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QUALITY CONTROL AND INFLUENCE OF A SLIGHT DEFECT ON PILE FOUNDATION: CASE STUDY OF PILE IN THE INDUSTRIAL ZONE OF BONABERI-DOUALA

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

Curriculum: Geotechnical Engineering

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DEDICATION

I dedicate this thesis

to the

Taku family

who offered unconditional love, invaluable educational facilities, guidance and support throughout this journey.

Thank you so much.

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This thesis is the result of combined direct and indirect contributions of numerous individuals whose names may not all be mentioned. Their contributions are wholeheartedly appreciated and indebtedly acknowledged. Nonetheless, it is with respect and pleasure that I address my thanks to:

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GLOSSARY

LIST OF ABREVIATIONS

AASHTO	American Association of State Highway and Transportation Officials
ASCE	American Society of Civil Engineers
ASTM	American Society for Testing and Materials
CPT	Cone Penetration Test
CPTu	Cone Penetration Test – piezocone
EN	European Standard
FE	Finite Element
FEM	Finite Element Method
GTS NX	Geotechnical Analysis System new eXperience
MIDAS	Munich Image Data Analysis System
SCPT	Seismic Cone Penetration Test
SLS	Serviceability limit state
SPT	Standard Penetration Test
ULS	Ultimate Limit State
GWT	Ground water table
SSI	Soil structure interaction
LSIT	Low strain integrity test
CSL	Cross hole sonic logging

LIST OF SYMBOLS

Cc	Coefficient of consolidation
Cs	Coefficient of secondary consolidation or swelling index
Cv	Coefficient of primary consolidation
e_0	Initial void ratio
k_x	Permeability
LL	Liquid limit
PI	Plasticity index
PL	Plastic limit
γd	Dry weight
γs	Particles specific weight
γsat	Bulk density
σ'p	Pre-consolidation pressure
φ	Friction angle
C'	Cohesion
Ψ	Dilatancy angle
Cu	Undrain shear strength
ν	Poisson's ratio
k_0	Coefficient of at rest pressure
E	Young's modulus
fck	Characteristic compressive strength
fyk	Characteristic tensile strength
fyd	Design tensile strength
G	Shear modulus
ω_c	Natural water content

ABSTRACT

The main objectives of this study were to carry out a quality control of a pile foundation and assess the influence of a slight defect on pile foundation. This study was done for a pile subjected to an axial load present in a construction project of an oil and cosmetics processing unit in the Magzi industrial zone in the city of Douala. The analysis was conducted both analytically and with the 3D FEM software MIDAS GTS NX. In order to achieve the goal, a literature review was done on soil and pile foundation. Then a site recognition through documentary research, as well as the site visit in order to have information related to the project. This was then followed by a collection of data necessary for the study, a presentation of the design methods for the pile, the pile installation and testing processes and the finite element simulation procedures. The data obtained was used to make a geotechnical and structural design of the pile which was found to be good according to the norm, the testing (LSIT) results presented indicates all the piles on site were of good quality. The numerical analysis made indicates the pile was verified with a remark to consider pile-soil interaction during design. The parametric analysis made indicated the variation of the friction angle and cohesion with the model had a negligible variation of the pile behaviour where as a variation of the elastic modulus had a significant variation of the pile behaviour. The study of the influence of the defect did not compromise the capacity of the pile, with the influence of a slight defect closest to the pile head being most critical than those far away from pile head and concerning the type of defect in a particular location, a void around the shaft is most critical than a soil inclusion or void on one corner of the shaft though of slightly larger volume. The survey results for pile testing techniques in Cameroon indicates the major techniques used here in Cameroon are the LSIT and the CSL, which are the 2 preferred due to their availability, cost, prescription by clients, and local experience

Key words: Quality control, pile foundation, defect.

RESUME

Les objectifs principaux de cette étude étaient d'effectuer un contrôle de qualité d'une fondation de pieu et d'évaluer l'influence d'un léger défaut sur la fondation du pieu. Cette étude a été faite pour un pieu soumis à une charge axiale présente dans un projet de construction d'une unité de traitement des huiles et cosmétiques dans la zone industrielle de Magzi dans la ville de Douala. L'analyse a été menée à la fois de manière analytique et avec le logiciel FEM 3D MIDAS GTS NX. Afin d'atteindre l'objectif, une revue de la littérature a été faite sur le sol et les fondations sur pieux. Puis une reconnaissance du site à travers une recherche documentaire, ainsi que la visite du site afin d'avoir des informations liées au projet. Deplus, une collecte des données nécessaires à l'étude, une présentation des méthodes de conception du pieu, des processus d'installation et d'essai du pieu et des procédures de simulation par éléments finis. Les données obtenues ont été utilisées pour faire une conception géotechnique et structurelle du pieu qui s'est avérée bonne selon la norme, les résultats des tests (LSIT) présentés indiquent que tous les pieux sur le site étaient de bonne qualité. L'analyse numérique effectuée indique que le pieu a été vérifié avec une remarque pour considérer l'interaction pieu-sol pendant la conception. L'analyse paramétrique a indiqué que la variation de l'angle de frottement et de la cohésion avec le modèle a eu une variation négligeable du comportement du pieu alors que la variation du module élastique a eu une variation significative du comportement du pieu. L'étude de l'influence du défaut n'a pas compromis la capacité du pieu, l'influence d'un léger défaut le plus proche de la tête du pieu étant plus critique que ceux éloignés de la tête du pieu et concernant le type de défaut dans un emplacement particulier, un vide autour du pieu est plus critique qu'une inclusion de sol ou vide sur un coin du pieu bien que de volume légèrement plus grand. Les résultats de l'enquête sur les techniques d'essais de pieux au Cameroun indiquent que les principales techniques utilisées ici au Cameroun sont le LSIT et le CSL, qui sont les 2 préférés en raison de leur disponibilité, leur coût, leur prescription par les clients et l'expérience locale.

Mots clés: Contrôle de qualité, fondation sur pieux, défaut.

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GENERAL INTRODUCTION

Foundations are a critical element of any structure, because if not of good performance can lead to the structure collapse or the occurrence of large settlements. For piles the direct inspection of in-place elements is impossible, but in recent decades a growing list of test methods has been developed to indirectly evaluate the structural integrity and load bearing capacity of piles.

A process whereby confidence in the quality of the installed pile is expeditiously attained is essential to a contractor to confirm the adequacy of the deployed construction methods and vital to the engineer to verify the competence of the foundation installed.

To this effect, the main aims of this work are to perform a quality control of a pile foundation via the geotechnical capacity and structural integrity evaluation and assess the influence of a slight defect on pile foundation through FE analysis. To achieve these objectives, the work has been divided into 4 chapters. The first two chapters deals with the literature review on soils which has an important role in foundation design and pile foundations. The third chapter entitled methodology presents the steps adopted to achieve the goals of this work and finally chapter four which focuses on the presentation of the results of the adopted methodology both analytical and numerical with their interpretations.

CHAPTER 1. DEEP FOUNDATION

Introduction

Soil mechanics is the study of the response of soils to loads. These loads may come from human-made structures (e.g., buildings), gravity (earth pressures), and natural phenomena (e.g., earthquake). A good understanding of soil is necessary for us to analyse and design support systems (foundations) for infrastructures (e.g., roads and highways, pipelines, bridges, tunnels, embankments, high-rise buildings), energy systems (e.g., hydroelectric power stations, wind turbines, solar supports, geothermal and nuclear plants) and environmental systems (e.g., solid waste disposal, reservoirs, water treatment and water distribution systems, flood protection systems).Deep foundations are those with depth far greater than the breadth (D >> B). This chapter presents some bases of soil and pile foundation which is a widely applied deep foundation type for civil structures.

1.1. Soil

To the civil engineer, soil is any uncemented or weakly cemented accumulation of mineral particles formed by the weathering of rocks, the void space between the particles containing water and /or air (Knappett & Craig, 2012). To study soil behaviour, we have to couple concepts in solid mechanics (e.g., statics) and fluid mechanics. However, these mechanics are insufficient to obtain a complete understanding of soil behaviour because of the uncertainties of the applied loads, natural distribution of different soil types. We have to utilize these mechanics with simplifying assumptions and call on experience to make decisions (judgment) on soil behaviour. Soil plays a vital role in civil engineering in particular and to mankind in general. Soil therefore needs to be studied with emphasis on points such as its formation, types and characteristics, description of soils, soil types, methods of soil classification and some fundamental physical properties of soils.

1.1.1. Constituents of a soil

Soil is a porous medium with three phases: the solid phase, the liquid phase and the gaseous phase. The solid phase is made up of mineral and organic particles, and the gaseous and liquid phases are made up of air and water that occupy the voids between the solid particles (V. Robitaille & D. Tremblay, 1997).

1.1.1.1. Solid fraction

The solid phase (more than 95% mineral fraction) occupies 40% (highly fragmented soil) to 70% (highly compacted soil) of the volume of the soil, the rest corresponding to the fluid phase (liquid and gas). It is composed of a mineral fraction and an organic fraction.

1.1.1.2. Liquid fraction

The liquid fraction consists of water and dissolved elements. Water is in three states in the soil: gravity water (also called saturation water, percolation water or free water) which circulates in the macroporosity, the capillary water (also called pendular water) which occupies the mesoporosity and the pellicular water (also called bound water) which occupies the microporosity. The capillary water and the film water form the retention water (water retained in all soil horizons).

1.1.1.3. Gas fraction

The air in the soil is made up of free and dissolved gases that occupy the pores left behind by the water during its withdrawal. Its composition is always quite similar to that of the atmospheric air with which it is in permanent contact. Variations in the composition of the air in the soil, due to the biochemical transformation processes occurring in the soil, tend to be compensated for by exchanges with the atmosphere. However, there are differences, a higher concentration of carbon dioxide and a lower concentration of oxygen. These differences are mainly attributable to microbial respiration (to a lesser extent to the respiration of roots and aerobic flora), which results in the consumption of oxygen, taken from the soil, which is accompanied by the production of CO_2 gas.

1.1.2. Soil formation

The study of soil formation begins with the factors of soil formation before moving on to the soil formation process (R. L'Herminier, 1967).

1.1.2.1. Formation factors

Soil is continually changing under the influence of physical, chemical, biological and human processes. It evolves in time and space. Many factors interfere in the formation of soil, which explains the great diversity of soil types encountered. The most important factors are the nature of the parent rock or source material, the climate, weather and vegetation. Other factors such as the relief, the topology and the intervention of man are also important.

a. The mother rock

Of various geological origins (magmatic, metamorphic or sedimentary), the initial rock, known as the mother rock, is a hard or loose rock that was formed thousands of years ago. A rock is made up of an assembly of several identical or different minerals. As an example, granite is a magmatic rock formed by the assembly of minerals of quartz, micas and feldspars; limestone is a sedimentary rock that contains a high proportion of a mineral called calcite. These main minerals are called primary minerals. Rocks also contain small amounts of other minerals called accessory minerals. The surface formations are either ancient alluvial deposits terraced along the rivers, or colluvium that has slide down the slopes, or silt deposited by the winds, or glacial deposits can also constitute the original material and according to their nature and age of deposition influence the morphological characteristics of the soil. In the course of time, homogeneous layers are formed, superimposed, and parallel to the surface, called horizons. Depending on the number of horizons, soils are more or less diversified.

b. Climate

Climate plays an important role in the formation and characteristics of soils. It has an impact on vegetation. Temperature affects the rate of weathering of rocks and the rate of decomposition of organic matter. In a humid climate, rain dissolves mineral elements, causes chemical reactions, washes away, depletes the upper part of the soil of and carries away the fine particles in the lower layers. It has been observed that ancient climates (ice ages, etc.) have had an important influence on the characteristics of today's soils.

c. The relief

Knowledge of the relief factor is also important from various points of view. Indeed, it cannot be considered as an independent factor, comparable to the previous ones, because it depends on most of the other factors of soil formation. It is at the same time a particular manifestation of variations in rock, climate and time of evolution, and a cause of the soil's own evolution. On small scales, relief is mainly related to tectonic phenomena and to the distribution of geological domains. But the shape of the terrain also depends on the parent rock. In the same climate, surfaces of the same age have different topographies depending on whether they are granitic outcrops (domes, flat valleys) or hard limestone (karst with dolomites).

d. The weather

Soils form slowly. The time scale for the formation of a soil, for it to reach (complete development of the horizons) is measured in thousands of years. The time it takes for a soil to form depends on the climate. The time it takes for a soil to reach maturity can be ten thousand years in cold areas to one hundred years in tropical areas. But soils can be quickly degraded, and are only very slowly renewable.

e. The vegetation

The vegetation is different according to the geographical places (mountains, valleys ...), climates, and the rocks of the subsoil. Plant debris spreads out on the ground, forming litter (layers of fresh organic matter) which decompose over time more or less rapidly depending on the climatic decompose more or less rapidly depending on the climatic zone and are transformed into humus by the action of organisms and micro-organisms living in the soil (bacteria, fungi, mites, earthworms, slugs, snails, beetles, ants, larvae, moles, field mice, etc.). Organic matter are rich in carbon (C), hydrogen (H) and oxygen (O). Humus and clays from the minerals of the original rock will associate and form the different aggregates of elementary particles that make up the fine earth of the soil. The soil is exploited by the roots of plants (root system) which draw water and nutrients necessary for their development. In exchange, they give back carbon-rich substances that feed certain microbes, microbes, they aerate and form conduits that allow the passage of water and gases. They play an important role in the life of the soil.

f. Human activity

Human activities in a region have a great influence on the soil formation process. Indeed, all the factors we have listed above can be modified by human activity because everything that man does has a significant impact on the nature around him. Pollution, for example, will have consequences such as global warming which will affect the climate and vegetation. From agricultural activities will have a significant impact on the type of soil encountered.

1.1.2.2. Formation process

Under the action of the factors of the environment, a series of transformations takes place which that lead to the formation of the soil. Three examples of phenomena contribute to this evolution the alteration of rocks, the decomposition of organic matter, the transfer and organisation of the materials formed. According to Landry & al (1992), these phenomena involve mechanisms, very dependent on each other. They are not successive, but simultaneous; their effects add up and it is often difficult to dissociate them.

a. Alteration of rocks

Two mechanisms of rock transformation come into play, a physical process (fragmentation, disintegration) which separates the primary minerals from the rocks (for example quartz, feldspar, micas which are the primary minerals of granite) and produces fragments the same chemical composition as the original rock and a chemical process (mainly hydrolysis and neoformation) that transforms the primary minerals such as quartz, feldspars, micas... into secondary minerals (clays, oxyhydroxides,...). Organic matter also plays an important role in the process of transformation and solubilisation (chemical alteration).

b. Decomposition of organic matter

The soil contains a large quantity of living organisms (bacteria, fungi, mites, earthworms, slugs, snails, beetles, ants, larvae, moles, field mice ...) which decompose the fresh organic matter, release its compounds which will undergo a process of process of mineralization or humification. Mineralization consists in the release of soluble mineral compounds such as sulphates, phosphates, ammonium, nitrates or gaseous compounds such as carbon dioxide. Another part of the compounds gives new complex molecules, of colloidal nature which will constitute humus (humification process).

c. Transport of the formed materials

Mineral and organic decomposition leads to the formation of more or less soluble compounds that can be soluble compounds which will be able to move. In the alteration mantle, the movements of water movements, linked to the laws of gravity and evaporation, carry these elements away and firstly cause a loss of matter. The concentration of the elements in solution varies according to the nature of the water. If continental waters are little mineralised, generally of the order of 100mg/L, sea waters reach very important concentrations which can exceptionally exceed 200g/L. The dissolved ions are also different $Ca2^{+}$ and HCO_3^{-} being preponderant in the case of freshwater, while Cl^{-} and Na^{+} largely dominate in seawater. The transport of solid elements depends on two types of parameters which are the specific parameters of the elements themselves, i.e. their size, their shape, density, surface properties, etc., and the parameters dependent on the transport agent, in particular its nature (water, wind, ice, its speed, its force, etc.). Transport is accompanied by shaping and sorting of the elements.

1.1.3. Soil characteristics

The characteristics of the soil are mainly physical, mechanical and chemical. To these we can add the electrical properties of the soil (V. Robitaille & D.

Tremblay, 1997). In this part the interest is based on the engineering and geotechnical characteristics.

1.1.3.1. Engineering characteristics

The major engineering characteristics of the main soil groups as related to foundation design are summarized as follows. A discussion on the practical aspects of the engineering characteristics is presented for granular and fine-grained soils following these summaries.

a. Engineering characteristics of coarse-grained soils (sands and gravels)

Generally very good foundation material for supporting structures and roads, very good embankment material, the best backfill material for retaining walls, might settle under vibratory loads or blasts, dewatering may be difficult in open-graded gravels due to high permeability, generally not frost susceptible, etc. (Naresh C.et al, 2006)

b. Engineering characteristics of fine-grained soils (inorganic clays)

Generally possess low shear strength, plastic and compressible, can lose part of shear strength upon wetting, can lose part of shear strength upon disturbance, can shrink upon drying and expand upon wetting, very poor material for backfill, poor material for embankments, can be practically impervious, clay slopes are prone to landslides, etc. (Naresh C.et al, 2006)

c. Engineering characteristics of organic soils

The term organic designates those soils, other than topsoil, that contain an appreciable amount of vegetative matter and occasionally animal organisms in various states of decomposition. Any soil containing a sufficient amount of organic matter to influence its engineering properties is called organic soil. The organic matter is objectionable as it reduces load carrying capacity of the soil, increases compressibility considerably, frequently contains toxic gases that are released during the excavation process. Generally organic soils, whether peat, organic clays, organic silts, or even organic sands, are not used as construction materials. (Naresh C.et al, 2006)

1.1.3.2. Geotechnical characteristics

Basically in soil mechanics, properties of soil are broadly classified as physical (such as index) properties which give us a general indication of the type and state of the soil we are dealing with and engineering (mechanical) properties which are properties we use in our calculations. In this work the index properties, consolidation, settlement and shear strengths of soil will be considered.

a. Index properties

The properties of soil which are used for identification and classification are termed index properties. These properties could include grain size distribution, water content, consistency limits, specific gravity, relative density and others.

i. Grain size distribution

The grain size distribution permits us to classify the soil and to determine gradation. This distribution of a soil can be determined from sieve analysis (screening a known weight of the soil through a stack of sieves of progressively finer mesh size) and/or hydrometer tests (mixing a small amount of soil into a suspension and observing how the suspension settles in time.

Larger particles will settle quickly, followed by smaller particles. When the hydrometer is lowered into the suspension, it will sink until the buoyancy force is sufficient to balance the weight of the hydrometer) according to ASTM standard test method D 422. We may assume that the smallest size that we can see by the naked eye as single grain is about 0.06 mm (0.02–0.06 mm is silt). Particles below this size are called fines. Hence we should first examine the soil to find out the percentage of fines with reference to the whole sample. We should also be aware that it is the finest 20 to 25 percent part of the soil that will have the greatest influence on the mechanical behaviour of the soil (P.C Varghese, 2012).The grain size distribution may be conveniently shown on a diagram as in fig1.3.

