Velloso (1975) Aoki *et al.* (1978)
	0.014 for sand 0.016 for silty sand 0.020 for silty sand with clay 0.024 fror clayey sand with silt 0.026 for clayey sand	Lopes, Laprovitera (1988)

ANNEXE 8 : Shaft resistance in clay from SPT

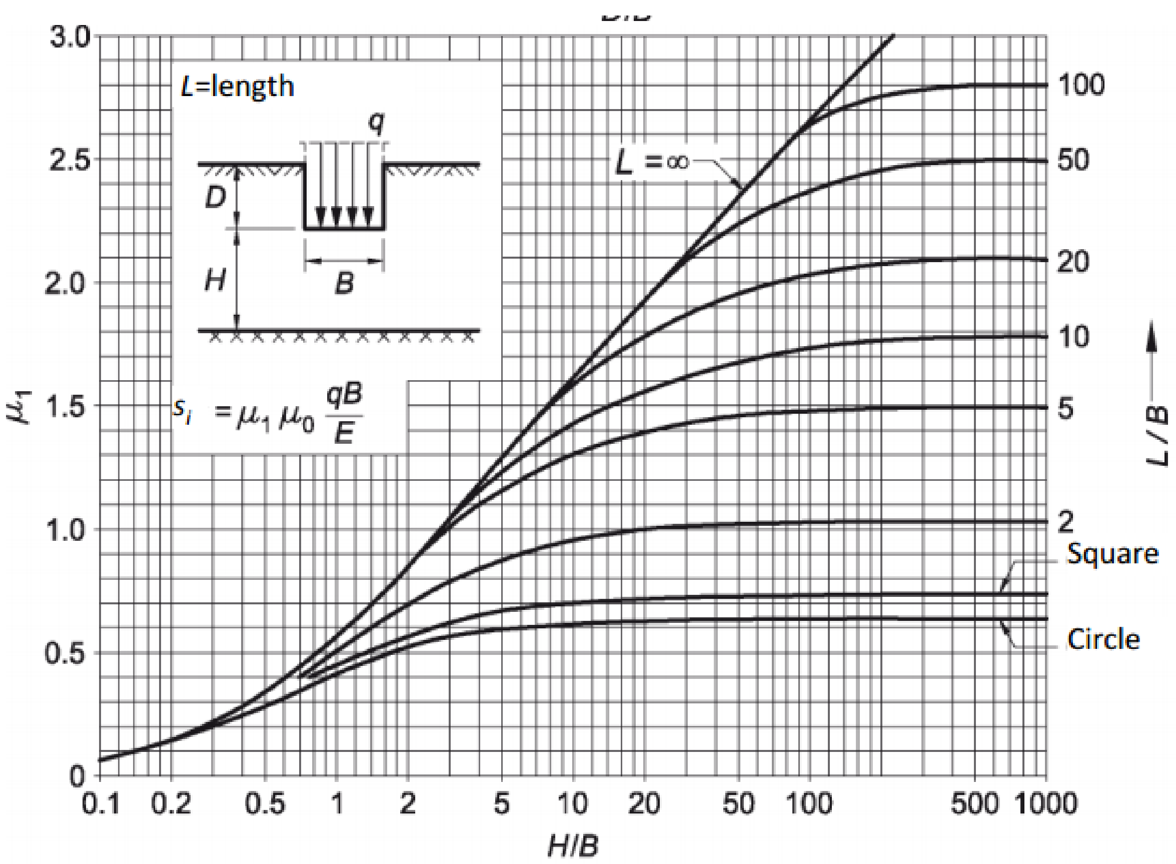
Shaft resistance in clay from SPT

Pile type	n_s	*Source*
Displacement	0.029 for clay 0.021 for silty clay 0.024 for silty clay with sand 0.020 for sandy clay with silt and sandy clay For Franki piles: multiply number above by 0.7	Aoki and Velloso (1975) Aoki *et al.* (1978)
Replacement	0.014 for clay	Aoki and Villoso (1975)
	0.010 for silty clay 0.012 for silty clay with sand 0.010 for sandy clay with silt and sandy clay	Aoki *et al.* (1978)
	0.024 for clay 0.022 for silty clay 0.024 for silty clay with sand 0.022 for sandy clay with silt 0.031 for sandy clay Non-displacement piles	Lopes and Laprovitera (1988)

ANNEXE 9 : Curve of relation between μ_0 and foundation dimensions



ANNEXE 10 : Curve of relation between μ_1 and foundation dimensions



ANNEXE 11 : Typical cement requirements for various soil types (after Anon., 1990(d))