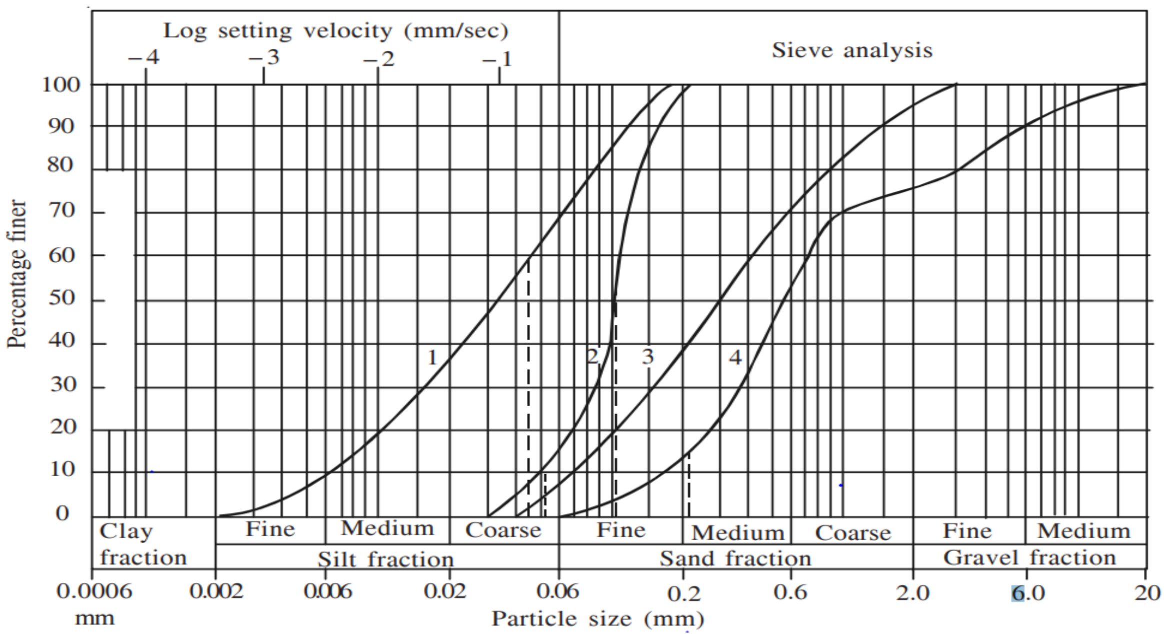


Figure 1. 1 Grain size distribution chart and classification of soils: 1. well graded sandy silt; 2. Uniform fine sand; 3. well graded sand; 4. poorly graded gravelly sand (P.C Varghese, 2012)

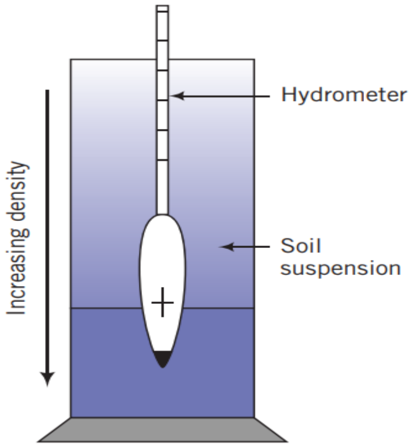


Figure 1. 2 Hydrometer in soil-water suspension (Budhu, 2015)

ii. Water content

Water content (w) by Budhu (2015) is the ratio, often expressed as a percentage, of weight of water to the weight of solids.

$$w = \frac{W_W}{W_S} X100\% \qquad 1.1$$

The water content can be determined using an oven dry method or a sand bath method (a rapid and approximate method) or the pycnometer method or a rapid moisture meter (also called calcium carbide method).According to Budhu (2015) the water content of a soil is found by weighing a sample of the soil and then placing it in an oven at 110 ± 5°C until the weight of the sample remains constant; that is, all the absorbed water is driven out. For most soils, a constant weight is achieved in about 24 hours. The soil is removed from the oven, cooled, and then weighed. The detailed procedure to determine the water content of soils is described in ASTM D 2216.

Figure 1. 3 Rapid moisture meter (Source: C.M Jadar, 2018)

iii. Consistency limits

Soil consistency or simply consistency is analogous to viscosity in liquids and indicates internal resistance to forces that tend to deform the soil (Budhu, 2015) or consistency is a term used to indicate the degree of firmness of cohesive soils (V.N.S Murthy, 2007). Consistency is expressed qualitatively by such terms as stiff, hard, firm, plastic, soft, and very soft. The consistency of a soil can be expressed in terms of atterberg limits (liquid and plastic limits) and unconfined compressive strengths. Consistency changes with water content. A measure of consistency is provided by the consistency index defined as

$$CI = \frac{LL - w}{LL - PL} = \frac{LL - w}{PI} \qquad 1.2$$

Where:

LL = liquid limit w = water content PL = plastic limit PI = plasticity index

The physical and mechanical behaviour of fine-grained soils is linked to four distinct states: solid, semisolid, plastic, and liquid, in order of increasing water content. Considering a soil initially in a liquid state that is allowed to dry uniformly, a plot of volume versus water content as shown in **Figure 1.4**, with the liquid limit considered as the water content at which the soil behaves as a semi-solid and the plastic limit as the point where it behaves as a semi-solid and the shrinkage limit where it behaves as a solid. The range of water contents over which the soil deforms plastically is known as the plasticity index (PI).

$$PI = LL - PL \qquad 1.3$$

If the in-situ water content is below but near about the liquid limit, then a normally consolidated soft clay is expected. On the other hand if the water content is very low, near the plastic limit, it will generally be an over consolidated clay (P.C Varghese, 2012). The liquid and plastic limits are determined by means of arbitrary test procedures (like the Casagrande's apparatus for liquid limit). In the UK, these are fully detailed in BS 1377, part 2 (1990)

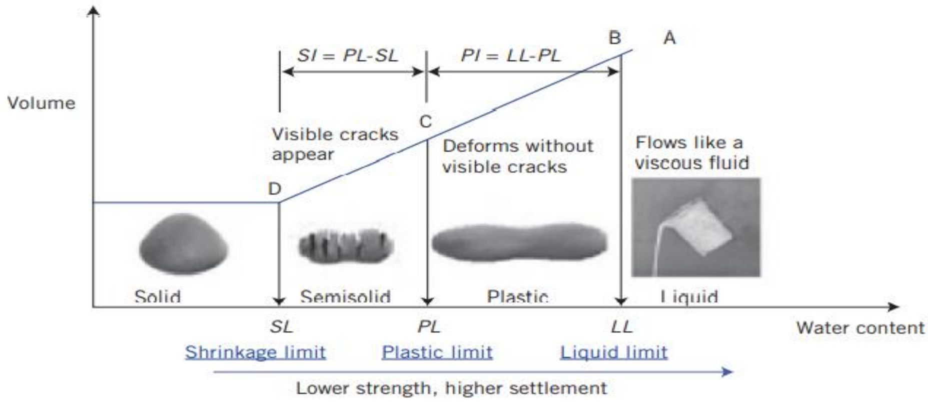


Figure 1. 4 Changes in soil states as a function of soil volume and water content (Budhu, 2011)

iv. Specific gravity

Specific gravity (Gs) is the ratio of the weight of the soil solids to the weight of water of equal volume (Budhu,2015) or by C.M Jadar (2018) specific gravity is the ratio of the weight of the soil solids to the weight of equal volume of water content at 4° standard temperature.

$$G_s = \frac{W_s}{V_s \gamma_w} \qquad 1.4$$

Where γw = 9.81 kN/m3 is the unit weight of water. The specific gravity of soils ranges from approximately 2.6 to 2.8. For most problems, Gs can be assumed, with little error, to be equal to 2.7 (Budhu, 2011). Two types of container are used to determine the specific gravity. One is a volumetric flask (at least 100mL) that is used for coarse-grained soils. The other is a 50-mL density bottle (stoppered bottle) that is used for fine-grained soils. The procedure to determine the specific gravity of soils is described in ASTM D 854 for soil particles less than 2.75mm

(No. 4 sieve).

v. Relative density

Relative density (Dr) is an index that indicates the degree of packing between the loosest and densest possible state of coarse-grained soils as determined by experiments (Budhu, 2015). It could be used to express the relationship between the in-situ void ratio (e), or the void ratio of a sample, and the limiting values e_{max} and e_{min} representing the loosest and densest possible soil parking states respectively.

$$D_r = \frac{e_{max} - e}{e_{max} - e_{min}} \qquad 1.5$$

where

e_{max} is the maximum void ratio (loosest condition), e_{min} is the minimum void ratio (densest condition), and e is the current void ratio.

In terms of dry unit weight of soil, the relative density can also be written as

$$D_r = \frac{\gamma_d - (\gamma_d)_{min}}{(\gamma_d)_{max} - (\gamma_d)_{min}} \left\{ \frac{(\gamma_d)_{max}}{\gamma_d} \right\} \qquad 1.6$$

The procedures for the determination of maximum and minimum void ratios for coarse-grained soils are outlined in ASTM D 4253 and ASTM D 4254.

b. Shear strength

The strength of geologic material is a variable property that is dependent on many factors, including material properties, magnitude and direction of applied forces and their rate of application, drainage conditions of the mass, and the magnitude of confining pressure. Unlike steel whose strength is usually discussed in terms of either tension or compression and concrete whose strength is generally discussed in terms of compressive strength only, the strength of soil is generally discussed in shear strength. Typical geotechnical failures occur when the shear stresses induced by applied loads exceed the soil´s shear strength somewhere within the soil mass (FHWA, 2006).

The shear strength of a soil is the maximum internal resistance to applied shearing forces (Budhu, 2015). A soil mass will distort when shear forces are applied to it. A measure of this distortion is the shear strain. Considering engineering (mechanical) properties strength is the most important property. Coulomb in 1773 gave a law for shear strength of soil.

$$\tau = c + \sigma \tan \emptyset \qquad 1.7$$

Where:

τ = shear strength

c = cohesion

σ = total compressive stress

$\emptyset$ = angle of internal friction (apparent friction)

In 1920 Terzaghi pointed out the limitations of Coulomb's law and the importance of pore pressure in mobilizing friction in soils. He modified the above equation to

$$\tau = c' + (\sigma - u) \tan \emptyset' = c' + \sigma' \tan \emptyset' \qquad 1.8$$

Where:

c' = true cohesion

u = pore water pressure

σ' = effective pressure

$\emptyset'$ = true friction

This theory has been used since 1920 and is referred to as the Classical Theory of strength of soils. For granular soils with c = 0,

$$\tau = \sigma' \tan \emptyset' \qquad 1.9$$

The shear strength of the soil is generally determined by laboratory tests (Unconfined compression test, direct shear test (granular soils) or triaxial test).

1.1.4. Soil typology

Common descriptive terms such as gravels, sands, silts, and clays are used to identify specific textures in soils. These soil textures will refer as soil types; that is, sand is one soil type, clay is another. Texture refers to the appearance or feel of a soil. Sands and gravels are grouped together as coarse-grained soils. Clays and silts are fine-grained soils. Coarse-grained soils feel gritty and hard. Fine-grained soils feel smooth. The coarseness of soils is determined from knowing the distribution of particle sizes, which is the primary means of classifying coarse-grained soils. To characterize fine-grained soils, we need further information on the types of minerals present and their contents. The response of fine-grained soils to loads, known as the mechanical behaviour, depends on the type of predominant minerals present.

Currently, many soil descriptions and soil types are in usage. A few of these are listed below.

- Alluvial soils are fine sediments that have been eroded from rock and transported by water, and have settled on river- and streambeds.
- Calcareous soil contains calcium carbonate and effervesces when treated with hydrochloric acid.
- Caliche consists of gravel, sand, and clay cemented together by calcium carbonate.
- Collovial soils (collovium) are soils found at the base of mountains that have been eroded by the combination of water and gravity.
- Eolian soils are sand-sized particles deposited by wind.
- Expansive soils are clays that undergo large volume changes from cycles of wetting and drying.

- Glacial soils are mixed soils consisting of rock debris, sand, silt, clays, and boulders.
- Glacial till is a soil that consists mainly of coarse particles.
- Gypsum is calcium sulfate formed under heat and pressure from sediments in ocean brine.
- Lacustrine soils are mostly silts and clays deposited in glacial lake waters.
- Lateritic soils are residual soils that are cemented with iron oxides and are found in tropical regions.
- Loam is a mixture of sand, silt, and clay that may contain organic material.
- Loess is a wind-blown, uniform, fine-grained soil.
- Marine soils are sand, silts, and clays deposited in salt or brackish water.
- Marl (marlstone) is a mud (see definition of mud below) cemented by calcium carbonate or lime.
- Mud is clay and silt mixed with water into a viscous fluid

1.1.5. Soil classification

A soil classification represents a system a language of communication among engineers. It provides a systematic method of categorizing soils according to their probable engineering behaviour. There are a number of systems and methods used to classify soils. The most common of these systems are the American Association of State Highway and Transportation Officials (AASHTO) soil classification system, the Unified Soil Classification System (USCS), and the United States Department of Agriculture (USDA) soil classification system. The common feature of these systems is the use of particle size distribution to differentiate the various groupings of each particular system. However, both the AASHTO and the Unified systems also use plasticity to further define classification. As a result, each system has a distinct method and nomenclature to identify and classify soils.

1.1.5.1. AASHTO classification system

The AASHTO system was developed specifically for highway construction and is still widely used for that purpose. With practice and experience, a reasonably accurate field classification can be determined.

Table 1. 1 AASHTO soil classification system (Wisconsin Department of Transportation,Caduto, 1998)

General Classification	Granular Materials (35% or less passing the 0.075 mm sieve)							Silt-Clay Materials (>35% passing the 0.075 mm sieve)			
Group Classification	A-1		A-3	A-2				A-4	A-5	A-6	A-7
	A-1-a	A-1-b		A-2-4	A-2-5	A-2-6	A-2-7				A-7-5 A-7-6
Sieve Analysis, % passing											
2.00 mm (No. 10)	50 max	...	...	...	...	...	...	...	...	...	...
0.425 (No. 40)	30 max	50 max	51 min	...	...	...	...	...	...	...	...
0.075 (No. 200)	15 max	25 max	10 max	35 max	35 max	35 max	35 max	36 min	36 min	36 min	36 min
Characteristics of fraction passing 0.425 mm (No. 40)											
Liquid Limit	...		...	40 max	41 min	40 max	41 min	40 max	41 min	40 max	41 min
Plasticity Index	6 max		N.P.	10 max	10 max	11 min	11 min	10 max	10 max	11 min	11 min
Usual types of significant constituent materials	stone fragments, gravel and sand		fine sand	silty or clayey gravel and sand				silty soils		clayey soils	
General rating as a subgrade	excellent to good							fair to poor			

Note: Plasticity index of A-7-5 subgroup is equal to or less than the LL - 30. Plasticity index of A-7-6 subgroup is greater than LL - 30

However, it is necessary to run sieve analyses and plasticity determinations to precisely classify a soil with this method. Table 1.1 presents the basic AASHTO soil classification system.

According to the AASHTO classification system, there are two general soil groups: Coarse-grained or granular and fine-grained or cohesive soils. The distinction between coarse and fine grained soil is 35% passing the 0.075mm sieve. The system also contains eight classes to identify soils and granular materials of which the classes range from A-1 to A-3 are coarse-grained material, A-4 to A-7 being fine-grained materials, and lastly A-8 which are organic soils.

1.1.5.2. Unified soil classification system

The unified soil classification system is based on the engineering properties of soil and it is most appropriate for earthwork construction. The USCS has been through several transitions since it was developed. Upon recognizing a USCS symbol of a classification group, one can immediately deduce the approximate permeability, shear strength, and volume change potential of soil and how it may be affected by water, frost, and other physical conditions.

Table 1. 2 Unified soil classification system (Wisconsin Department of Transportation)

Major Divisions			Group Symbol	Typical Names
Course-Grained Soils More than 50% retained on the 0.075 mm (No. 200) sieve	**Gravels** 50% or more of course fraction retained on the 4.75 mm (No. 4) sieve	Clean Gravels	GW	Well-graded gravels and gravel-sand mixtures, little or no fines
			GP	Poorly graded gravels and gravel-sand mixtures, little or no fines
		Gravels with Fines	GM	Silty gravels, gravel-sand-silt mixtures
			GC	Clayey gravels, gravel-sand-clay mixtures
	Sands 50% or more of course fraction passes the 4.75 (No. 4) sieve	Clean Sands	SW	Well-graded sands and gravelly sands, little or no fines
			SP	Poorly graded sands and gravelly sands, little or no fines
		Sands with Fines	SM	Silty sands, sand-silt mixtures
			SC	Clayey sands, sand-clay mixtures
Fine-Grained Soils More than 50% passes the 0.075 mm (No. 200) sieve	**Silts and Clays** Liquid Limit 50% or less		ML	Inorganic silts, very fine sands, rock four, silty or clayey fine sands
			CL	Inorganic clays of low to medium plasticity, gravelly/sandy/silty/lean clays
			OL	Organic silts and organic silty clays of low plasticity
	Silts and Clays Liquid Limit greater than 50%		MH	Inorganic silts, micaceous or diatomaceous fine sands or silts, elastic silts
			CH	Inorganic clays or high plasticity, fat clays
			OH	Organic clays of medium to high plasticity
Highly Organic Soils			PT	Peat, muck, and other highly organic soils

Prefix: G = Gravel, S = Sand, M = Silt, C = Clay, O = Organic
Suffix: W = Well Graded, P = Poorly Graded, M = Silty, L = Clay, LL < 50%, H = Clay, LL > 50%

It can also be used in estimating excavation and compaction characteristics, potential dewatering situations, and workability.

1.1.5.3. USDA Classification system

The USDA system was developed for agricultural purposes. It has some engineering applications in that it provides a relatively easy method for general field classification of soils.

However, “loamy”, while descriptive, is not an engineering term and should be avoided when discussing the engineering properties of a soil.

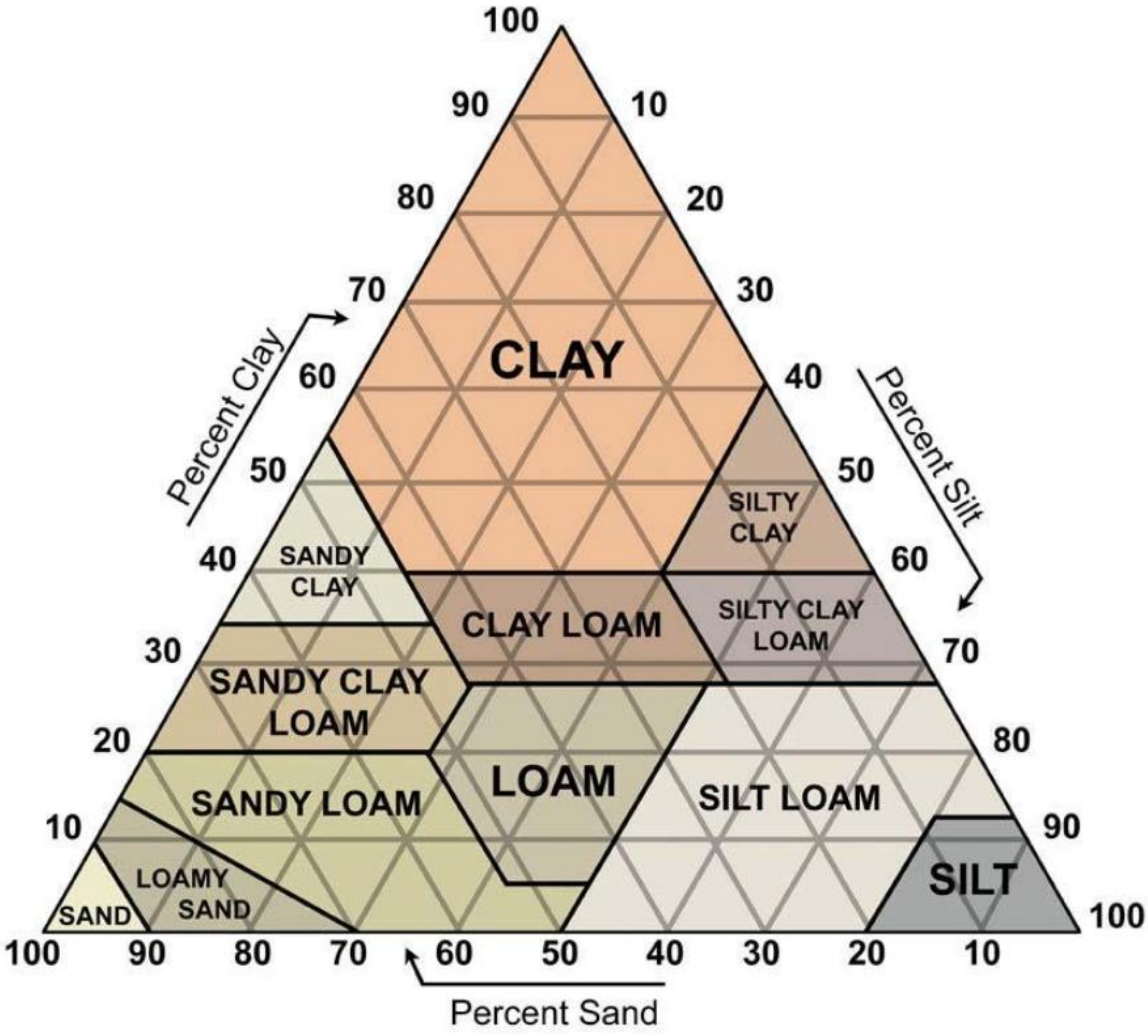


Figure 1. 5 USDA Soil Classification System (Wisconsin Department of Transportation)

1.1.5.4. Classification of soil type based on particle size

The grading curve is used for textural classification of soils. Various classification systems have evolved over the years to describe soils based on their particle size distribution. Each system was developed for a specific engineering purpose. In Figure 1.3, four systems are compared. These are the Unified Soil Classification System (USCS), the American Society for Testing and Materials (ASTM) system (a modification of the USCS system), the American Association of State Highway and Transportation Officials (AASHTO), and the British Standards (BS). Soils shall be classified into soil groups on the basis of their nature which is the composition only, irrespective of their water content or compactness, taking into account the particle size distribution, plasticity, organic content and genesis (Cola S. , 2016).

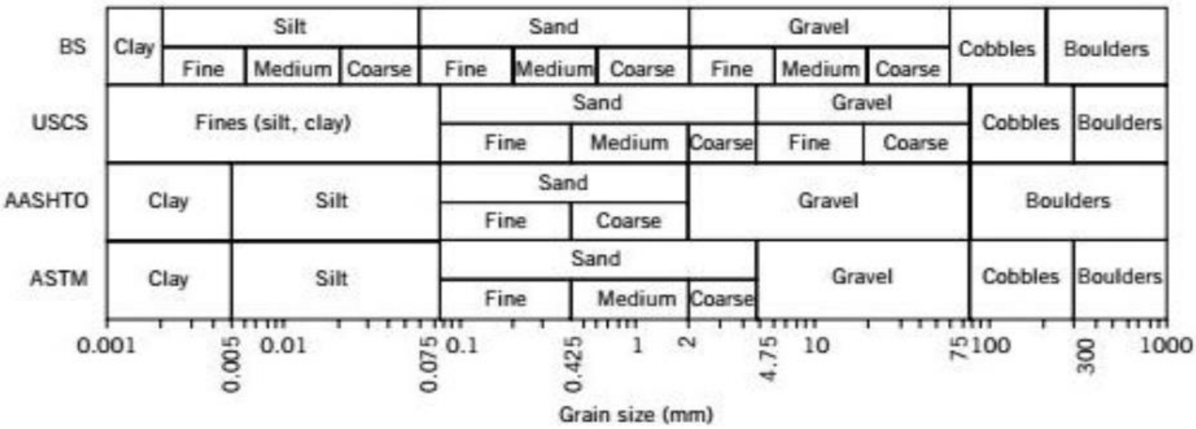


Figure 1. 6 Comparison of 4 soil classification systems based on particle size (Budhu, 2015).)

1.1.6. Uses of soil

A study of soils cannot be carried out without giving details of their use. We will therefore revisit the fields of construction and equipment, agriculture, the preservation of biodiversity and other areas of land use (www.wikipedia.fr, accessed on 28/02/2021).

1.1.6.1. In the construction sector

For construction, soil components such as gravel, clay, sand, etc. are widely used for building houses, roads, tiles, etc. The soil is the main foundation on which structures (buildings, roads, bridges, etc.) rest; it is used as a foundation and base layer in road, rail and airport design. Soil is used as the main material in the construction of dykes and earthworks, but also of supporting structures such as embankments and slopes and gravity walls and bricks.

1.1.6.2. In the field of equipment

Soil is used to make utensils, statues, etc. Utensils such as cups, plates, porcelain, plaster of Paris, etc.

Tiles used in houses or buildings are made of earthenware-like components similar to ceramics.

1.1.6.3. In the agricultural field

Soil promotes the growth of plants. In turn, these plants produce vital needs for humans such as food, clothing, furniture, medicines, etc. Even other animals, such as insects, get their food by grazing on plants.

1.1.6.4. In the preservation of biodiversity

The soil is a support for soil microbes; it allows the survival of many bacteria, algae, fungi, etc. Soil microbiology contributes to the environmental balance, such as moisture retention, decomposition of animal and plant remains, etc. These bacteria, fungi and other microbes present help to remove waste and other toxic chemicals, including plastic. Topsoil provides shelter for insects, reptiles, birds and animals. In addition, topsoil is needed for other activities such as nesting, breeding, egg hatching. Soil is the natural home of many living things. Animals such as mice, guinea pigs, mongooses, squirrels, etc. live in burrows in the ground. Reptiles such as snakes and lizards live in the soil.

Soil is also useful as a source of minerals, source of medicines, regulation of atmospheric temperature.

1.2. Pile foundation

Piles and pile foundations have been in use since prehistoric times. If the soil near the surface is incapable of adequately supporting the structural loads, piles, or other forms of deep foundations such as piers or caissons, are used to transmit the applied loads to suitable soil (or rock) at greater depth where the effective stresses are larger. A pile is a slender, structural member installed in the ground to transfer the structural loads through weak compressible strata or through water onto stiffer or more compact and less-compressible soils or onto rocks at some significant depth below the base of the structure (J.E. Bowles, 1988). In this part, interest would be based on some functions or applications of pile foundations, different pile foundation types, factors governing the choice of a pile type.

1.2.1. Functions or applications of pile foundations

When designing foundations, there are often situations where the use of shallow foundations is uneconomic or impractical (Knappett & Craig, 2012). In this case deep foundation (Piles) is an alternative foundation and are used for one or more of these purposes:

- To carry loads which are too heavy to be supported by a shallow foundation. The loads are to be transferred to deeper, stronger and less compressible strata or over a larger depth of the foundation soil as in foundations of tall buildings.
- To resist uplift, or overturning, forces as for basement mats below the water table or to support tower legs subjected to overturning
- To stiffen the soil beneath machine foundations to control both amplitudes of vibration and the natural frequency of the system.
- To carry horizontal loads as in bridge abutments or retaining walls and also to increase the stability of tall buildings. Inclined piles are also used to carry inclined loads with horizontal force components.
- To control settlements when spread footings or a mat is on a marginal soil or is underlain by a highly compressible stratum.
- In offshore construction to transmit loads above the water surface through the water into the underlying soil. This is a case of partially embedded piling subjected to vertical (and buckling) as well as lateral loads.
- To carry part of the load to deeper soil for reducing the settlement as in piled raft foundations.

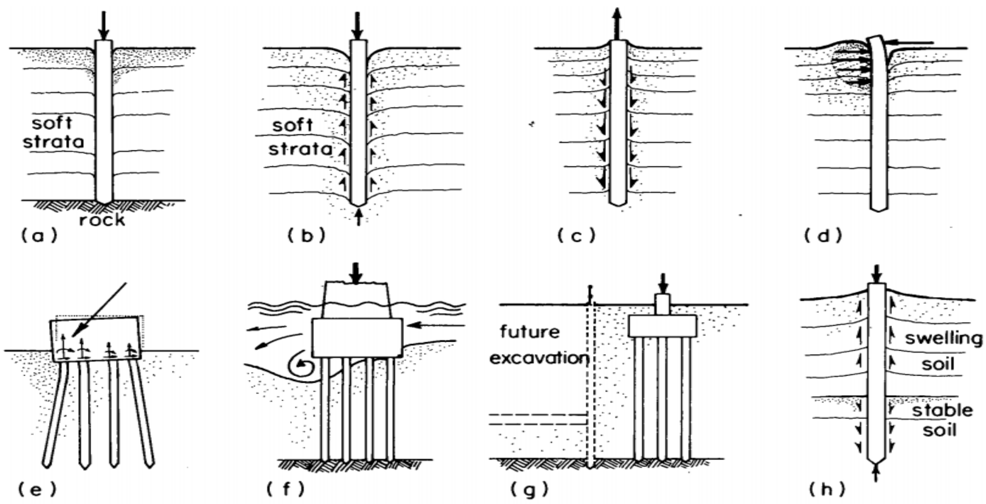


Figure 1. 7 Typical pile applications (after Vesic 1977)

1.2.2. Pile foundation types

Piles can be classified traditionally under three basic categories of bearing piles into large displacement piles, small displacement piles and replacement piles (M.J Tomlinson & J.C Woodward, 2015). Piles can as well be classified based on material, pile diameter, on the amount of disturbance during installation, method of pile installation into the ground, method of pile fabrication, and the method of load transfer.

1.2.2.1. Types according to pile material

This classification identifies piles on the basis of their principal material, such as timber; concrete, steel and composite piles. Common composite piles are either made of timber and concrete or steel and concrete. Also, polymer piles are being produced. These are usually tubular, filled with concrete. They are still rarely used.

a. Timber piles

Timber piles have been used since antiquity. The natural cross section of these piles is approximately circular (square piles are also available), with diameter varying slightly from one end of the pile to the other as shown in (figure 1.11). Timber piles have varying diameters because tree diameters tend to be larger near the roots and smaller near the branches. The piles diameters and lengths range from 150 to 400mm and 6 to 20m, respectively (Salgado, 2006; Prakash & Sharma, 1990). Timber piles are always installed by driving them into the ground. Timber piles are susceptible to termites, marine organisms, and rot within zones exposed to seasonal changes so require treatment before use.

Figure 1. 8 Timber piles (JICA, 2019)

b. Concrete piles

Concrete piles can either be cast-in-place by pouring concrete into a predrilled hole or are precast piles installed by driving them into the ground. Precast concrete piles are manufactured in plants and transported to the job site. In very large projects, temporary plants may be set up at the site to manufacture the piles. Precast concrete piles are either reinforced or prestressed concrete piles. Cast-in-place concrete piles are installed by drilling a hole into the ground, and then filling with concrete.

Figure 1. 9 Concrete piles (www.nationalpilecroppers.com, 2021)

c. Steel piles

The most common types of steel piles are pipe piles and H-piles (figure 1.13), although other types of cross sections are occasionally used (circular or C sections). Advantages of steel piles include their high resistance to driving and handling, as well as their large lateral stiffness. The obvious shortcoming of steel piles is their susceptibility to corrosion in marine environments and the cost.

Figure 1. 10 H-Steel piles (foundation.blogspot.com, 24 April 2014)

1.2.2.2. Types according to pile diameter

Referring to their size, piles can be distinguished into small-diameter (less than 300 mm) bored piles or micropiles (Bellato et al, 2016) also known as minipiles and are used as alternatives to conventional piles and as anchors in retaining systems and slopes, medium (300 mm ≤ d ≤ 600 mm) and large diameter (d ≥ 800 mm) piles. Of course, these limits are largely conventional but they are of some practical use since design criteria are different for piles of different size.

1.2.2.3. Types according to the amount of disturbance during installation

Pile types based on the amount of ground disturbance during pile installation can be placed into large displacement piles, small displacement piles and nondisplacement (replacement) piles

a. Large displacement piles

Commonly known as displacement piles. These are piles that displace soil during their installation which could be through jacking, driving or vibration into the ground. The piles here could either be prefabricated or cast-in-place.

i. Prefabricated
  - Wood
  - Concrete: reinforced, prestressed
  - Steel: closed ended pipes

ii. Cast-in-place
  - Closed end concrete/steel piles driven with mandrel, left in place and filed with concrete (Raymond piles)
  - Recoverable casing and expanded base (Franki piles)
  - Displacement screw piles (Atlas, Omega piles)

b. Small displacement piles

Piles displace a relatively small amount of soil during installation. These categories are based on the amount of soil disturbed. The terms “large” or “small displacement” used are for qualitative description only, since no quantitative values of displacement have been assigned (J.Wiley & Sons, 1990). We can have examples like;

- Small displacement (driven H or I sections, open ended pipes with soil inside removed, casing recovered after concreting)
- Continuous flight auger (CFA) with large central stem, extracted with partial soil removal but allowing the placement of reinforcement before concreting

c. Nondisplacement (Replacement) piles

Piles that do not displace soil during installation. These piles are formed by first removing the soil by boring and then placing prefabricated or cast-in-place pile into the hole from which an equal volume of soil was removed. Their placement causes little or no change in lateral ground stress and consequently, such piles develop less shaft friction than displacement piles of same size and shape. Pile operation are done by methods such as augering (drilling, rotary boring) or by grabbing (percussion boring).

1.2.2.4. Types according to the method of pile installation into ground

Pile types based on the method of pile installation into the ground can be divided into driven piles, bored (or drilled) piles and a combination of driven and bored piles. Timber, steel (both H-piles and pipe piles) and concrete (both the precast and compacted expanded base piles) are examples of driven piles. Bored piles are necessarily cast-in-place concrete piles.

1.2.2.5. Types according to the method of pile fabrication

Pile types based on the method of pile fabrication identifies piles if they are prefabricated that is precast or cast-in-place. Timber and steel piles are always prefabricated. Concrete piles on the other hand, can either be precast or cast-in-place.

1.2.2.6. Types according to the method of load transfer

Piles here are classified based on the method of load transfer from the pile to the surrounding soil and consists of end bearing piles, friction piles, laterally loaded piles, tension piles, and settlement reducing piles.

a. End bearing piles

End bearing piles are driven through soft and loose material and their tips rest on the underlying stiff stratum such as a rock layer or very dense sand or gravel. Therefore, most of the bearing capacity of these piles are derived from the tip resistance of the pile. These piles derive most of their carrying capacity from the penetration resistance of the soil at the toe of the pile (Beakawi, 2016).

b. Friction piles

Also referred to as floating piles. In these piles, most of the carrying capacity is from the skin friction resistance of the pile. This is mostly common in cases in which the pile is not able to reach the underlying stiff stratum, but that the pile is just driven into the ground to a certain level. The tip bearing resistance to a lesser degree, also contributes to the total load bearing capacity of the pile but most of the load is transmitted to the surrounding soil by the adhesion or friction between the surface of the pile and the soil. Therefore, the load bearing capacity of these kind of piles is directly proportional to their length meaning the longer the pile is embedded into the ground surface, the greater the skin frictional resistance of the pile, therefore the greater the pile bearing capacity.

c. Laterally loaded piles

Piles are not only capable of transferring the axial loads to the ground surface but the horizontal (or inclined) loads as well. In cases where the magnitude of these horizontal loads are small when compared to the axial loads, they are often neglected in the design. Piles are usually used for structures such as tall buildings, bridges, offshore platforms, defence structures, dams, transmission towers and earth retaining walls. For structures such as the bridge piers, tall chimneys and retaining walls, the horizontal components have to be considered in their design as they are relatively large. The most common source of the lateral force are gushes from the wind which is most effective in tall buildings and transmission towers. Another major cause is seismic activity for example from earthquakes which are capable of generating lateral forces which the piles would not be able to withstand. Therefore, as seem, the principal purpose of laterally loaded piles is in resisting lateral forces and bending moments in a structure hence they are designed as thus. An example can be seen in earth retaining structures where they are meant to resist the lateral forces that the soil masses are exerting on them from behind the retaining walls. Another case could be in slopes of potential landslides were slow ground movements are occurring. These piles are installed in this slope and they are capable of hindering or slowing down these ground movements. These piles can also be referred to as batter piles.

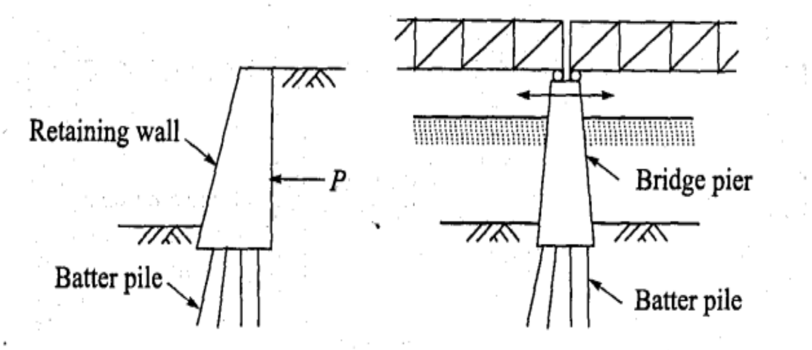


Figure 1. 11 Piles used to resist lateral loads (Source: V.N.S Murthy, 2007)

d. Tension piles

Tension piles can also be known as the uplift piles or the anchor piles. As their name implies, these are vertical piles are used to oppose the uplift forces that are acting on the ground against the structure. These uplift forces could be as a result of several factors such as the seismic forces which could arise due to earthquakes and other vibrations that may be imposed on the earth surface, the overturning moments and the hydrostatic pressures of the soil amongst others. For very tall structures or building constructions, significant wind loads can be experienced that can lead to high overturning moments that can result in uplifts. The main resistance components of tension piles are the shaft frictional resistance that is capable of resisting these produced uplift forces therefore they are constructed with sufficient depths. In some situations, the hard underlying soil layer such as the rock stratum and prevent the piles from getting to these depths. One solution to this problem could be by anchoring the pile head to the rock stratum or by increasing the dead weight of the piles.