Unified soil classification	*Typical range of cement requirement,** (% by wt)	*Typical cement content for moisture-density test* (ASTM D 558),† (% by wt)	*Typical cement contents for durability tests* (ASTM D 559 and D 506),‡ (% by wt)
GW, GP, GM, SW, SP, SM	3–5	5	3–5–7
GM, GP, SM, SP	5–8	6	4–6–8
GM, GC, SM, SC	5–9	7	5–7–9
SP	7–11	9	7–9–11
CL, ML	7–12	10	8–10–12
ML, MH, CH	8–13	10	8–10–12
CL, CH	9–15	12	10–12–14
MH, CH	10–16	13	11–13–15

ANNEXE 12 : Typical average properties of soil-cement and soil-lime mixtures (after Ingles and Metcalf, 1972

(a) Typical mean* properties of soil–cement†

Soil type (unified classification)	*Compressive strength* (MN/m²)	*Young's Modulus, E* (MN/m²)	*CBR*	*Permeability* (m/s)	*Shrinkage*	*Comments*
GW, GP, GM, GC, SW	6.5	2×10^4	>600	Decreases ($\approx 2 \times 10^{-7}$)	Negligible	Too strong; liable to wide spaced cracks‡
SM, SC	2.5	1×10^4	600	Decreases	Small	Good material
SP, ML, CL	1.2	5×10^3	200	Decreases ($\approx 1 \times 10^{-9}$)	Low	Fair material
ML, CL, MH, VH	0.6	2.5×10^3	<100	Increases	Moderate	Poor material
CH, OL, OH, Pt	<0.6	1×10^3	<50	Increases ($\approx 1 \times 20^{-11}$)	High	Difficult to mix; needs excessive cement

* Variations of approximately 50% around the mean may be expected.
† Values shown are at 10% cement content.
‡ Good material if *less cement* is used.

(b) Typical mean* properties of soil–lime†

Soil type (unified classification)	*Compressive strength* (MN/m²)	*Young's Modulus, E* (MN/m²)	*CBR*	*Permeability* (m/s)	*Linear shrinkage* (%)	*Plasticity index*	*Comments*
GW, GP, GM, GC, SW, SP	≤0.3	–	75	Increases ($\geq 10^{-7}$)	Nil	Non-plastic	Suitable only for plasticity reduction
SM, SC	1.1	$<1 \times 10^2$	50	Increases	Very low	<5	Poor to fair material
ML, CL, MH, VH	2.5	2×10^4	30	Increases	5	10	Good material
CH	3.5	1×10^3	25	Increases ($\leq 10^{-10}$)	10	20	Good effect, fair to good material
OL, OH, Pt	≤1.0	1×10^2	≤10	–	–	15	Not suitable *per se*‡

* Variations of approximately 50% around the mean may be expected.
† Values shown are at the additive level optimum for the respective soil types.
‡ Results may be improved by admixture of the lime with gypsum.

ANNEXE 13 : Results from geotechnical tests

1- Tests carried out

Sondage	Altitude locale (m)	Prof. (m/TN	Diam. Outil foration (mm)	Essais réalisés (u)	Nombre d'échantillons (u)	Long ; tubage (m)
FP7 SPT	1,99	40,4	51 puis 63	27	27 échantillons remaniés	34,5 m
FP7 SPT puis carottage	1,99	40,4 – 80	96		12 EI. + échantillons de carottes	
FP7 pressiomètre	2,48 m	80	63	53		33 m

Tableau 1 : investigations réalisées

N.B. : EI veut dire Echantillons Intacts

2- Tests materials

Sondage	Machine	Outils
Sondage Standard Penetration Test (SPT)	Sondeuse sur chenilles : GEO 305	Carottier fendu SPT ; Ø ext. 51 mm ; longueur intérieure 450 mm
Sondage carotté	Sondeuse sur chenilles : GEO 305	Ø carottes = 63,5 mm ; diamètre trou = 96 mm
Sondage rotary et pressiométrique	Foreuse sur camion type BERLIET	Tricone Ø 63 mm

Tableau 2 : Matériel d'investigation

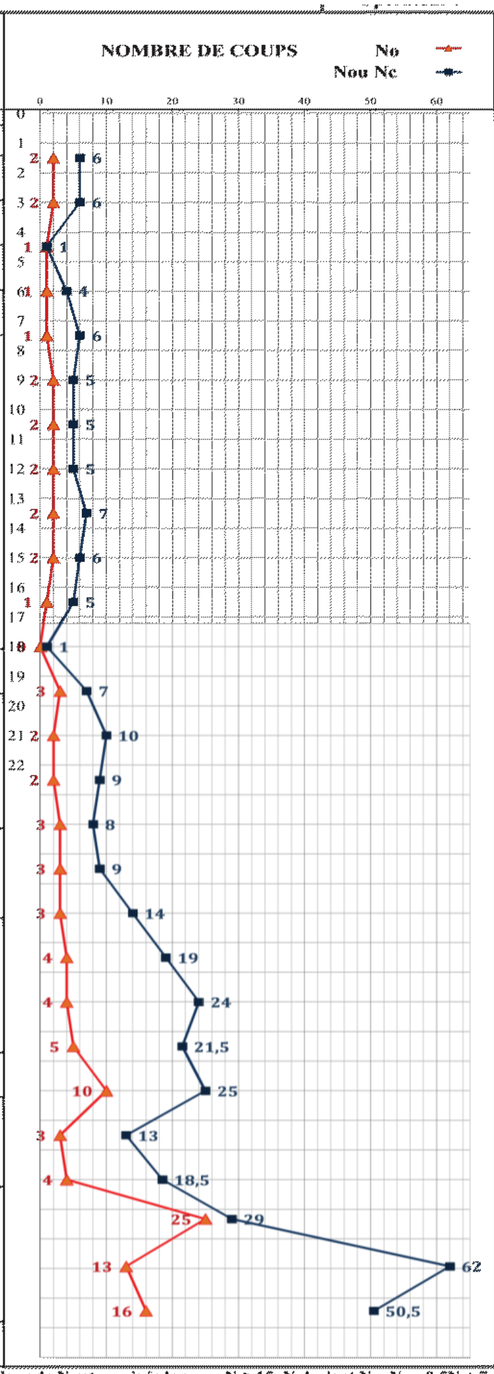
3- Types of laboratory tests carried out with their norms

Dénomination	FP 7	Total	Norme
Granulométrie par tamisage	Echantillon remanié	24	NF P94 - 056
Analyse sédimentométrique	Echantillon remanié	5	NF P94 - 057
Limites d'Atterberg	Echantillon remanié	5	NF P94 - 051
Teneur en eau naturelle	Echantillon remaniés	24	NF P94 - 050
Teneur en matière organique	Echantillon remanié	4	X31 - 109
	Echantillon carotté	4	
Teneur en carbonates de calcium	Echantillon remanié	4	X31 - 106
	Echantillon carotté	7	
Masse volumique des grains solides	Echantillon remanié	24	NF P 94 - 054
Equivalent de sable	Echantillon remanié	18	P18 - 598
Essai de compression uniaxiale	Echantillon (carotte meuble)	22	NF ¨P 94 - 077

Tableau 4 :types d'essais en laboratoire réalisés sur les échantillons (remaniés, intacts et de carottes)

4- Lithological section and profile from Standard Penetrometer Tests

Altitude	Profondeur Essai	ESSAI SPT N°	N0	N1	N2	N	Nc	Description du sol
2	0,2							Terre végétale
0,5	1,5	1	2	2	4	6	6	Sable fin jaune brun
-1	3	2	2	3	3	6	6	Sable fin jaune brun à gris
-2,5	4,5	3	1	0,5	0,5	1	1	Sable très fin micacé à sable vasard, gris noirâtre
-4	6	4	1	2	2	4	4	Vase très micacée à débris vgtx, très peu sableuse
-5,5	7,5	5	1	3	3	6	6	Vase micacée à d.v. (15 cm) et sable fin peu micacé g. c.
-7	9	6	2	2	3	5	5	Sable fin gris
-8,5	10,5	7	2	2	3	5	5	
-10	12	8	2	2	3	5	5	
-12	13,5	9	2	2	5	7	7	
-13	15	10	2	2	4	6	6	
-15	16,5	11	1	2	3	5	5	
-16	18	12	0	0,5	0,5	1	1	
-18	19,5	13	3	3	4	7	7	Sable fin à moyen avec de petits passages argileux
-19	21	14	2	5	5	10	10	Sable moyen gris avec entre 23,5 et 24,0 m, quelques éléments grossiers + débris végétaux
-21	22,5	15	2	5	4	9	9	
-22	24	16	3	4	4	8	8	
-24	25,5	17	3	3	6	9	9	Sable grossier grsi clair, graveleux entre 26,5 et 26,85 m
-25	27	18	3	6	8	14	14	
-27	28,5	19	4	9	14	23	19	Sable moyen blanchâtre
-28	30	20	4	14	19	33	24	
-30	31,5	21	5	10	18	28	21,5	
-31	33	22	10	15	20	35	25	Sable grossier + gravier blanchâtre
-33	34,5	23	3	5	8	13	13	Argile grise
-34	36	24	4	9	13	22	18,5	
-35	37,3	25	25	43		43	29	Argile raide grise
-37	38,9	26	13	44	65	109	62	
-38	40,4	27	16	35	51	86	50,5	
-40	41,9	FIN DU SONDAGE A 40,43 m						