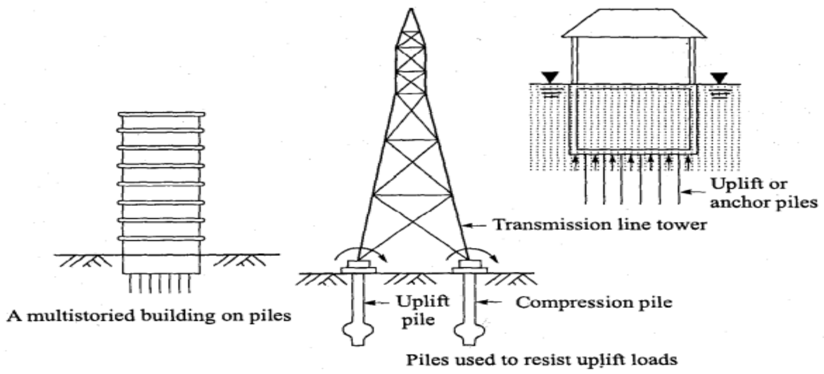


Figure 1. 12 Piles for multi-storey buildings and to resist up lift loads (Source: V.N.S Murthy, 2007)

e. Settlement reducing piles

The function of these piles is to reduce differential settlements of the superstructure to a reasonable level, so as to avoid damages. These piles are usually used beneath the central part of the raft foundations.

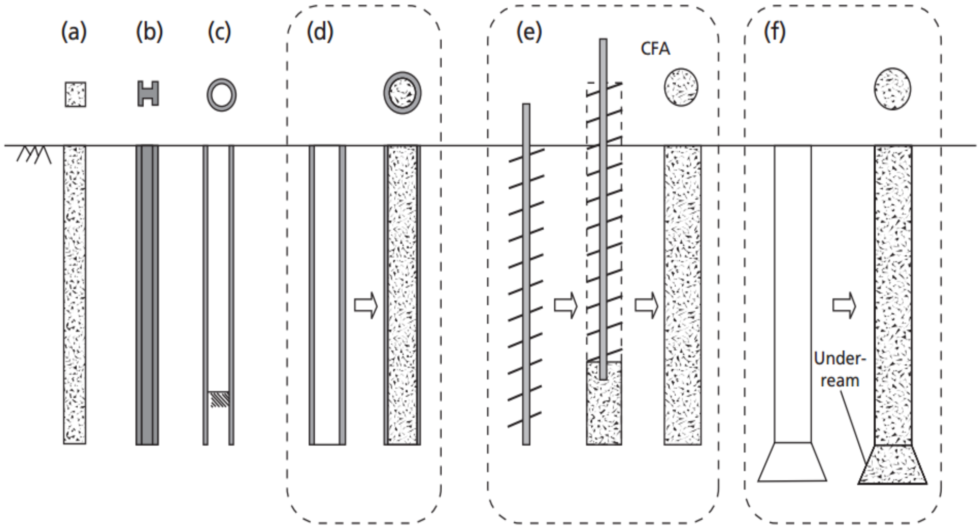


Figure 1. 13 Principal types of pile: (a) precast RC pile, (b) steel H pile, (c) steel tubular pile (plugged), (d) shell pile, (e) CFA pile, (f) under-reamed bored piles (cast-in-situ) (Source: Knappett &Craig, 2012)

1.2.3. Factors governing choice of type of pile

According to VNS Murthy (2007) the selection of the type, length and capacity is usually made from estimation based on the soil conditions and the magnitude of the load. In large cities, where the soil conditions are well known and where a large number of pile foundations have been constructed, the experience gained in the past is extremely useful. Generally, the foundation design is made on the preliminary estimated values. The foundation design must be revised according to the test results. The factors that govern the selection of piles are:

- The length of pile in relation to the load and soil condition
- Character of structure
- Availability of materials
- Type of loading
- Factors causing deterioration
- Ease of maintenance
- Estimated costs of types of piles, taking into account the initial cost, life expectancy and cost of maintenance, and

- Availability of funds

All these factors have to be largely analysed before deciding upon a particular type of pile to use in a project.

Conclusion

The purpose of this chapter was to give a general concept of pile foundation as a major type of deep foundation. Due to the importance of the soil to a foundation, the description of the soil investigation methods, classification and characterisation, important soil parameters was made as well as the description of pile foundation indicating the function and application, the different types according to material, diameter, amount of disturbance during installation, method of installation, method of fabrication, method of load transfer and some factors governing the choice of the type of pile to use. The quality of the foundation depends on the soil information obtained from the site. The next chapter present the description of the quality control of piles.

CHAPTER 2. QUALITY CONTROL OF PILE FOUNDATION

Introduction

The previous literature review made a description from the soil recognition to the selection of pile type. This chapter presents the quality control of the foundation type.

Pile quality control are the suitable methods by which well established procedures and test standards are used to verify the strength and the consistency of the pile materials. The quality control of a pile foundation can be assured by a proper control of the soil parameters obtained through investigation, a control of the foundation design process (verification of design values) which can either by controlled by a static load test on a test pile or other methods, a good supervision of the pile installation method and then a quality control (integrity) test to evaluate the constructed pile foundation. This chapter focuses of the pile design process, pile foundation failure, soil-pile interaction as it has an influence on the design of piles and then the testing of piles with the use of non-destructive testing (NDT) methods.

2.1. Pile foundation design

The design of the foundation can be divided into two stages: geotechnical (bearing capacity) of the foundation, and structural design (determination of the reinforcement from working loads). Piles must be able to sustain axial and horizontal loads without suffering structural damage, without failing in bearing capacity and without undergoing excessive settlements or deflections. Due to some uncertainties in soil properties, inaccuracies in the method of analysis and allowable settlement, uncertainties on the applied load, geometry of the foundation, behaviour and characteristic of the superstructure, following well defined steps, appropriate design of a foundation require that a factor of safety (FS) should be taken into consideration. This section describes the methods and steps used for designing a foundation, laying emphasis on the geotechnical design steps.

2.1.1. Design methods

Three basic design methods using factors of safety to achieve safe, workable structures have been developed:

2.1.1.1. The permissible stress method

The ultimate bearing capacity (q_{lim}) is reduced by a lumped factor of safety in order to obtain the allowable bearing capacity (q_{adm}), which is usually within the elastic range. This method has proved to be simple and useful but with some serious inconsistencies, since based on an elastic stress distribution, it is not really applicable to semi-plastic material neither is it suitable when the deformations are not proportional to the load. Also, it has been found to be unsafe when dealing with the stability of structures subjected to overturning forces.

2.1.1.2. Load factor method

In this method the factor of safety is applied to the working loads, but no factor of safety is applied to the material stresses and so cannot directly take account of the variability of the material. Hence, the ultimate strength of the material should be used in the calculations.

2.1.1.3. Limit state design method

The limit state method of design overcomes many of the disadvantages of the permissible stress and load factor methods. This is done by applying partial factors of safety both to the loads and to the material strengths, and the magnitude of the factors may be varied so that they may be used either with the plastic conditions in the ultimate state or with the more elastic stress range at working loads. A limit state is a set of conditions to be avoided; it may either be ultimate and the serviceability limit state.

The ultimate limit state is associated with the concept of failure. In foundation engineering, it results from a bearing capacity failure and/or excessive differential settlement leading to partial or full structural collapse. Check of the ultimate limit state requires that the factored load should not exceed the resistance as computed by factored shear strength parameters.

A structure reaches serviceability limit state when it does not perform its intended function. In foundation engineering, it is concerned with settlement. Excessive settlement, even if uniform, might lead to failure of connections to utility or access problems. The design against serviceability limit states requires that the overall and differential displacement of a foundation should not exceed admissible values. Prevention of serviceability limit state usually precludes ultimate limit state, but checks are carried out independently.

The limit states that need to be considered in the design of piles are the:

- Loss of overall stability
- Bearing resistance failure of the pile foundation
- Uplift or insufficient tensile resistance of the pile foundation
- Failure in the ground due to transverse loading of the pile foundation
- Structural failure of the pile in compression, tension, bending, buckling or shear
- Combined failure in the ground and in the pile foundation
- Combined failure in the ground and in the structure
- Excessive settlement
- Excessive heave
- Excessive lateral movement
- Unacceptable vibrations

2.1.1.4. Design steps

Viggiani et al (2012) proposes for the design of any foundation (including piles), the steps required are:

- Collection of geological evidence and any other available information on the subsoil; development of geotechnical characterisation of the soil.
- Determination of the magnitude, nature and distribution of the loads exerted by the structure on the foundation.
- Choice of the type of the foundation.
- Determination of the bearing capacity of the foundation. Choice of a tentative service load, obtained assuming a suitable margin of safety against collapse (bearing capacity failure)
- Prediction of the total and differential settlement of the foundation; assessing its admissibility taking into account the static and functional characteristic of the structures.

- Evaluation of the stress in the foundation structure and structural design.
- Definition of the techniques and preparation of the technical specifications.
- Evaluation of the cost, also to assist in the choice between possible alternative solutions.

2.1.2. Bearing Capacity under vertical load

Vertical load is by far the most common and most relevant load condition for pile foundations, as for any other kind of foundation. Concerning pile foundations, the resistance to vertical load is provided by friction along the surfaces of the pile (called skin friction) and by the resistance at the base, called end bearing resistance or tip resistance or base resistance. Values for skin resistance and base resistance can be obtained separately since they are independent of each other. It is therefore essential to calculate the bearing or ultimate load capacity of a pile. There are many approaches to determine the bearing capacity of a single pile. Only the most used in the world (limit state design) will be developed.

The design axial compressive load $F_{c;d}$ is obtained by multiplying the representative permanent and variable loads, G and Q by the corresponding partial action factors γ_G and γ_Q:

$$F_{c;d} = \gamma_G G + \gamma_Q Q \qquad 2.1$$

The two sets of recommended partial factors on actions and the effects of actions provided in Appendix 1 (representing Table A3 of Annex A of EN 1997-1).

The bearing capacity can be evaluated by means of the analytic method, the empirical method, Full scale load test and Dynamic formulas.

2.1.2.1. Analytical method

The analytical method here is that presented in section 3.4.2.1 of this work.

2.1.2.2. Empirical method

The empirical method is based on the results of in situ tests such as the static cone penetration test (CPT) and the standard penetration test (SPT). Correlations (1.14) and (1.15) for the tip resistance and skin resistance respectively can be obtained from the SPT data.

$$q_b = C_b N_{60} \tag{2.10}$$

Where:

N60 is the value of the standard penetration resistance in the vicinity of the pile base and C_b is a soil dependent constant whose values are given in after Poulos (1989).

$$q_s = C_s \overline{N}_{60} \tag{2.11}$$

Where:

$\overline{N}_{60}$ is the average value of N_{60} along the length of the pile and Cs is a constant which in unknown soil can be taken as 2 (Clayton, 1995).

The cone penetration test (CPT) is particularly used for displacement piles since the mode of pile installation is similar to the CPT. The base resistance q_b is related to the average cone resistance $\overline{q}_c$ in the vicinity of the pile base after equation (2.12).

$$q_b = c_{cpt} \overline{q}_c \tag{2.12}$$

Where:

C_{cpt} depends on the pile and soil type. Suggested values of C_{cpt} based on findings of Jardine et al (1995) and Lee and Salgado (1999).

2.1.2.3. Pile load test

The information about this method is presented in section 2.4.1.

2.1.2.4. The dynamic formula method

This method is based on the driving energy of driven piles. The basis of these formulae is the simple energy relation which may be stated by equation (1.17), as proposed by Sander (1857).

$$Q_u = \frac{Wh}{s} \tag{2.13}$$

Where:

Q_u: ultimate resistance to penetration h: height of the harmer

W: weight of the driving harmer

s: pile penetration under on harmer blow

By taking into account energy dissipated in elastic rebound e, Wellington (1893) suggested the formula given by equation (1.18), with n = 1 and e = 2.5 mm.

$$Q_u = \frac{nWh}{s + \frac{e}{2}} \qquad 2.14$$

Where:

n is the efficiency of the harmer.

Hiley (1925) went further by considering the energy loss due to pile deformation, the cushion and the soil, and by using Newton's theory of impact between two bodies coming out with the formula given by

$$Q_u = \frac{nWh}{s + \frac{c_1 + c_2 + c_3}{2}} \frac{W + Cr^2 W_p}{W + W_p} \qquad 2.15$$

Where:

Wp: weight of pile

Cr: coefficient of restitution

C1: elastic compression of the pile cap

C2: Elastic compression of the pile

C3: Elastic compression of the soil generally taken as 2.5 mm

2.1.3. Settlement

The term settlement can be defined as the vertical displacement of a loaded area of the base of the foundation or of the ground surface (Salgado, 2006). The design against serviceability limit state requires that the total settlement and distortion of a foundation be kept below corresponding admissible values.

As for shallow foundations, verification of the SLS involves ensuring that the settlement of the pile under the applied action will not adversely affect the supported structure. The settlement of a pile is much more difficult to compute than that for a shallow foundation for a number of reasons, including

- constitutive behaviour – the mechanisms of stress transfer at the base and along the shaft of the pile are very different;
- layering – piles often pass through layers of soil with dissimilar stiffness;
- pile slenderness (Lp/D0) – because piles are long compared to the cross-sectional area, the compression (shortening) of the pile itself can be significant in magnitude;
- elasto-plastic condition – loads carried by the pile are highest at the top, where the strength of the soil is weakest; therefore, soil may be at failure towards the top of the pile and elastic at depth while still being below the ULS (as this requires all of the soil to be at failure).

As a result of these difficulties, piles will always be load tested to the working load to verify that the SLS has been met. In cases where many piles of the same design will be used, only a proportion of the piles will need to be tested (Craig & Knappett, 2012). However, in order to minimise any possible re-design, a good estimate can be made using analytical or numerical techniques, or based on previous experience of design in similar ground conditions, to produce a pile which can confidently be expected meet the SLS during the load test.

2.1.3.1. Analytical method

Three analytical techniques will be considered in this section.

- Randolph and Wroth method
- T–z method
- hyperbolic method

a. Randolph and wroth method

This method considers the soil to behave elastically and the pile to be axially rigid in comparison. Simple closed form solutions for the vertical stiffness of a pile can be derived, from which the displacement under a given load can be determined. The steps involve with this method are present in section 3.4.2.2

b. T-z method

The pile is split up into a number of discrete sections, to each of which a spring is attached which represents the soil–pile interaction. The properties of these springs may be determined analytically using the elastic models of the Randolph and Wroth model, defining the reaction on the pile T for a given relative pile–soil displacement z. Having defined the T–z springs, the resulting set of equations can be solved iteratively using a Finite Difference Scheme; the method is thus amenable to computer analysis using a spreadsheet. The use of a finite difference scheme allows this method to account for pile compressibility and variation in soil or pile properties along the length of the pile. The complete steps of the method is given by Craig& Knappett (2012).

c. Hyperbolic method

The method is based on the observation that the overall pile load–settlement curve can in almost all cases be reasonably approximated by a hyperbolic curve. This method requires knowledge of Q_{bu} and Q_{su} in addition to some additional fitting parameters determined empirically from a large database of pile load tests (to which the method is fairly insensitive). This is also amenable to solution using a spreadsheet, and has the added advantage that data from a subsequent load test can be used to update the soil parameters used in the analysis for subsequent application to other piles in the same unit of soil which might be carrying different working loads. The complete steps of the method is given by Craig& Knappett (2012).

2.1.3.2. Settlement from in-situ tests

The relation used in obtaining the settlement of the foundation of width or diameter, from pressuremeter test results parameters is given in equation (2.16) (Briaud, 1992).

$$S = q_s\left[\frac{2}{9E_d}B_0\left(\lambda_d\frac{B}{B_0}\right)^{\alpha_r} + \frac{\alpha_r}{E_c}\lambda_c B\right] \qquad 2.16$$

Where:

q_s: net average bearing pressure at the footing base

Ec: pressuremeter modulus of the first layer under the footing

Ed: equivalent harmonic mean pressuremeter modulus for 16 layers, each B/2 thick, below the footing B_0: reference width equal to 60 cm

B: footing width or diameter ($B \geq B_0$)

α_r: rheological factor linked to the state and granulometry of the soil

λ_c, λ_d= shape factors function of the dimension of the footing

Correlation from CPT and SPT as proposed by Schmertmann et al. (1978), and Burland and Bridge (1985), respectively are given in Appendix 3.

2.2. Pile foundation failure

The failure of foundation can lead to the collapse of structures. The causes of failure can be classified as design causes concerning the failure due to the bad design, natural causes from natural phenomenon like erosion, accidental causes that can occur accidentally like earthquakes and construction causes concerning the poor realisation of the designed piles, the bad quality of the materials, the man errors. In the case it is possible, we can prevent those failures by investigating well the soil or remedy to the failure by improving the soil or underpinning the foundation. To understand the cause in the city of Douala, it is inserting to study a case of collapse in the city.

Unlike piles, which are manufactured in a factory (e.g., steel pipe piles) or a casting yard (e.g., precast concrete piles), drilled shafts are “manufactured” at the site. Anomalies often develop during the construction of drilled shafts as shown in Figure 9-57a. An anomaly is a deviation from an assumed uniform geometry of the shaft and/or from the required physical properties of the shaft. Typical anomalies may include necking or bulging, “soft bottom” conditions, voids or soil intrusions, poor quality concrete, debonding, lack of concrete cover over the reinforcement steel and honey-combing. Non-destructive test (NDT) methods are used for Quality Assurance (QA) integrity testing of drilled shaft foundations to identify anomalies

2.2.1. Causes of pile foundation failure

Pile foundation being the most popular choices for heavy loaded structures and in cases where poor soil conditions are found at a shallow depth may fail due to different reasons. Proper precautions must be taken before designing pile foundations so that the possibility of such failure reduces. Most damage account to three main ground movement that is, settlement (the downward movement of the ground usually occurring in new or relatively new structures), subsidence (the vertical downward movement of a building foundation caused by the loss of support of the site beneath the foundations) and heave (the upward movement of the ground) which are linked to the soil´s features, water in the soil, man and natural effect.. This ground movement are results of different phenomenon.

2.2.1.1. Design causes

Geotechnical investigation is the first step in construction. An inadequate realisation of this can lead to many foundation failures. The problem is, if ground investigation is not well realized, information like soil characteristics, groundwater level will not be available which are useful for foundation choice and design.