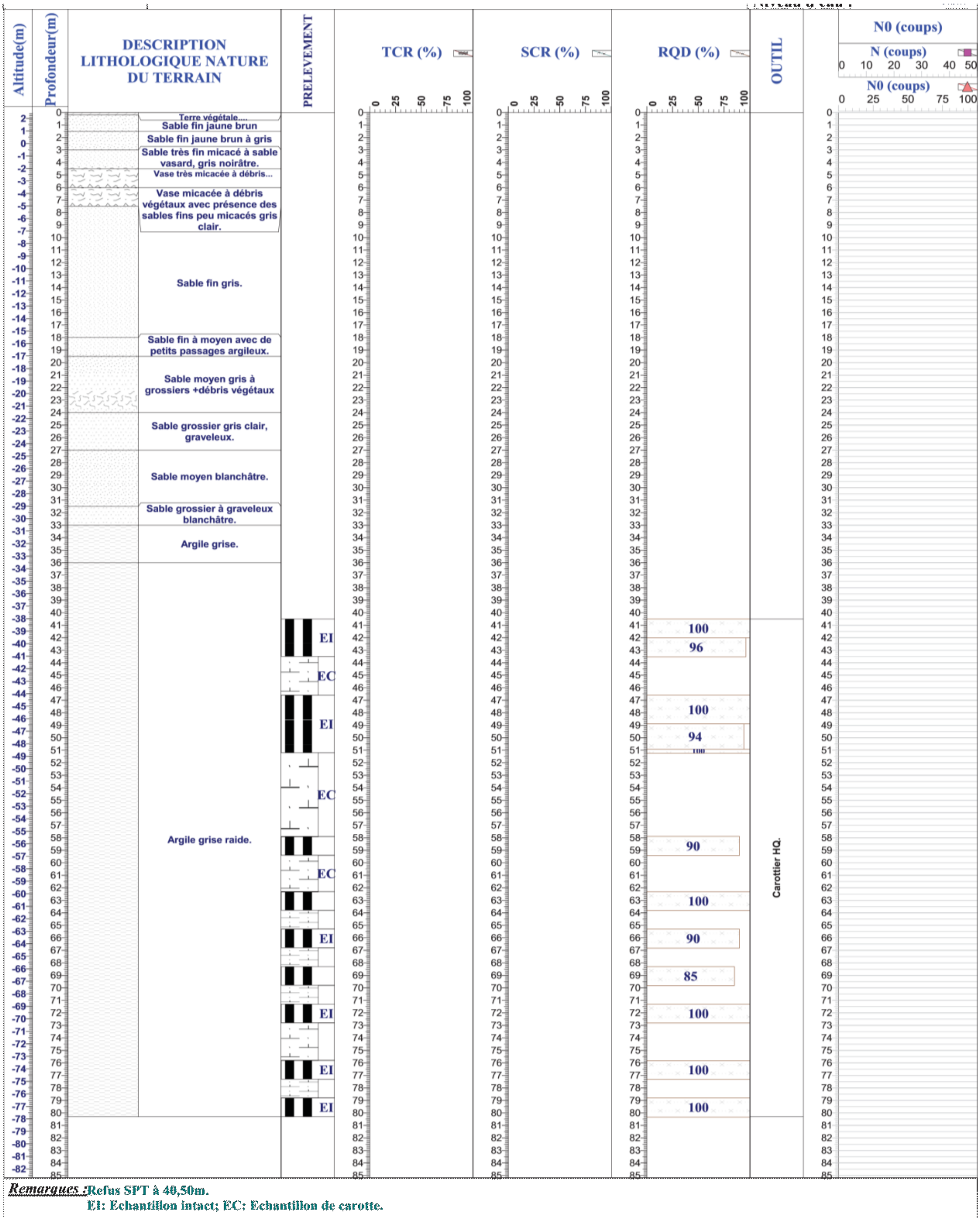
Outil: Tricône dia.63 mm tous les 105 premiers cm de chaque mètre et Carro ier SPT les 45 derniers cm de chaque 1,5 m