Design error is one of the causes of foundation failure occasioned by construction engineer. For example, when a soil undergoes a water level uplift after rainfall, the foundation design should be done taking in account a reduction of bearing capacity or design in undrained condition if following Eurocode 7, what is not done most of the time.

Other design failures could arise such as an inaccurate determination of the bearing capacity of pile, buckling of piles due to insufficient lateral support, under-estimated pile loads, inadequate pile reinforcement design, bearing pile laying on delicate strata, etc.

2.2.1.2. Natural causes

a. Shrinkage or swelling

Most commonly foundation failure is caused by the movement of expansive and highly plastic soils beneath different sections of the foundation. An expansive soil is a soil which exhibit large volume changes when their water content changes. The volume changes conduct to movement that can be in the form of shrinkage, which causes settlement, or expansion, which causes heave. Soil shrinkage is reduction of bulk volume that occurs during drying, so when dry conditions prevail, soils consistently lose moisture and shrink. It occurs in varying degrees in all soils but most in clayey soil. Also, if a soil shrinks it can usually swell. When moisture levels are high, soils swell. Alternate heave and settlement due to seasonal climatic variations result in distress and damage foundation.

Hot dry wind and intense heat will often cause the soil to shrink beneath the foundation. Improper drainage is one of the leading causes of foundation failure. Poor drainage from yard run-off and gutter downspouts discharging at the base of the foundation, plumbing leaks will heavily increase the moisture. Excess moisture will erode or consolidate soils and cause settlement (civiltoday.com).

b. Soil erosion and scour

Erosion is defined by the International Building Code (IBC, 2006) as the "wearing away of the ground surface as a result of the movement of wind, water or ice". Erosion can occur across a wide range of timeframes –it can be gradual, occurring over a long period of time (many years); more rapid, occurring over a relatively short period of time (weeks or months); or episodic, occurring during a single coastal storm event over a short period of time (hours or days).

The closer a structure is to the shoreline, the more likely erosion will occur and the greater the erosion depth will be. Erosion and scour have several impacts on foundation, which are more observable on coastal structures:

- Erosion and scour reduce the embedment of the foundation into the soil making buildings on deep foundations more susceptible to settlement, lateral movement, or overturning from lateral loads.

- Erosion and scour increase the unbraced length of pile foundations, increase the bending moment to which they are subjected, and can overstress piles.
- Erosion over a large area between a foundation and a flood source exposes the foundation to increased lateral flood loads (i.e., greater Still water depths, possible higher wave heights, and higher flow velocities).

If the foundation embedment into the ground is not sufficient to account for erosion and scour that may occur over the life of the structure, the structure is vulnerable to collapse under design flood and wind conditions. When foundation of a structure rests on rock, the stability and subsequent damage of foundation depends on strength and durability of rock. Some rocks have greater strength but possess lesser durability and durability is very important factor in respect to longevity of foundation and structure over it.

c. Changes in ground water level

It is important to note that both rising and falling groundwater levels can affect soil behaviour. Rise in GWT reduces the bearing capacity of the soil and on the other hand rapid fall in the GWT causes ground subsidence or formation of sinkholes due to increased overburden effective stress value. The former is induced naturally due to heavy rain or seepage flow but the latter is caused due to human activity such as uncontrolled pumping or dewatering during construction of deep basement. It is clear that the effect of water on geotechnical properties is significant. Formation of sinkhole is another major cause of foundation failure due to increased water usage, altered drainage pathways, overloaded ground surface, and redistributed soil. The majority of the cases related to formation of sinkholes are associated with collapsible soils.

2.2.1.3. Accidental causes

a. Earthquakes

Earthquake is the shaking of the surface of the Earth, resulting from the sudden release of energy in the Earth's lithosphere that creates seismic waves. Earthquakes are the most common cause of foundation failure due to ground vibrations. Violent shaking of an earthquake is capable of damaging homes, buildings, bridges or any other man-made structures. The most noticeable damage appears in the walls or roofs of buildings, but building foundations are also drastically affected by the Earth's sudden movement. During an earthquake, the foundation of the building moves with the ground and the superstructure and its contents shake and vibrate in an irregular manner due to the inertia of their masses (weight).

b. Landslide

Foundation failure due to rapid movement of landmass over a slope results when a natural or man-made slope on which structure exists becomes unstable. The major causes of slope instability/landslide can be identified as, steep slope, groundwater table changes /heavy rainfall, earthquakes and other vibrations, and, removal of the toe of a slope or loading the head of a slope, both of which may be the result of man-made and geological factors. The resistance to a landslide or slope instability is offered by the type of soil and the geometry of the slope.

2.2.1.4. Construction causes

During construction, adequate sequence of realisation if not followed can make damage on the foundation. There are two common sources of the construction errors, temporary protection measures, error relating to temporary shoring, bracings and temporary coffer dams, and foundation work itself. The former concern all the works done after foundation construction.

When foundation is subjected to any failure causes, it is no longer able to sustain the structure. Foundation failure describes the way a foundation is damaged. It is linked to a cause of failure and can have different causes. For each failure type there is a possibility to remedy, this remedy can be preventive or repairing but in some case like earthquake there is no way to prevent.

2.2.2. Possible remedies to Pile foundation failures

Some of the precautions or remedies that can be taken to assist pile failures are possibly

- Treatment of timber piles with creosote, oil-borne preservatives, or salts. Creosote application by pressure treatment is the most effective method of protection for long preservation.
- Concerning concrete piles can be protected against destructive environments by painting, asphalt impregnation, steel points, concrete armor, shotcrete encasement, wrought iron armor, creosoted wood jackets, and fabriform pile jacket.

- Steel piles can be remedied by providing additional metal to increase pile section, isolate pile from its surroundings by either surface coating or by encasement, and Cathodic protection method
- Early repair such as encasement or replacement
- Removal of partial load
- Underpinning
- Injection technique
- Pressure grouting through core holes

2.2.3. Some common deformities of piles

Occurrences of deformities and flaws in the pile structure is what in the first place gives necessities for the pile integrity test and other test. This is because either directly or indirectly, these deformities that appear affects the normal functionality of the pile according to its design. The occurrence of the deformities in the pile structure could either be at the level of the fabrication or at the level of the installation. Either way, the deformities interfere with the normal functionality of the pile so therefore they must be detected and dealt with. This part presents a brief discussion of some of the most common pile deformities, which are, soil inclusion, bulging, necking, major voids, and cracks.

2.2.3.1. Soil inclusion

When unwanted material finds its way into the pile structure, it could result to negative impacts on the overall functioning of the pile. These unwanted materials could be in the form of soil lumps, stones, sticks etc. The inclusion of this foreign material could go as far as affecting the bearing capacity of the pile in terms of reduction so their neglect is not wise. Figure 2.1 demonstrates an inclusion occurring in a pile, thereby causing pile defect.

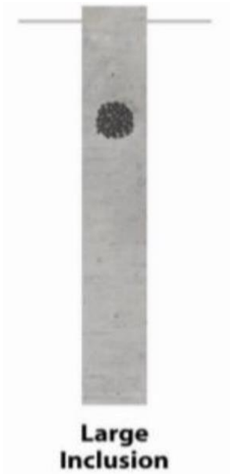


Figure 2. 1 Soil inclusion deformity in pile structure (Olson instrument, 2012)

2.2.3.2. Bulging

This is a kind of pile deformation that affects the shape of the pile. Bulging is an increase in pile cross sectional area at a certain depth of the pile. That is that potion of the pile length where the cross-sectional area is relatively higher than the intended cross-sectional area for design of a pile, occurring at a particular depth along the pile length. Since bulging increase the cross-sectional area of a pile, it can lead to an increase in the pile bearing capacity and design strength but however it is still considered as a deformity that has to be investigated.

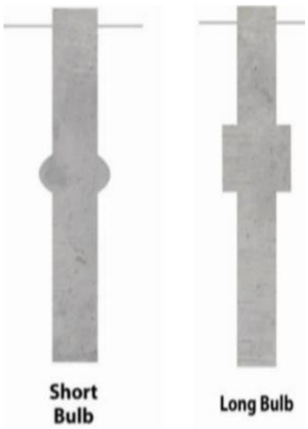


Figure 2. 2 Bulging deformation in pile structure (Olson instrument, 2012)

2.2.3.3. Necking

Necking occurs when there is a significant decrease in cross sectional area of the pile at a certain depth relative to the overall cross-sectional area of the pile. Necking reduces load bearing capacity of the pile, and also increase the chances of compressive failure to occur at the depth in which the necking has occurred.

Figure 2. 3 Necking deformity in pile structure (Olson instrument, 2012)

2.2.3.4. Major voids

These are the areas of the pile structure that are empty of any material thus rendering the pile structure inconsistent. If the voids are minor voids, problems could arise if they occur in many locations. Voids, greatly reduce the quality of the pile and pile bearing capacity. Just like necking just like necking, voids also contribute to the reduction in pile cross sectional area. When a major void occurs in the pile, the integrity testing of the portion of the pile that lies underneath the void becomes hindered, because the signals that provide the information necessary for analysis are not able to pass through the void.

2.2.3.5. Cracks

Concrete material is good at resisting compressive forces, but very poor at resisting tensile forces. Cracking of concrete always begin when the tensile stresses that are acting on the concrete, becomes greater than the concrete can withstand. This can usually be an indication of tensile failure. If the crack occurs but in a pile at a particular depth, due to the tensile stresses it underwent, this just like most deformities would affect the load bearing capacity of the pile. Cracking can be detected by the integrity tests but if the crack is through the entire width of the cross-sectional area of the pile, then the integrity testing of the part underneath would pose problems as it would interrupt the descending signals from trespassing the crack to the pile toe.

This could get worse, because the signals from these types of cracks could easily be confused with the signal from the pile toe.

Figure 2. 4 Crack deformity in pile structure (Olson instrument, 2012)

2.2.4. Review of failures due to lack of integrity testing

Numerous failures with disastrous consequences has occurred in the past due to negligence and underestimation to the importance of integrity testing on deep foundations in history. In this part are given three instances as accounted for by Amir in his book "Pile Integrity Testing: History, Present Situation and Future Agenda" which were as a result of these negligence. The three cases reported here are the John Hancock Centre, a building constructed in America in 1969, another incidence at the Tel Aviv tower in 1996 and the last incident in Hong Kong in 1999. A brief account of these unfortunate events due mainly to negligence in the application of the LSIT.

2.2.4.1. The John Hancock Centre

When the John Hancock Centre was completed in 1969, it was one of the tallest buildings at that time. During the construction of this infrastructure, something unexpected happened that was a major call for concern. While the construction was only 20 stories high, huge settlements of this building infrastructure began to occur, and these settlements should be expected if the building was over 90 stories which was not the case. Apparently, this was enough indication that something is seriously wrong so the construction had to be halted for verification of this problem.

Obviously, the foundation of this building was a deep foundation, and it was composed of 57 concrete caissons. The boreholes for these caissons where produced by massive drills, that drilled right down to the bed rock. After the drilling, casings were then placed into the bore holes, but they were placed in stages. During the first stage, the casing would be place in the hole till a certain level. After that, concrete was then poured into this casing and allowed to harden. After hardening of the poured concrete to a certain degree, the casing is then pulled up to a point where the bottom of the casing is in contact with the tip of the previously hardened concrete. That is, the casing is not pulled out completely, such that there is an overlap. Then the empty part of the concrete is poured again with concrete, and allowed to harden to a certain degree. Then the process continues until the bore hole is filled to the top. After the completion of the caissons, then the construction of the mega structure began. While the infrastructure was still at the level of the first floor, excessive settlements of one of the caissons was noticed, under a load of just 120KN, which of course was beyond normal. The project was halted and a testing program was launched. The tests that were carried out was a number of non-destructive testing techniques and coring (Baker and Khan, 1971). The testing program took up to 4 months, and cost about 11 million US dollars, which is equivalent to about 26 million today.

2.2.4.2. The Tel Arviv 1996

A tower was being constructed in 1996, in Tel Arviv, on a large piece of land, of about 6000 square meters. It was being constructed as a residential tower with three basement levels. A large wall was also being constructed at the site, with the use of polymer slurry, a material which at the time was not known to the contractor, neither the origin of this material. In order to save more money, integrity testing of the wall was neglected. Then after the excavation of the wall up until it was almost completed, the defects of the wall started becoming more and more apparent. Because of this the contractor had to begin excavation for repairs which was costly. These repairs cost several million dollars which was even far more expensive than the integrity test that were originally ignored. It also cost considerable time and energy as well. Figure 1.17 below shows the flawed diaphragm wall of the Tel Aviv as a result of a lack of integrity testing.

Figure 1. 14. A flawed diaphragm wall in Tel Aviv. (Amir, 2017).

2.2.4.3. Hong Kong 1999

In Yuen Chau Kok, a site in Japan in 1999, five residential buildings were being constructed. They were each, four stories high and required deep foundations with large diameter bored piles. The company that demanded the construction of these building infrastructures was the Hong Kong housing department, who later on hired the Zen Pacific Ltd, to do the construction works. The piles that were to be constructed for these buildings was to penetrate and go right down to the solid granite rock, passing the unstable layers such as the marine deposits and the decomposed granite. The Hong Kong department, before the project, instructed the contractor responsible to use continuous steel casing in order to avoid caving of the pile foundation. Zen

Pacific, then took the work and subcontracted it to Hui Hon Ltd that did not have good enough drilling rigs as well as sufficient casing material to go about the project. Hue Hon Ltd, during their construction works began to encounter massive borehole collapse. They reacted to this phenomenon, by replacing the casing with super mud, which did not remedy the situation in any way. Because of this, a lot of piles had to be abandoned, and these piles had not even reached the bed rock. At this level, Hui Hon Ltd, seeing that they were headed towards bankruptcy, began to cover up their errors to the detriment of the construction project. They took the advantage of the absence of supervision of the Hong Kong Department staff at night, by working during that period. They falsified site records, diverted excess concrete amounts to other projects, and blocked access tubes for cross hole testing, replacing the test with different ones which provided no useful information. They went further to begin to alter the tape measure used to check depths, and replaced defective ones with good ones taken from the other piles. This cover-up failed because the first two buildings, which were already more than 13 stories tall, began to show excessive settlements which caught the attention of the authorities. These buildings had to be demolished, costing 650 million HK dollars. Two of Hui Hon directors were sentenced to 12 years in jail, and the site agent to five.

2.3. Soil/pile foundation interaction

The behaviour of civil engineering structures such as retaining walls, concrete reinforcement, tunnels and foundations is a soil-structure interaction problem. At present, it is commonly accepted that in the study of soil-structure interaction, the transmission of forces from the structure to the ground takes place through a thin layer of soil in contact with the structure called "interface". The interface is the seat of phenomena

It generates deformation localizations and concentrations of deformations.

The analysis of the soil-structure-interaction is a very important part during all stages of planning, design and construction. Soil-structure interaction calculations constitute a basis for design decision making on a structure of any building even an ordinary housing.

Any soil structure interaction model (SSI) can be deployed successfully or can fail depending on the reliability of the estimations of the soil structure interaction stiffness parameters. The stiffness evaluation of the engineered materials' structural elements and their combined role in the structure itself, commonly can be evaluated rather easily. However, the key issue and the much more problematic interaction parameters in any successful SSI analysis are linked to the soil stiffness, at the relevant strain levels, of the specific interacting soil layers under the corresponding structural loading conditions.

In most methods for dynamic analysis, soil is modelled with springs and dashpots and sometimes lump masses (which are representative of stiffness, damping, and inertia effects, respectively).

Possible model for pile soil interaction are, the wrinkle spring (T-Z) model, The P-y curves, Beam on elastic model, and finite element model.

2.4. Pile testing

The era of the integrity testing has brought forth much value to construction on whole in the terms of saving time and energy, as well as reducing costs of construction while at the same time increasing efficiency (Amir, 2015). This part, talks briefly about some of the most common types of integrity testing methods, with an outline on their methods of operation. It also tackles some of the advantages and disadvantages of the various test, and by so doing, demonstrates the need for the coupling of several integrity tests at the same time for more reliability of the results.

If a pile is found to contain a defect, we can Disregard the flaw and accept the pile as-is, or accept the pile as a partial support and strengthen the superstructure or repair the defective pile or reject the defective pile altogether, and adopt proper remedial measures.

2.4.1. Pile load tests

By John Wiley & sons (1990) the estimation of pile load capacity and settlement under a load is based on the results of field investigations, laboratory testing and the empirical and semi empirical methods. These estimated values should then be confirmed by field pile load tests. Pile load tests, in practice, are normally executed in two alternative ways

- **Test Pile:** Preliminary pile design is first carried out on the basis of site investigations, laboratory soil testing, and office study. Pile load tests are then carried out to refine and finalize the design. For these conditions, the test piles are generally tested to failure.
- **Test on a Working Pile:** In areas where previous experience is available, pile design is carried out based on the site investigations, laboratory soil testing, and office study. Pile load tests are then carried out on randomly selected actual piles to check the pile design capacities. In these situations, the piles are generally tested to two times the capacity.

The equipment and test procedures for these two alternatives are essentially similar. The main difference is the level of final loading.

2.4.1.1. Static load test

Static load testing is the most common form of pile testing, and the method that is most similar to the loading regime in the completed foundation. Figure 2.5 shows the set-up of a static load test. A hydraulic jack is used to push the pile under test into the ground (for a conventional compression test), using either the dead weight of kentledge (typically blocks of precast concrete or iron, Figure 2.5(a)) or a series of tension piles/anchors (Figure 2.5 (b)) to provide the reaction. If kentledge is used, the weight must be at least equal to the maximum test load, though this is normally increased by 20% to account for variability in the predicted capacity. In the case of tension piles the tensile resistance is less easy to predict with certainty, so the pile or anchor system should be proof-tested before use, usually to 130% of the required test load. An in-line load cell is used to measure the force applied at the pile head, while the displacement of the pile head may be measured either using local displacement transducers or by remote measurement using precision levelling equipment. The former method is generally more accurate, though will be affected by any ground settlements around the test pile.

Static load tests are usually conducted in one of two modes. **Constant rate of penetration tests** (CRP) are used for trial piles in which a penetration rate of 0.5–2mm/min in compression is used to displace the pile until either a steady ultimate load is reached or the settlement exceeds 10% of the pile diameter (or width for a square pile). This test is essentially a very large CPT test, using the pile instead of the CPT probe. CRP tests may also be conducted on tension piles (the reaction piles then being loaded in compression, Figure 2.5(b)), in which case the penetration rate is reduced to 0.1–0.3mm/min as the pile will mobilise its tension capacity at much smaller displacements than in compression (Craig & Knappett, 2012).

Maintained load tests (MLT) are used for working piles. This involves applying load to the pile through the jack which is then maintained for a period of time. A series of loading stages are normally applied as detailed below:

- Load to 100% of the design (working) load, also called the design verification load (DVL) in 25% increments;
- Unload fully in 25% increments;

- Reload directly to 100% DVL, then load to 150% of the working load (also called the proof load) in 25% increments;
- Unload fully in 25% increments.

The settlement recorded under the proof load is used to verify that the SLS has been met for the pile.

The pile capacity determined from a CRP load test is an in-situ measurement of the characteristic resistance (Rk) of the tested pile at ULS. This resistance can be used to provide an alternative value of the design load by appropriate factoring

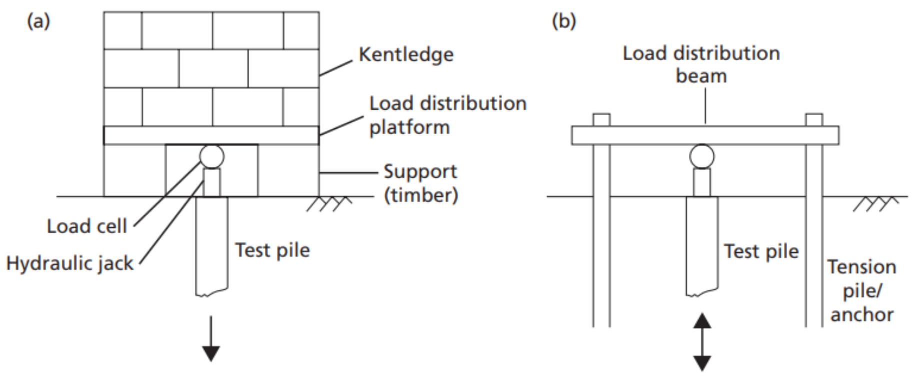


Figure 2. 5 Static load testing of piles: (a) using kentledge, (b) using reaction piles

2.4.1.2. Statnamic or rapid load test (RLT)

This is a method of pile testing which yields static results (load vs. settlement) by a dynamic approach, which makes it a combination of static and dynamic methods and hence its name **Statnamic.** In the test, a reaction mass is placed over a pressure chamber fixed to the pile top. High pressure of the gases produced in the pressure chamber by igniting a solid fuel propellant, accelerates the mass upwards with a corresponding force acting equally downwards as a load on the pile top. The force so generated is about 20 times the weight of the reaction mass. The force so applied and the corresponding pile displacement are measured directly using a load cell and laser beam system. The data so acquired is graphed as load-displacement diagrams which are comparable to the results from the conventional static load tests. The method is quick, reliable and efficient, allowing several tests to be performed in a single day. In the test a load cell measures the load and a built-in laser sensor measures the settlement, and by connecting them to a suitable laptop computer, one gets graphic results immediately. Verification tests for comparison of performance of a sample pile under static and statnamic loading methods have given rise to virtually identical load-settlement and load distribution diagrams (Kurian, 2013).

2.4.1.3. Dynamic load test (DLT)

Dynamic Pile Testing is a modern alternative to the conventional static pile load test. It is technology intensive, which makes it more user-friendly, and involves the dynamic application of load, either in the form of a strike by a small hand-held hammer ('low-strain testing' for checking pile integrity) or by a heavy weight falling on the pile ('high-strain' testing for determining load-carrying capacity). This means, in the case of a precast driven pile, the pile driving process itself serves the purpose of dynamic loading, which in other words means that the load carrying capacity (as sum of end bearing and skin resistance) can be determined in real time at every blow of the hammer throughout the pile driving process. This indicates that we can terminate the driving operation on reaching the required load carrying capacity as per design. It further shows that the hammer performance can be monitored at every blow and changes can be effected either modifying the height of fall or changing the hammer itself for more efficient operation (Likins, 1982).

In dynamic tests, an impulse loading (usually from an instrumented hammer) is applied to the top of the pile. From analysing the wave propagation through the pile, particularly the wave reflected from the base of the pile, the pile capacity can be determined (Craig & Knappett, 2012). In both the RLT and DLT procedures, the stresses applied to the pile on test are very rapid, and dynamic effects need to be taken into account when interpreting the test data (e.g. damping in the pile and soil).

2.4.2. Pile integrity testing

Integrity tests are also known as non-destructive tests. Non-destructive testing (NDT), is "a wide group of analysis techniques used... to evaluate the properties of a material, component or system without causing damage" (Wikipedia.org). This is one of the major benefits of these integrity tests, as they preserve the structural overview of the piles to be tested so there is no need for creating test samples, making the testing process time saving and less costly.

Generally, pile evaluation is accompanied by the application of one or more of several integrity tests on the piles, as detecting structural defects is very crucial in the construction process. Some of these integrity tests as specified by the American Society of Testing and Materials (ASTM), are:

- The low strain integrity test (ASTM D 5882)
- High strain dynamic testing (ASTM D 4945)
- Cross hole sonic logging (ASTM D 6760)
- Thermal integrity profiling (ASTM D 7949)

Other integrity tests that can be applied on pile structures include

- The parallel seismic test
- The ultrasonic testing
- The gamma-gamma radioactive logging

There are basically two categories of integrity tests which are either the non-intrusive or the intrusive (Amir, 2017). The non-intrusive methods require only access of the pile head thereby preparation of pile head in order to carry out these tests are necessary. On the other hand, the intrusive test require access to the pile shaft by access ducts that are installed prior to construction.

2.4.2.1. History of pile integrity testing

The trend of increased requirements for quality control/quality assurance, dimension quantification and existing condition verification on piling contracts has necessitated an accurate interpretation of data obtained from non-destructive tests (Bolarinwa et al., 2018). This statement implies that as there is an ever-rising need for quality control in the construction of pile structures, therefore the results obtained from the integrity tests applied to this process of quality control must be done with accuracy.

Before the onset of the integrity tests, one of the ways in which piles where tested for flaws was by digging around the piles in other to make external observations and analysis, done mostly for the cast in place concrete piles and drilled piles. Also, for other piles such as the precast piles, there could be arisen flaws at the level of installation therefore they would still need to be verified.

Well for this method of pile testing by excavation, besides the fact that they were ineffective, pose a danger to ultimate survival of the construction works or structure that is its being able to fulfil the specifications for which it was designed. This danger is mainly due to the fact that poor quality piles will mean poor quality foundations, and poor foundations would mean poor building construction as a whole, because if the foundation fails, then almost all the time, efforts, man power and money that were invested into the building construction, has just boiled down to a total waste.

It is for the reason of foundation failures that several projects have been abandoned and buildings demolished. Pile testing by excavation was useful but to a limited degree. Their usefulness was seen in the fact that they could also expose flaws, but only to the outer part of the pile, therefore leaving the internal integrity unchecked. Another major setback was that pile testing by excavation was limited only to the upper parts of the pile shaft.

Another way in which pile integrity testing was carried out, historically was by core drilling a method which required the usage of a core drill a device which is specifically designed to remove a cylinder of material and the material that is left inside the drill is referred to as the core. The core that has been taken from the pile is then analysed, and information about the integrity of the pile is then retrieve from the analysis. Unlike the extraction method of pile integrity testing that was limited only to a few meters down the pile head, core drilling could be carried out down to large depths, providing very useful and valuable information but just that results were limited to just a small fraction of the pile. The historical pile testing methods of excavation and core drilling are called the direct methods according to an article by Wikipedia.org on pile integrity testing.

The article as well states that the indirect methods or imaging where first developed in the early 1970's which are as follows.

- The nuclear radiation or the gamma-gamma method.
- The short wave ultrasonic acoustic method
- And the long wave sonic acoustic method.

Due to the advancement in technology and development of new and better ideas, the integrity testing of piles was birth into a new dimension, and not only were these new methods more effective in their analysis and their depiction in the quality of the piles, they were easy to carry out and less time consuming. The improvement of the integrity testing of piles was mainly driven by the fact that since most of the pile was buried under the ground, leaving only the pile head accessible, the new integrity testing methods had to be developed that could be carried out from the pile head.

This was eventually accomplished when tests like the low strain integrity test was developed. Other names of this test are the low strain dynamic test, or the sonic echo test. The low strain integrity test is probably the most popular and widely used test in the world (Cui et al., 2017). The outcome of the development and implementation of this test, was very successful because this test evaluate the entire shaft without any direct access to it. It also proved to be relatively cheap to perform and did not require a lot of time therefore making it possible for several piles to be able to be checked in a small amount of time. This test was not only able to determine the presence of flaws in the pile, but it was also capable of pin pointing the location and extend of these flaws.

Figure 2.6 shows the history of the integrity testing techniques on piles. In this figure is the demonstration of the times in which various integrity testing methods where incorporated into the testing of the integrity of piles from 1960 right up to 2010.

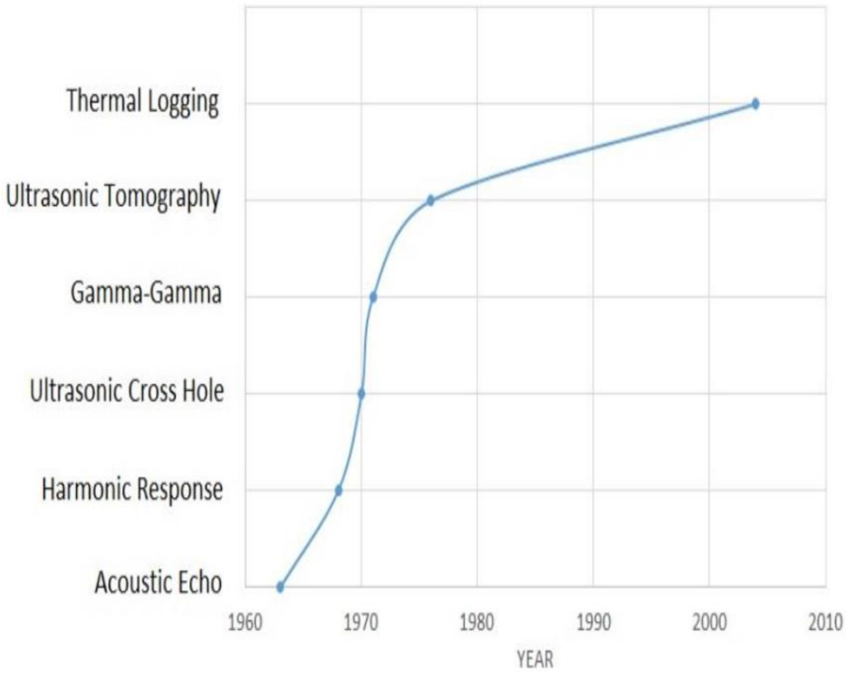


Figure 2. 6 History of integrity testing techniques (Amir, 2017)

2.4.2.2. Pile integrity testing methods

a. Low strain integrity test (LSIT)

Pile evaluation by this test is carried out on the bored pile after installation and regaining of pile strength. An accelerometer is installed on the pile head, which serves the role of registering the input which is the applied strain reflection of the produced strain from the pile toe. The strain is produced, in the form of a small shock wave, by directly hitting the pile head with a hand-held hammer hence the name “low strain”. The strain that is being produced by this hammer travels down the pile length at a velocity which is dependent on the material composition of the pile. As the acceleration of the strain in the pile known and the velocity is calculated by the integration of this acceleration or instead of an accelerometer, a velocimeter is used. Because the soil dampens the travelling waves, magnifying the wave signal with time is needed to view reflections (Garland Likins & Partner, n.d.). Other signal enhancing techniques are often required to evaluate these small signals (Likins and Rausche, 2000). Several results are recorded and then analysed using the wave equation, which is related to the behaviour of the reflected strain, in response to the structural integrity of the pile. Interpretation of results could be either in the time domain or the frequency domain.

Figure 2.7 shows a typical setup of the low strain integrity test.

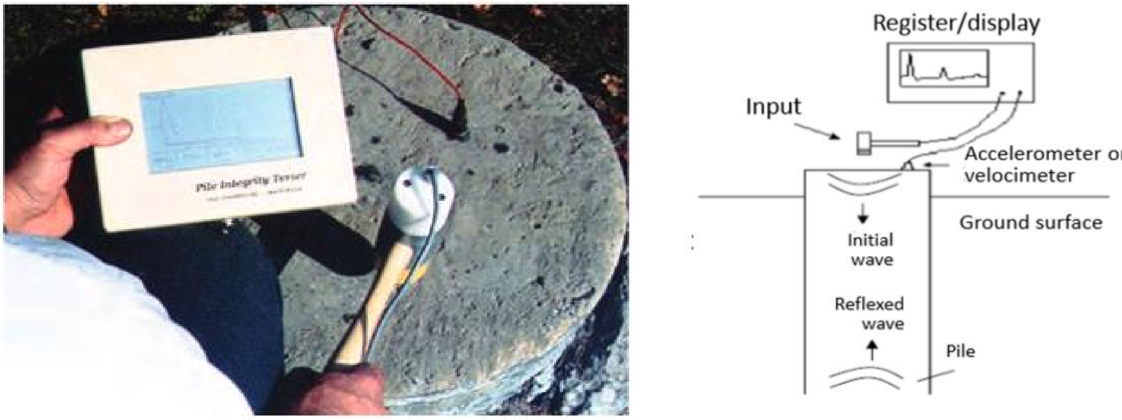


Figure 2. 7 Low strain integrity test setup (Mohamed & Likins, 2005)

The two main methods by which the Low strain integrity test is carried out are the sonic pulse echo method and the mobility test.

i. Pulse echo method (PEM)

In this method there is no measurement of the applied force, so only velocity or acceleration in being measured at the pile head. The applied strain then travels along the pile length in the form of compression waves that are reflected by the variation of impedance (F.Ceccato, 2017). That is, in areas along the pile length where there are minor defects, discontinuities, variation of pile cross sections, inclusion of materials in concrete, non- homogenous concrete and soil layering, a plot of a velocity time graph in the time domain will produce a wave motion from which can be gotten interpretations of results in relation to the pile integrity.

Figure 2.8 shows the a reflectogram, which is a signal gotten as a result of carrying out the LSIT.

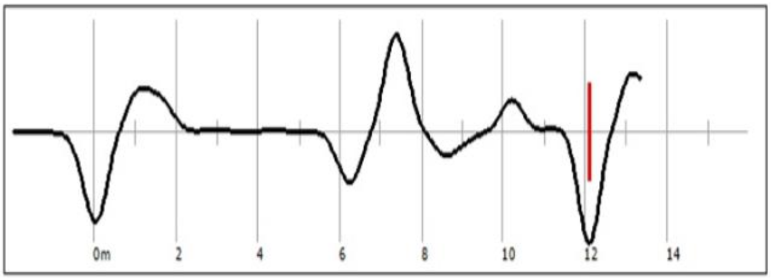


Figure 2. 8 Example of a typical reflectogram from the LSIT (Cui et al., 2017)

ii. Transient response method (TRM)

By this method, the force, produced by the hammer or exciter (any other device that could be used in place of the hammer to induce a low strain) is measured in time along with the velocity at the pile head, deriving a mobility function in the frequency domain. This generated force produces resonance peak at discontinuities that makes it possible to evaluate the mechanical properties of the pile.

Figure 2.9 shows the general setup of carrying out by the transient response method.

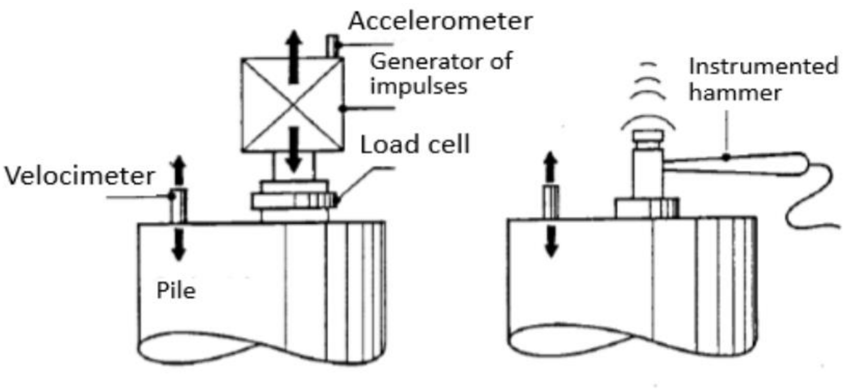


Figure 2. 9 Showing transient response measurement (F.Ceccato, 2017)

iii. Advantages of the low strain integrity test

The low strain testing has as some advantages over other integrity tests.

- The process is cheap and easy to perform (does not require much expertise of the one carrying out the test).
- There is minimal pile preparation procedure required.

- Not time consuming giving the possibility of multiple tests carried out in a short period of time.
- Since all that is required to carry out the test is access to the pile head, there is no need for it to be planned in advance.
- The depth and the quality of the existing foundation can be evaluated.
- The test is readily available to be carried out at any period of time as there is no heavy working equipment involved.

iv. Disadvantage of the low strain integrity test

Despite the numerous opportunities there are a number of setbacks with respect to the use of this method.

- There is no direct evaluation of the pile bearing capacity
- The intensity of the produced strain along the pile dampens (especially by the soil) in time, therefore it is not very agreeable with long piles (Length limitation of approximately 25 to 50 times the pile diameter)
- The accuracy in the determination of the length and the depth of the pile structure is dependent on the assumed wave speed.
- There must be accessibility of the pile head (cannot be effected over pile cap).
- The standard provides an approximation rather than an accurate result of the pile integrity.
- The results in the end could still be inconclusive in some cases.

b. Parallel seismic test

This is a test that functions on the wave reflection theory, rather than the wave transmission. In this test, an impulsive hammer is used to generate compressional and shear waves, which travel down the foundation and are refracted to the surrounding soil. The application of this force is to the exposed top of the foundation. A case bore hole is made at the foundation that is being tested in which the arrival of the produced wave is being tracked at regular intervals. The tracking of these waves is done with the help of a hydrophone, of a three-component geophone receiver in a cased borehole. From this process, the depth of the foundation can then now be determined. This is done by plotting the first arrival time, as a function of depth, and observing the depth, where a change of slope occurs. Also, by this process, the foundation depth can be determined by observing the depth where the signal amplitude of the first arrival energy is reduced by an appreciable amount.

Figure 2.10 shows the setup of the parallel seismic test.

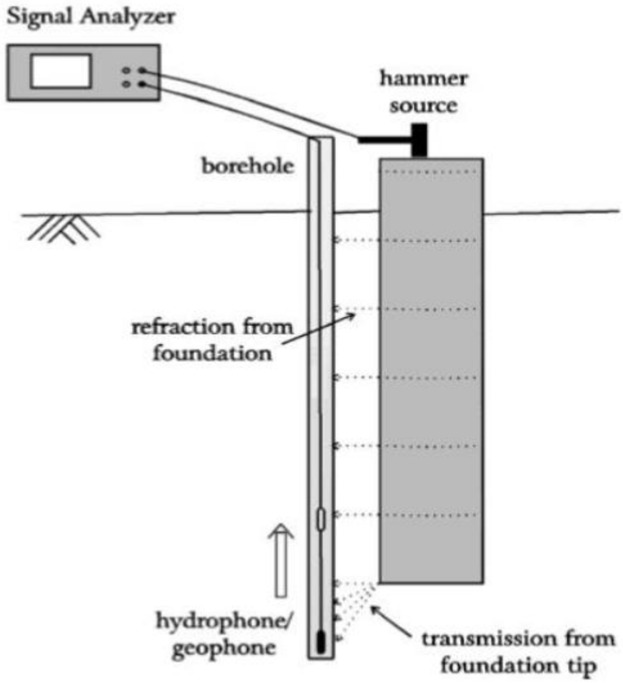


Figure 2. 10 Parallel seismic test (Geovision, 2019)

i. Advantages of the parallel seismic test

This method present some advantages as an integrity testing method over the the other pile testing methods.