Tubage: TUBAGE HQ

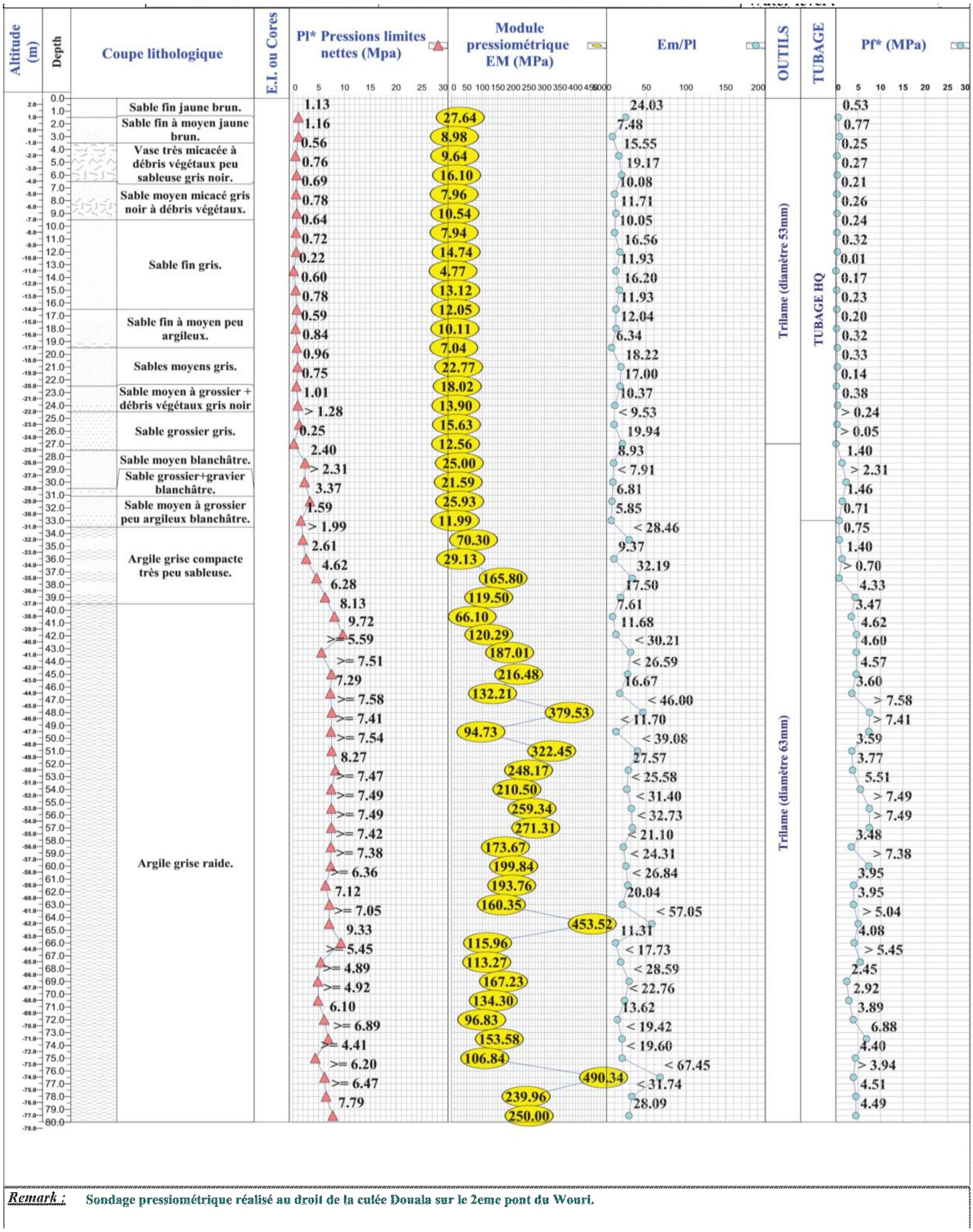
Remarques : Les valeurs de N sont obtenues par la relation N= N1 + N2. Dans la nappe, la valeur de N est corrigée lorsque N ≥ 15. N devient N . N = 0,5N + 7,5.
Le troisième refus a été obtenu à 40,43 m.

Masse mouton =

5- Lithological section and corings



6- Lithological section and pressuremetric values


Altitude (m)
Depth
Coupe lithologique
E.I. ou Cores
Pl* Pressions limites nettes (Mpa)
Module pressiométrique EM (MPa)
Em/Pl
OUTILS
TUBAGE
Pf* (MPa)
Sable fin jaune brun.
Sable fin à moyen jaune brun.
Vase très micacée à débris végétaux peu sableuse gris noir.
Sable moyen micacé gris noir à débris végétaux.
Sable fin gris.
Sable fin à moyen peu argileux.
Sables moyens gris.
Sable moyen à grossier + débris végétaux gris noir
Sable grossier gris.
Sable moyen blanchâtre.
Sable grossier+gravier blanchâtre.
Sable moyen à grossier peu argileux blanchâtre.
Argile grise compacte très peu sableuse.
Argile grise raide.
Trilame (diamètre 53mm)
Trilame (diamètre 63mm)
TUBAGE HQ
Remark : Sondage pressiométrique réalisé au droit de la culée Douala sur le 2eme pont du Wouri.

7- Data from laboratory identification tests

N°	N° Sondage carotté	N° Echantillon intact et Remanié	Profondeur (m)	ESSAIS D'IDENTIFICATION: Granulométrie (mm)			Limites d'Atterberg (%)			Teneur en eau naturelle	Teneur en matière organique	Teneur en carbonates	Densité (T/m³)			Classification	Coefficient de courbure	Coefficient d'uniformité	Diamètre efficace	Equivalent de sable ES		Essai de Bleu de méthylène	Nature géologique
				% < 2	% < 0,5	% < 0,08	W_L	W_P	IP	W (%)	M.O (%)	CaCO3 (%)	γ_h	γ_d	γ_s	LPC	Cc	Cu	D_{10} (mm)	A vue	Au Piston	VBS	
1	FP 7	ER 1	1,05 – 1,50	100	84,5	5,4	Nm	Nm	Nm	7,02	-	-	-	-	2,59	Sm	1,59	2,52	0,125	66,3	49,7	-	Sable grossier jaunâtre
2	FP 7	ER 2	2,55 – 3,00	99	73,3	3	Nm	Nm	Nm	27,8	-	-	-	-	2,6	Sm	1,04	2	0,2	90,2	84,1	-	Sable grossier jaunâtre
3	FP 7	ER 3	4,05 – 4,50	100	95,8	44,3	Nm	Nm	Nm	56,4	-	-	-	-	2,29	-	-	-	-	-	-	-	Débris végétaux peu sableux micacé
4	FP 7	ER 4	5,05 - 6,00	93,5	84,7	56	Nm	Nm	Nm	54,5	37,912	4	-	-	1,69	-	-	-	-	-	-	-	Débris végétaux peu sableux micacé (TOURBE)
5	FP 7	ER 5	7,05 - 7,50	97	78,2	10,5	Nm	Nm	Nm	76,4	-	-	-	-	2,49	Sm	0,64	4	0,1	62,1	56	-	Sable grossier noirâtre peu micacé + débris végétaux
6	FP 7	ER 6	9,05 - 9,50	98,7	72,3	7,2	Nm	Nm	Nm	29,9	-	-	-	-	2,44	Sm	0,98	2,5	0,16	41,9	41,2	-	Sable grossier jaunâtre
7	FP 7	ER 7	11,05 - 11,50	100	88,8	2,8	Nm	Nm	Nm	52,1	2,58	2	-	-	2,39	Sb	1,24	1,96	0,16	38,3	37,9	-	Sable grossier noirâtre peu micacé
8	FP 7	ER 8	11,55 - 12,00	100	82,4	13,7	Nm	Nm	Nm	38,6	-	-	-	-	2,42	SL	/	/	/		/	-	Sable limoneux noirâtre peu micacé
9	FP 7	ER 9	13,05 - 13,50	100	98,1	3,9	Nm	Nm	Nm	52,1	-	-	-	-	2,41	Sm	0,83	1,97	0,16	36,58	31,35	-	Sable grossier noirâtre peu micacé
10	FP 7	ER 11	16,05 - 16,50	98,9	81,3	5,2	Nm	Nm	Nm	31,9	-	-	-	-	2,55	Sm	0,78	2	0,2	32,5	31,5	-	Sable grossier noirâtre peu micacé
11	FP 7	ER 12	17,55 – 18,00	99,5	90,6	4,4	Nm	Nm	Nm	33,6	-	-	-	-	2,56	Sm	0,99	1,56	0,2	33,6	32,2	-	Sable grossier noirâtre micacé
12	FP 7	ER 13	19,05 - 19,50	99	55,1	2,9	Nm	Nm	Nm	28,7	-	-	-	-	2,55	Sm	0,8	2	0,25	49,8	49,3	-	Sable grossier noirâtre
13	FP 7	ER 14	20,55 - 21,00	97,7	56,5	11	Nm	Nm	Nm	14,6	-	-	-	-	2,55	SL	2,25	6,25	80µ	60,4	58,8	-	Sable limoneux noirâtre