- When it comes to the determination of the unknown depth, this test is much more accurate than other NDT.
- Depths are normally determined to within 5% accuracy or better.

ii. Disadvantages of the parallel seismic test

Some disadvantages presented by the parallel seismic test as an integrity test are.

- A borehole is typically needed for the parallel seismic test, which adds to the cost of the investigation.

- Interpretation of the PS data becomes more difficult as the borehole moves away from the foundation, and the uncertainty in the tip depth determination becomes even greater.

c. Ultrasonic testing

This is the testing that makes use of high frequency sound waves in order to conduct examinations and to carry out measurements. These high frequency sound waves typically range between 0.5 and 15 MHz.

In order to perform this test, a pulse echo reflection system is needed, which is composed of the pulsar, the receiver, the transducer, and a device for displaying data. The pulsar here is used as an electronic device which is capable of producing high voltage electrical pulses. The pulsar drives the transducer to produce high frequency ultrasonic energy. This sound energy is then propagated through the material medium that is to be tested, and travels through the material

in the form of waves. Deformities such as cracks, that occur in the testing structure, which in this case could be a pile, could be detected, when this travelling sound wave comes across it. This is because part of the wave energy would be reflected at the flaw surface, and the other part would continue to travel down the pile shaft. The transducer is then used now to transform this reflected wave into an electrical signal for data analysis, for this converted signal is then displayed on the screen. Since the velocity of the wave is known forehand, the distance and the time the wave travelled are related.

Figure 2.11 gives a depiction of the setup of the ultrasonic test.

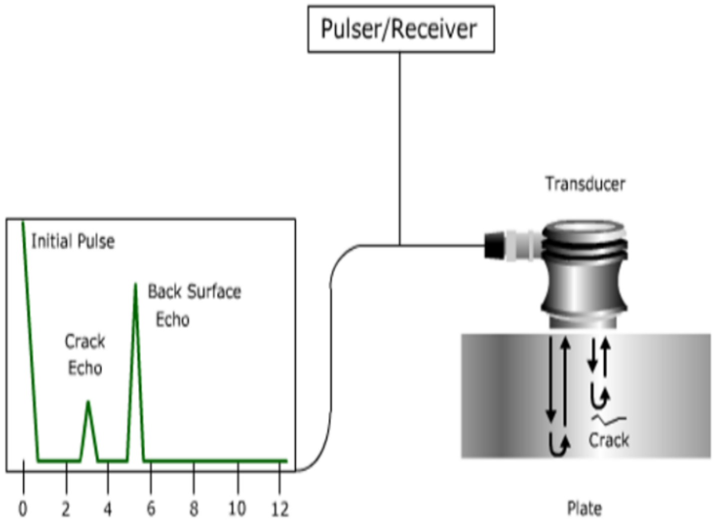


Figure 2. 11 The ultrasonic test (A. Hijazi, 2019)

i. Advantages of the ultrasonic testing

Some advantages of the ultrasonic test as an integrity test for testing piles are.

- Both the surface and the sub surface discontinuities can be detected with sensitivity.
- Compared to other NDT methods it can penetrate deeper into the tested structure for the detection of flaws.
- When this technique is being applied, only a single sided access is needed.
- There is high accuracy with the ultrasonic testing in the determination of the position of the flaw, and also in determining both its size and its shape.
- This test is rapid to carry out and it provides instantaneous results.
- The automated systems can be used to produce images that are detailed.
- This test is a non-hazardous test towards the personnel that is responsible for carrying it out.
- The equipment for carrying out the test is highly portable as well as automated.

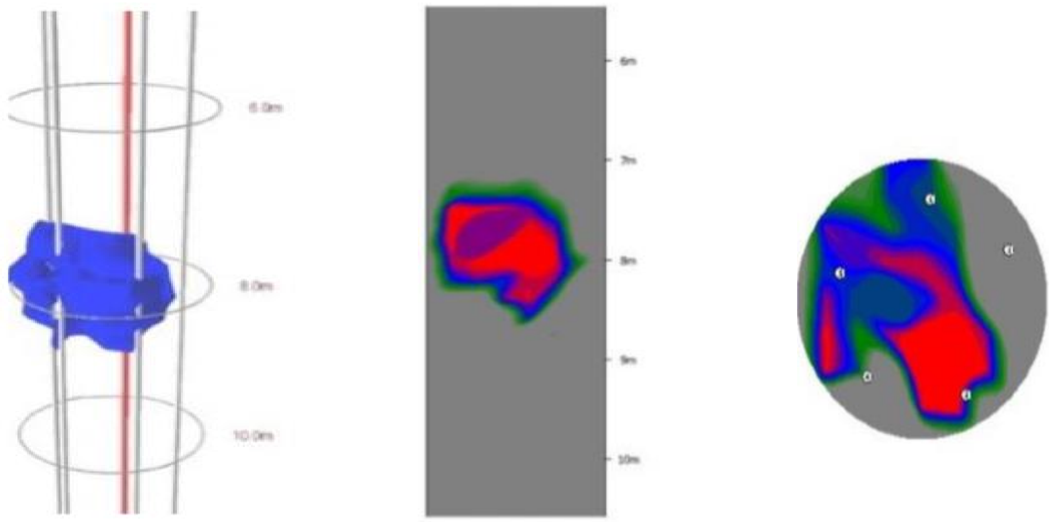
ii. Disadvantages of the ultrasonic testing

Some of the disadvantages of the ultrasonic test as an integrity test for testing piles are.

- There must be access to the surface in order for the sound transmission process to be possible.

- Expenses for the acquisition of skill and training is more burdensome as compared to other methods.
- The test materials that have a lot of irregularities such as roughness, irregularities in shape, very thin and not homogenous makes carrying out this test difficult.

Figure 2.12 is a depiction of the results the 3D ultrasonic testing, indicating the vertical cross section and the horizontal cross section.



a b c

Figure 2. 12 Result of three dimensional ultrasonic testing: (a) 3D interactive model (b) vertical cross section (c) horizontal cross section

d. Gamma-gamma (radioactive) logging

This is a non-destructive testing method that is used for the quality assurance of piles and other concrete structures. This test is generally used to determine the homogeneity of the concrete of a pile, by using a cylindrical probe that contains a weak radioactive source. The radioactive source type is usually Caesium 137 (Amir, 2017), and a photon counter that is being separated by a lead shield. The gamma radiations that are emitted by this cylindrical probe, is in all directions in the pile, where some of the photons are absorbed by the concrete, and the rest are scattered. The backscattered photons are that which are recorded by the counter and analysed. Low concrete densities would produce high photon counts and high concrete densities would produce rather low photon counts. This is mainly because the higher the density of the concrete, the more capable it is to absorb more photons. As the density of concrete can also vary with time, this method of testing can be applied soon after casting. Analysis is done by changing the photon count readings to density, after proper calibration of the instrument responsible.

Figure 2.13 shows an example of the gamma-gamma geophysical log, showing the edge measurements of density found in the iron ore exploration.

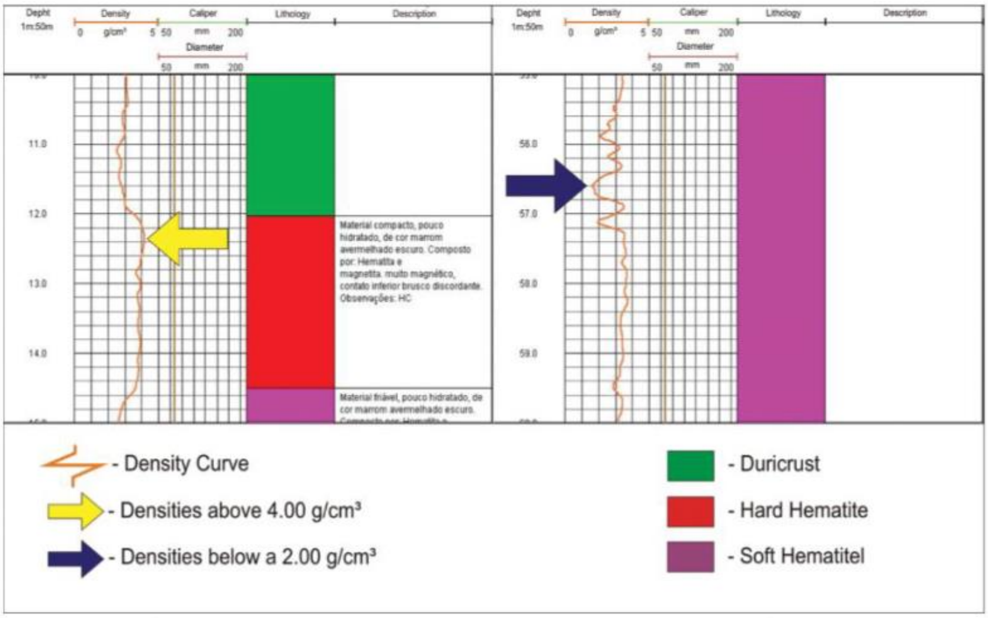


Figure 2. 13 Example of gamma-gamma geophysical log, showing edge measurements of density (Pereira et al, 2016)

i. Advantages of gamma-gamma (radioactive) logging

An advantage of the gamma-gamma logging as an integrity test for testing pile structures is that it can work through the steel and cement walls of cased bored piles.

ii. Disadvantages of gamma-gamma (radioactive) logging

This method has as disadvantages

- The readings are strongly dependent on the proximity of rebars.

- The typical range of the probe is less than 100mm.

e. Cross hole sonic logging (CSL)

This is a non-destructive testing method that is used to verify the structural integrity of drilled shafts and other concrete piles by evaluating the speed of propagation of sound waves in concrete. Figure 2.14 depicts the setup of the process of carrying out the cross sonic logging test.

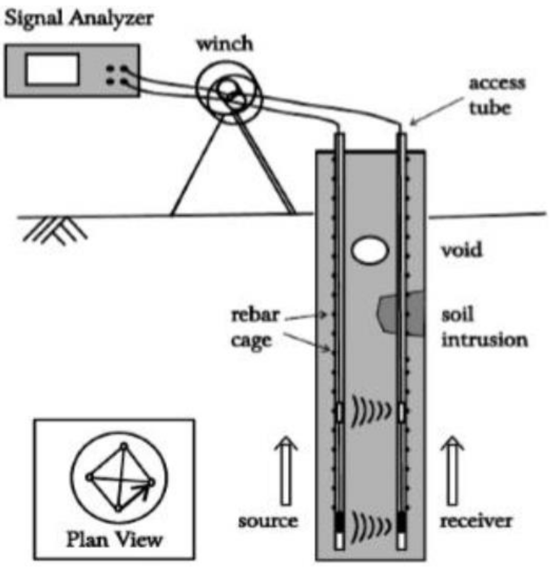


Figure 2. 14 Cross hole sonic logging setup (F. Ceccato, 2017)

"The Cross hole Sonic Logging (CSL) system is designed for Quality Assurance (QA) testing of newly placed critical drilled shaft foundations and auger cast piles, but can also be applied to slurry walls, mat foundations, and mass concrete pours."(Aci, n.d.)

This is because the sound wave speed in the concrete, is as intrinsic property, as it is affected by the properties and the geometry of the concrete material of which it is being propagated.

The sound wave signal that is transferred through the shaft is made possible by 2 parallel water filled tubes called the access tubes.

These access tubes could either be made of mild steel for cross hole testing and required to be made of polyenyl chloride (PVC) or equivalent for single hole testing (Logging et al., 2015), and are filled with water. They are installed into the pile shaft by being tied to the rebar cage in the interior, and before the rebar cage is then inserted into the drilled hole of the pile.

The number of accessed tubes that are being installed is in relation to the pile diameter that is one duct for every 0.25 to 0.30 m of deep foundation element diameter, spaced equally around the circumference (Logging et al., 2015).

After the rebar caged has been lowered into the drilled hole, the concrete is the placed into the hole and given a curing period of about 3 to 7 days depending on the concrete strength and the diameter of the shaft as larger diameter shafts require longer curing periods.

After the curing two probes, to carry out the testing procedure are required. They are the transmitter probe, which transmits the wave signal in the form of an ultrasonic pulse, though the concrete pile shaft and the receiver probe, which receives the wave signal from the transmitter probe.

The transmitter probe is then lowered into one of the pre-installed access ducts right down to the bottom, taking note of its depth, and the receiver probe into another, where this sonic wave pulse is generated by the receiver probe.

The testing procedure is carried out for various depths of the transmitter probe in the access duct, preferable every 50mm as the transmitter probe is pulled up the access duct of the pile, in order to obtain results for the analysis of the entire pile shaft for defects. This means that the signals that would be captured by the receiver probe would be every 50mm.

Figure 2.15 shows the depiction of the signals collected after carrying out the cross sonic logging test.

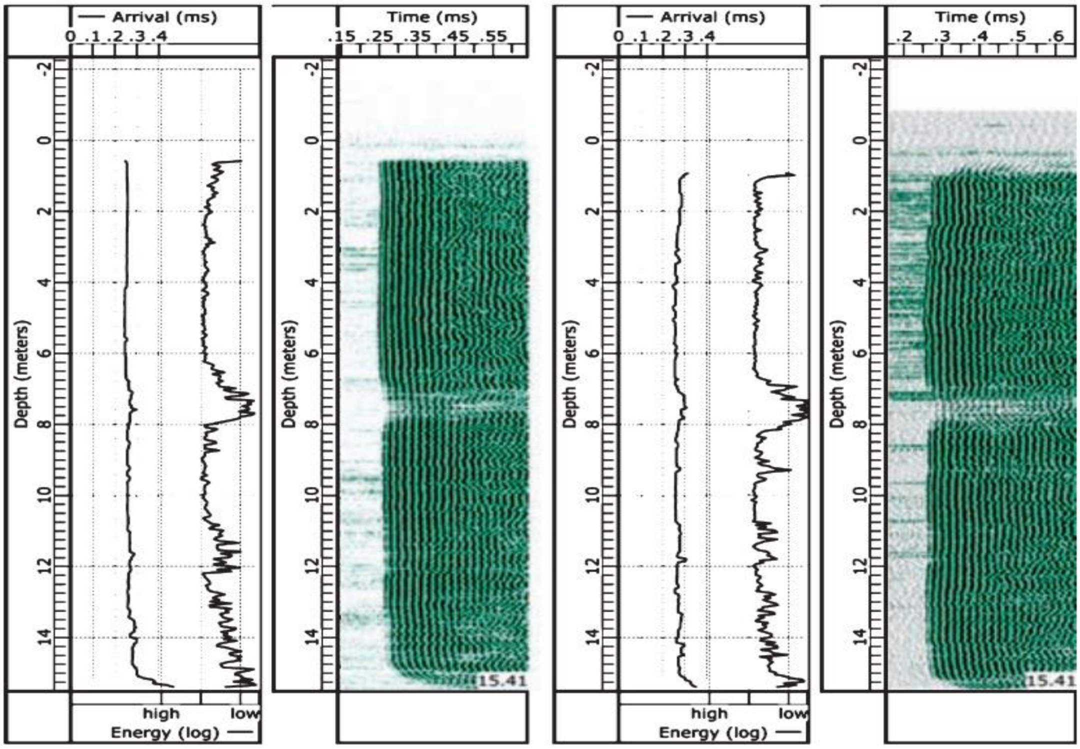


Figure 2. 15 Depiction of the signals collected for data analysis showing defect (White et al., 2008)

The testing procedure is then repeated in different access duct combinations in order to obtain more detailed and accurate results for data analysis.

From these results, the wave speed in the concrete pile shaft can be calculated, from the First Arrival Time (FAT), since the distance between the two pile ducts is already known.

From the analysis of the FAT, the cross sections in the pile shaft for which there are defects such as soil inclusions, contaminated concrete or poor concrete quality can be detected as the would be a delay in the FAT of the sonic pulse along the section where the defect is found.

i. Advantages of the cross hole sonic logging

Some of the advantages of the cross sonic logging test as an integrity test are.

- Relative simple interpretation when a major defect is present
- The test is not affected by pile length (no length limits).
- Can determine the depth of an anomaly with reasonable accuracy
- Can estimate horizontal extent of a defect if enough access tubes are present

ii. Disadvantages of the cross hole sonic logging

The following are some of the disadvantages in carrying out the cross sonic logging test as an integrity test.

- The test needs to be planned in advanced as it requires the installation of the access tubes before the casting of the concrete.
- Only concrete between access tubes can be assessed
- Evaluation often requires experience and engineering judgment when results are complicated or not outwardly conclusive

f. Thermal integrity profiling (TIP)

This is an integrity testing method that utilises the emission of heat, when cement reacts with water during the curing process, called the heat of hydration. It is used to check cement integrity and quality of drilled shafts, augered cast in place and concrete foundations (Wikipedia contributors, 2019). The heat of hydration is measured by the aid of thermal probes that can be inserted into the pile through access tubes. Wires to which thermal sensors have been attached could be another alternative as these wires are attached to the reinforcing cage prior to casting.

Figure 2.16 gives a depiction of the thermal integrity profiling test setup.

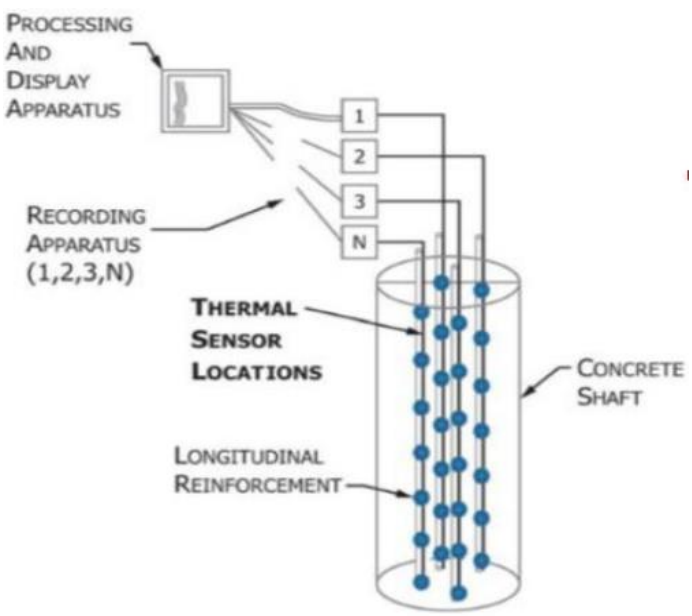


Figure 2. 16 Depiction of the thermal integrity profiling (F. Ceccato, 2017)

After the casting, the cement temperatures produced by curing concrete, is monitored until the concrete curing process is over or the monitoring can begin just before the time that peak curing temperatures are attained.