14	FP 7	ER 15	22,05 - 22,50	95	56,9	7	Nm	Nm	Nm	39,5	-	-	-	-	2,49	Sm	1,12	3,12	0,16	26,9	26,1	-	Sable grossier noirâtre
15	FP 7	ER 16	23,55 - 24,00	85,3	32,9	3,6	Nm	Nm	Nm	26,7	0,788	4	-	-	2,12	Sm	1,4	4,5	0,2	24,2	23,5	-	Sable grossier noirâtre
16	FP 7	ER 17	25,05 - 25,55	72,1	15,6	2,1	Nm	Nm	Nm	20,1	-	-	-	-	2,58	Sm	1,28	3,12	0,4	67,7	67	-	Sable grossier noirâtre
17	FP 7	ER 18	26,50 - 27,00	56,3	27,6	8,4	Nm	Nm	Nm	30,6	-	-	-	-	2,65	Sb	1,31	23	0,1	66,6	39,2	-	Sable grossier noirâtre
18	FP 7	ER 19	28,05 - 28,50	95,3	52,2	7,2	Nm	Nm	Nm	21,4	-	-	-	-	2,62	Sm	0,82	3	0,2	34,4	33,4	-	Sable grossier grisâtre
19	FP 7	ER 20	29,55 - 30,00	91,9	55,2	28,5	37,1	21,3	15,8	17,9	-	-	-	-	2,57	SA	-	-	-	51,1	40,2	-	Sable argileux grisâtre
20	FP 7	ER 22	32,55 - 33,00	69,2	31,7	7,6	Nm	Nm	Nm	13,5	0	11	-	-	2,61	Sb	1	6,25	0,2	46,3	45,4	-	Sable grossier grisâtre
21	FP 7	ER 24	35,55 - 36,00	96,4	95	80,6	41,4	23,2	18,2	37,3	-	-	-	-	2,53	Ap	-	-	-	-	-	-	Argile limoneuse grisâtre
22	FP 7	ER 25	37,05 - 37,50	100	100	99,5	30,5	17,9	12,6	28,5	-	-	-	-	2,67	Ap	-	-	-	-	-	-	Argile limoneuse grisâtre
23	FP 7	ER 26	38,55 - 39,00	97,6	96,5	91,5	41,7	29,4	12,3	35,5	-	-	-	-	2,6	Lp	-	-	-	-	-	-	Limon argileux grisâtre
24	FP 7	ER 27	40,05 - 40,50	98,4	96,5	81,3	44,5	21,3	23,2	30,5	-	-	-	-	2,61	Ap	-	-	-	-	-	-	Argile limoneuse grisâtre
25	FP 7	EC caisse N°1 40,50 - 48,10	43,62 - 43,85	-	-	-	-	-	-	-	0,214	9	-	-	-	-	-	-	-	-	-	-	Argile compacte
26	FP 7	EC caisse N°2 48,10 - 53,76	51,97 - 52,09	-	-	-	-	-	-	-	-	8,7	-	-	-	-	-	-	-	-	-	-	Argile compacte
27	FP 7	EC caisse N°3	54,00 - 54,13	-	-	-	-	-	-	-	-	12	-	-	-	-	-	-	-	-	-	-	Argile marneuse compacte
		53,76 - 57,90	56,72 - 56,87								0	6											Argile compacte
			57,48 - 57,60								-	2											Argile compacte
28	FP 7	EC caisse N°4	57,90 - 63,80	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	Argile compacte
29	FP 7	EC caisse N°5 63,80 - 71,30	69,87 - 70,10	-	-	-	-	-	-	-	2,419	10,5	-	-	-	-	-	-	-	-	-	-	Argile marneuse compacte
30	FP 7	EC caisse N°6 71,30 - 77,30	73,67 - 73,80	-	-	-	-	-	-	-	-	10	-	-	-	-	-	-	-	-	-	-	Argile compacte
31	FP 7	EC caisse N°7 77,30 - 80,30	77,95 - 78,08	-	-	-	-	-	-	-	1,433	3,5	-	-	-	-	-	-	-	-	-	-	Argile compacte

ER: Echantillon remanié

EC: Echantillon de carotte

MATIERES ORGANIQUES: méthode de Anne, norme afnor X31 - 109

CARBONATE DE CALCUIM: méthode de Drouineau, norme afnor X31 - 106

TENEUR EN MATIÈRE ORGANIQUE

Teneur en matière organique (%)	Désignation
0 – 3	Sol inorganique
3 – 10	Vase
10 – 30	Sol tourbeux
> 30	Tourbe

TENEUR EN CARBONATE DE CALCIUM

Teneur en $CaCO_3$ (%)	Désignation géotechnique

8- Data from uniaxial compression tests

TABLEAU DES ECHANTILLONS INTACTS DE SOL DU FP7 de 40,50 à 80,30m POUR ESSAI DE COMPRESSION UNIAXIALE								
N°Echantillon	Profondeur (m)	Φ (mm)	H (mm)	Nature	Date de prélèvement	Date d'essai	Frupt (KN)	RC (Mpa)
1	51,97	63	126	Limon argileux à Argile limoneuse (peu compact à compact)	06/06/2013	22/07/2013	5,796	1,860
2	52,50	63	126	Limon argileux à Argile limoneuse (peu compact à compact)	06/06/2013	22/07/2013	5,861	1,881
3	53,10	63	126	Limon argileux à Argile limoneuse (peu compact à compact)	06/06/2013	22/07/2013	5,991	1,923
4	53,40	63	126	Limon argileux à Argile limoneuse (peu compact à compact)	06/06/2013	22/07/2013	5,1655	1,658
5	53,53	63	126	Limon argileux à Argile limoneuse (peu compact à compact)	06/06/2013	22/07/2013	5,959	1,912
6	53,87	63	126	Limon argileux à Argile limoneuse (peu compact à compact)	07/06/2013	22/07/2013	5,341	1,714
7	54,00	63	126	Limon argileux à Argile limoneuse (peu compact à compact)	07/06/2013	22/07/2013	6,511	2,090
8	54,35	63	126	Limon argileux à Argile limoneuse (peu compact à compact)	07/06/2013	22/07/2013	6,771	2,173
9	56,00	63	126	Limon argileux à Argile limoneuse (peu compact à compact)	07/06/2013	22/07/2013	33,356	10,705
10	57,00	63	126	Limon argileux à Argile limoneuse (peu compact à compact)	07/06/2013	22/07/2013	6,999	2,246
11	57,15	63	126	Limon argileux à Argile limoneuse (peu compact à compact)	07/06/2013	22/07/2013	3,879	1,245
12	57,48	63	126	Limon argileux à Argile limoneuse (peu compact à compact)	07/06/2013	22/07/2013	4,821	1,547
13	57,60	63	126	Limon argileux à Argile limoneuse (peu compact à compact)	07/06/2013	22/07/2013	4,041	1,297
14	59,60	63	126	Limon argileux à Argile limoneuse (peu compact à compact)	07/06/2013	22/07/2013	2,221	0,713
15	59,75	63	126	Limon argileux à Argile limoneuse (peu compact à compact)	07/06/2013	22/07/2013	3,001	0,963
16	66,00	63	126	Argile marneuse	08/06/2013	22/07/2013	5,666	1,818
17	66,20	63	126	Argile marneuse	08/06/2013	22/07/2013	4,561	1,464
18	66,37	63	126	Argile marneuse	08/06/2013	22/07/2013	6,089	1,954
19	73,67	63	126	Argile marneuse	10/06/2013	22/07/2013	8,949	2,872
20	74,80	63	126	Argile marneuse	10/06/2013	22/07/2013	9,956	3,195
21	76,00	63	126	Argile marneuse	10/06/2013	22/07/2013	8,656	2,778
22	78,00	63	126	Argile marneuse	10/06/2013	22/07/2013	12,491	4,009

BIBLIOGRAPHY

[1] Foundations on weak and/or compressible soils

[2] Oxford dictionary of mechanical engineering.

[3] V.J. Sharma et. al. 2015

[4] Soil mechanics and foundations / Muni BUDHU—3rd ed.

[5] Craig's soil mechanics / R.F. Craig. — 7th ed.

[6] ASTM D 2487

[7] FHWA NHI-06-088 Soils and Foundations – Volume I, December 2006

[8] Yetimoglu et al., 1994; Mitchell and Collins, 1997; Sharma et al. 2009; El Sarwaf and Nazir, 2011

[9] P.C. VARGHESE. Foundation Engineering

[10] Shallow foundations discussions and problem solving / WILEY Blackwell—p.81

[11] SETRA-LCPC. (2000). Etude et realisation des remblais sur sols compressibles.

[12] SALGADO 2008. The engineering of foundations. McGraw-Hill International Edition.

[13] Bowles, J. E. 1996. Foundation Analysis and Design, 5th Ed., New York, McGraw Hill, 1175

; Sridharan and Nagaraj, 2000

[14] Arnold VERRUIJT. Soil mechanics. Delft University of Technology, 2001, 2006.

[15] AMSOIL 2010

[16] Wikipedia 2021

[17] Douala Climate Normals 1961–1990, n.d National Oceanic and Atmospheric Administration

[18] Chiarelli, n.d.

[19] Avameg, Inc.

[20] Thesis on STRUCTURAL ANALYSIS AND DESIGN OF TALL BUILDING: COMPARISON BETWEEN STEEL AND CONCRETE by TCHABO NKWENGOUA Rejane Urielle supervised by Pr. Carmelo MAJORANA

[21] PLAXIS 2D V20 reference manual

[22] Miura et al. 2000

[23] Dadouche, 2000

[24] Geotechnics for land protection course for Master in Engineering level 5 at National Advanced School of Public Works by Dr. Lorenzo BREZZI. Course not published on line.

[25] Sanja K,2016

[26] NCHRP, 1989

[27] Krishna et. al. 2004

[28] Croce, P., Flora, A. & Modoni, G. (2014). Jet Grouting Technology, Design and Control, CRC Press, Taylor & Francis Group.

[29] Chu, J., Varaksin, S., Klotz, U. & Mengé, P. 2009. State of the Art Report: Construction Processes. 17th International Conference on Soil Mechanics & Geotechnical Engineering: TC17 meeting ground improvement, Alexandria, Egypt, 7 October 2009, 130

[30] Combarieu, O. 1988. Amélioration des sols par inclusions rigides verticales. Application à l'édification de remblais sur sols médiocres. Revue Française de Géotechnique, 44, 57-79

[31] Serge Varaksin, Babak Hamidi, Noël Huybrechts and Nicolas Denies. Ground Improvement vs. Pile Foundations? ISSMGE - ETC 3 International Symposium on Design of Piles in Europe. Leuven, Belgium, 28 & 29 April 2016

[32] Terzaghi,K. and Peck, R.B. (1948) Soil Mechanics in Engineering Practice. John Wiley