The temperatures are measured along the entire pile cross section at various depths of the pile and these temperatures are then analysed for defects.

If at a certain depth the relative temperature should be cooler relative to the average temperature of that depth, then it could be that there is a defect in the area, such as a void, low density or poor-quality concrete, or inclusion of foreign materials such as soils or wood.

On the other hand, if the average temperature at a certain depth are analysed and compared to the average temperatures of other depths, and seen it is relatively lower, then there could be a defect such as already stated. If the relative temperature emitted by the concrete cross section is higher, then perhaps a pile deformity such as a bulge is expected. Figure 2.17 shows a depiction of a defect collected by the TIP.

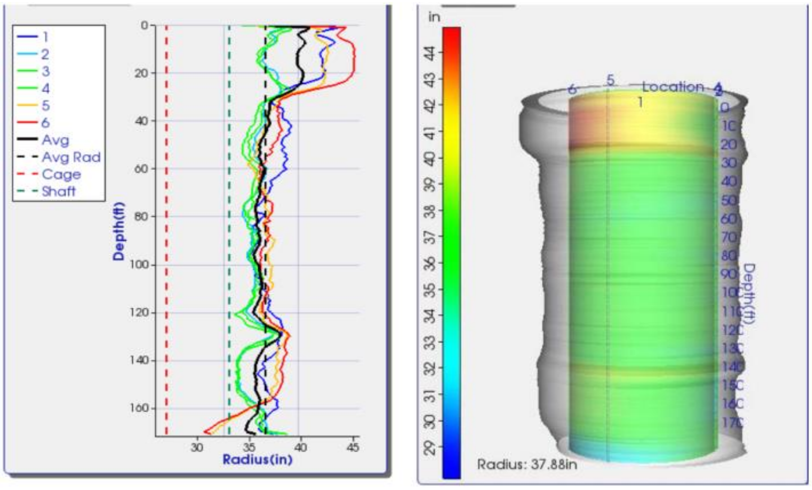


Figure 2. 17 Radius versus depth (left) and 3D image right from thermal measurements (Likins et al., 2015)

By the TIP testing method, the effective area of concrete can be found as well as the alignment of the concrete cage.

For a cage that is not in alignment along the pile shaft, the side which is closer to the centre would absorb more heat of hydration so therefore will be depicted as normal, and that closer to the soil would be cooler.

i. Advantages of thermal integrity profiling

Some of the advantages of the thermal integrity profiling test as an integrity test are.

- The test does not depend on the length of the pile; therefore, long piles would not affect it in any way
- The test is able to test the alignment of the reinforcement cage, as opposed to other integrity tests which cannot.
- The test can be used to verify the concrete cover of the pile.

ii. Disadvantages of thermal integrity profiling

The following are some of the disadvantages of the thermal integrity profiling test as an integrity test.

- The test must be planned in advance.
- Must be performed during concrete hardening.

g. High strain dynamic test

This is an integrity test coded by the ASTM D4945-12 for the testing of piles of deep foundations.

A depiction of the test apparatus of the high strain dynamic test is as shown in the figure,,,

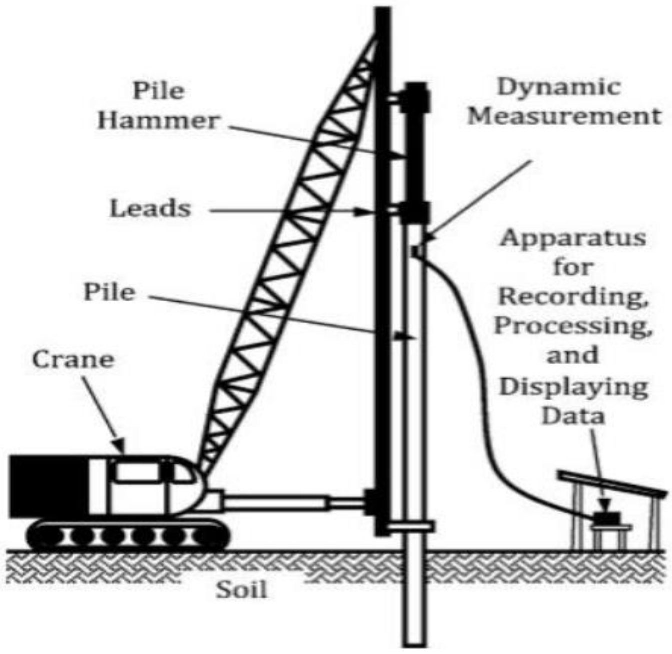


Figure 2. 18 The high strain dynamic testing (ASTM D 4945, 2008).

In this test, a device to produce an impact force of sufficient energy such as a drop or driving hammer is used. The impact of this force produced could go up to the order of magnitude or even beyond the ultimate bearing capacity of the pile.

The impact force from the hammer is applied at the pile head in an axial manner in a concentric alignment and this force is to provide sufficient energy for the pile to be able to penetrate the soil strata as the hammer strikes the pile head.

The impact then produces a compressive wave as a short duration shock pulse that is able to travel down the pile shaft to the pile toe, where it is reflected as it encounters the soil resistance at the pile toe.

It then travels back up to the pile shaft where it is registered and analysed, from which the total resistance of the soil can be gotten.

Since by this test the pile is driven, energy delivered to the pile, as well as compressive and tension stresses can be measured from the data analysis.

i. Advantages of high strain dynamic test

An advantage of the high strain dynamic test as an integrity test could be stated as. Since the impact force is high, the wave produced, as well as the reflections are of a higher intensity therefore stronger signals are produced which make analysis easier.

ii. Disadvantages of high strain dynamic test

Some disadvantages of the high strain dynamic test as an integrity test are.

- After a set period, the hammer is not able to mobilise the pile after a set period of time therefore a heavier hammer has to be used.
- The testing procedure can cause damage to the pile.

Table is a depiction of the most common pile integrity tests, showing the time required between the shaft testing and casting, showing if the material should be cast in the shaft, showing the evaluation of the cross section, showing the advantages of the particular integrity test, together with its limitations.

Table 2. 1 Overview of commonly used integrity testing methods (F. Ceccato, 2017)

Method	ASTM standard	Time Required Between Shaft Testing and casting	Requires Material to be cast in shaft	Cross section evaluated	Advantages	limitations
Pulse echo (PEM)	D 5882	No sooner than 7 days casting or after the concrete achieves atleast 75% of its design strength	No	Only major cross-sectional changes	Quick. Economical.	Depth often limited to 30 or 40 shaft diameters
Transient Response (TRM)	D 5882	No sooner than 7 days casting or after the concrete achieves atleast 75% of its design strength	No	Only major cross-sectional changes	Quick Economical	Depth often limited to 30 or 40 shaft diameters
Cross hole sonic logging (CSL)	D 6760	No sooner than 3 to 7 days. Larger diameter shafts closer to 7 days	Yes, one steel or PVC acess tube per 305mm of shaft diameter	Cross sectional area delineated by perimeter of acess tube	Widely available integrity test. Depth limited only to probe cable length	Sensitive to acess tube/concrete bond. Fine horizontal cracks unlikely to be detected. Depth limited only by cable length
Gamma-gamma logging	None	No time restriction. Test can be performed immediately after concrete placement	Yes, one PVC access tube per 305mm of shaft diameter	Cross sectional area extending 102mm from centre to access tube	Concrete cover evaluated in vicinity of access tubes	Storage and of damma source. Depth limited only to probe cable length
Thermal integrity profiling (TIP)	D 7949	12 to 48 hours depending on shaft diameter	Yes, one thermal wire or one access tube (thermal probe method) per 305mm of shaft diameter	Full shaft cross sectional area	Accesses cage alignment. Evaluates concrete cover	Fine horizontal cracks unlikely to be detected. Depth limited only by thermal wire or probe cable length

2.4.2.3. Pile integrity test according to Eurocode

According to BS EN 1997-1, if site observations or inspection of records reveal uncertainties about the quality of installed piles, investigations shall be carried out to determine their condition and if remedial measures are necessary. These investigations shall include either performing a static pile load or integrity test, installing a new pile or, in the case of a displacement pile, re-driving the pile, in combination with ground tests adjoining the suspect pile.

Dynamic low strain integrity tests may be used for a global evaluation of piles that might have severe defects or that may have caused a serious loss of strength in the soil during construction. Defects such as insufficient quality of concrete and thickness of concrete cover, both of which can affect the long term performance of a pile, often cannot be found by dynamic tests and other tests, such as sonic tests, vibration tests or coring, may be needed in supervising the execution.

The test load applied to working piles shall be at least equal to the design load for the foundation. With the practical load up to 1.5 or 2 times the design load.

Qlim is the value measured at a total settlement of:

• 0.10D for small and medium (d<80 cm) diameter pile

• 0.05D for large diameter pile (d≥80 cm)

2.4.2.4. Pile integrity test according to the Italian norm (NTC 2018)

In cases where the quality of the piles depends to a significant extend on the execution procedures and the geotechnical characteristics of the foundation soils, integrity checks must be carried out.

The integrity check, to be carried out by direct or indirect tests of proven validity, shall cover at least 5% ≥of the foundation piles with a minimum of 2 piles

In case of groups of large diameter piles (d ≥ 80 cm), the integrity check must be carried out on all piles in each group if there are 4 or fewer piles in the group.

Static load tests must be executed on foundation piles to verify the behaviour of piles under the design actions, except for piles with a prevalently horizontal load. The tests have to reach loads equal to 1.5QSLS.

The maximum load applied in the test may be reduced to 1.2QSLS if the tested piles are instrumented for the determination of transfer curves. The number and position of tested piles must be chosen in relation to the importance of the structure and the heterogeneity of soil stratigraphy.

some static tests may be substituted with dynamic tests, only is the dynamic tests have been calibrated with design static tests executed on trial piles and non-destructive quality tests are executed on at least 50% of piles. In any case, at least one static test must be carried out.

For foundation in very difficult conditions (e.g. offshore structures in deep sea) specific codes of demonstrated validity may be applied.

Conclusion

The purpose of this chapter was to review the quality control of pile foundation which involved the design of pile, understand the different failures and possible remedies of pile foundations indicating a review of a few cases of failure due to lack of quality control testing, the interaction between pile and soil which has an influence on the behaviour on the pile, and the different methods of pile testing with both their applicability and limitations.

However, to better represent the context of study, a practical case was selected and the method to accomplish this study was presented in the next chapter.

CHAPTER 3. METHODOLOGY

Introduction

This chapter presents the different methods and steps involved in the establishment of the objectives of this work. In other word it was a question of describing the different constitutive elements of the research. The presentation starts with the description of the methods used to obtain the different data and parameters needed for the study, the norms used, the analytical analysis steps, the pile installation sequence used, the pile testing methodology and the numerical analyses of pile in the case.

3.1. General site recognition

The site recognition was done through a documentary research. This is aimed at providing the geographical location of the project area and then presenting it on a map, its relief, the climatic conditions presenting the temperatures and precipitations, the hydrology and hydrogeology, vegetation, the geological and geotechnical context and on the other hand the socioeconomic parameters and activities carried out in the region.

3.2. Site visit

The site visit consists essentially of the physical description and observation of the study area. Indeed, with the piles already constructed, the consultation and use of available project documents (reports), as well as documented or academic reports related to our study area for more information of interest.

3.3. Data collection

The main data collected for the study consisted mainly of the geometric data and the geotechnical data.

3.3.1. Geometrical data

The geometric data of the project consisted mainly of the 2D plan drawings obtain from project documents in which the information such as the distribution of the different piles.

3.3.2. Geotechnical data

In geotechnics there exists a wide variety of test that can be perform on a soil, these tests can be classified according to the place of realisation. There are two (2) major groups, in-situ tests (realised on site) and laboratory tests (realised in the laboratory). In this study, geotechnical data was extracted from in-situ test results done on the site and documentary research from which the soil stratigraphy was reconstructed and provided some parameters necessary for the study as expressed in Table 3.1

Table 3. 1 Soil parameters

Parameters	Symbol
Material model	Type
Loading	Conditions
Wet soils unit weight	γ_{wet}(kN/m^3)
Dry soils unit weight	γ_{dry}(kN/m^3)
Young´s modulus	E(kN/m^2)
Poisson´s ratio	N
Cohesion	C’
Frictional angle	Φ
Dilatance angle	Ψ

3.4. Pile design method

In this part a theoretical design of a pile was made considering the project conditions that is using all the information obtained concerning the site. The design was made using the limit state design method and the procedure of the design process elaborated.

3.4.1. Design codes and standards

The norms that were used for the design of elements are the Eurocode 0, basis of structural design, Eurocode 1, actions on structures, Eurocode 2, design of concrete structures and Eurocode 7, geotechnical design. These European standards define the loads and the combination of loads for the design.

3.4.1.1. Loads

Loads in general can be divided into permanent and variable loads. Permanent loads are those constituted by the self-weight of structural and non-structural elements. The weight of the structural elements obtained by multiplying the specific weight of concrete by the section of the elements. Variable loads are those arising from occupancy or external actions unto the structure such as earthquakes.

The loading of our structure were extracted from both the geotechnical report and the Eurocode 1.

3.4.1.2. Design approach

To take account of the special features of soil, and also to accommodate the different design traditions and views on how partial factors should be applied in geotechnical design, the following three design approaches (DAs) have been introduced

- DA1 with partial factors applied in separate combinations either to just the actions (DA1.C1) or to the material properties and the variable actions (DA1.C2)

$$DA1.C1: A_1 + M_1 + R_1 \qquad 3.1$$

$$DA1.C2: A_2 + M_2 + R_4 \qquad 3.2$$

- DA2 with partial factors applied to the resistances and to the actions or action effects

$$DA2: A_1 + M_1 + R_2 \qquad 3.3$$

- DA3 with partial factors applied to both the actions and the material properties.

$$DA3: A_2 + M_2 + R_3 \qquad 3.4$$

With, A: partial coefficients of the actions,

M: partial coefficients of the material parameters

R: partial coefficients of the resistance.

3.4.2. Geotechnical design of a pile

The pile in this case was designed under compressive loads and this was done by calculating the bearing resistance (ULS) and the settlement (SLS) of the pile and verifying the limit conditions (which are $\sum Q \leq R(X)$ for ULS and $E_A < C_A$ for SLS).

3.4.2.1. Bearing capacity

The bearing capacity of a pile depends on type, size, and length of pile, type of soil, and method of installation. Static analysis based on soil parameters was used to estimate the ultimate load carrying capacity of pile in this part. The maximum axial load is considered to be composed of the contributions of the tip base resistance and the shaft resistance.

Tip resistance obtained using the formula; in drained condition (equation.3.5),

$$Q_{bu,des} = \frac{Q_{bu}}{\gamma_{Rb}} = \frac{A_p(N_c\left(\frac{C'}{\gamma_c}\right) + \sigma'_{VL}N_q)}{\gamma_{Rb}} \qquad 3.5$$

A_p: Cross sectional area of pile base ($= \frac{\pi D_0^2}{4}$ or B_p^2 for circular or square piles respectively).

N_c,N_q: Bearing capacity coefficients

In undrained condition (equation 3.6),

$$Q_{bu,des} = \frac{A_p\left[S_c N_c\left(\frac{C_u}{\gamma_{cu}}\right) + \sigma_q\right]}{\gamma_{Rb}} \qquad 3.6$$

σ_q: Vertical total stress at the base of the pile ($= \gamma L_p$)

C_u: undrained shear strength

S_c: Shape correction factor

Shaft resistance obtained using the formula; in drained condition (equation 3.7),

$$Q_{su,des} = \frac{Q_{su}}{\gamma_{Rs}} = \frac{\pi D \int_0^L K\sigma'_z \tan\delta'_{des}\, dz}{\gamma_{Rs}} \qquad 3.7$$

With σ'_z: Effective stress at midpoint of each soil layer involved

K: horizontal earth pressure coefficient, and is a function of the soil properties and the installation method (for drilled piles, $K = K_0 = 1 - \sin\emptyset'$)

δ'_{des}: interface friction angle ($\delta'_{des} \leq \emptyset'$)

Alternatively, we can have $\beta = Ktan\delta'$ and thus

$$Q_{su,des} = \frac{Q_{su}}{\gamma_{Rs}} = \frac{\pi D \int_0^L \beta\sigma'_z dz}{\gamma_{Rs}} \qquad 3.8$$

Where $\beta = 0.52\left(\frac{C_u}{\sigma'_{V0}}\right) + 0.11$

In undrained condition (equation 3.9),

$$Q_{su,des} = \frac{A_s\alpha\left(\frac{C_u}{\gamma_{cu}}\right)}{\gamma_{Rs}} \qquad 3.9$$

With α: adhesion factor (Values given in the annex)

All the partial safety factors are given in tables of the annex and are dependent on the design approach considered for the design.

The design capacity of the pile $R = Q_{bu,des} + Q_{su,des}$ is then compared with the factored actions on the pile which are the applied load Q and the self-weight of the pile W_p =25 x A_p x L_p. For ULS to be satisfied to EC, $Q + W_p < R$.

3.4.2.2. Settlement

Verification of SLS involves ensuring that the settlement of the pile under the applied action will not adversely affect the supported structure. There are several methods to estimate the settlement of a pile both analytically and numerically. In this part the analytical technique used is the Randolph and Wroth method from which the settlement of the pile will be estimated and then compared with the limit settlement.

This method considers the soil to respond in a linear elastic way until failure and the pile to be axially rigid in comparison. The procedure involves computing the stiffness of the soil layers, using the soil stiffness to compute both the shaft stiffness and the base stiffness of the pile and with the sum of the stiffness, determine the settlement of the pile.

Soil stiffness (shear modulus) obtained from the equation;

$$E = 2G(1 + v) \qquad 3.10$$

Where E is the young's modulus, G the shear modulus and ν the Poisson's ratio of soil layer.

The shaft stiffness is given by equation 3.11

$$K_{si} = \frac{2\pi L_{si}\overline{G_{s\iota}}}{\ln\left(\frac{2r_m}{D}\right)} \qquad 3.11$$

Where K_{si} is the shaft stiffness of a particular layer, the value of r_m usually approximated by L_p that is the pile length, L_{si} the embedded length of pile in a given soil layer and $\overline{G_{s\iota}}$ the average shear modulus of soil layer of length L_{si}.

The base stiffness is given by equation 3.12

$$K_{bi} = 2.00\left(\frac{DG}{1 - v}\right) \qquad 3.12$$

Where G is the shear modulus of soil at the base of the pile and D the pile diameter.

The overall settlement at the top of the rigid pile is

$$S_r = \frac{Q}{K_{bi} + K_{si}} \qquad 3.13$$

With Q being the applied load at the top of the pile.

For SLS to be satisfied, the value of S_r obtained is verified to ensure it is less than the limit settlement of the structure.

3.4.3. Structural design of a pile

In general, the soil governs the design and it is necessary to provide only minimum steel in the pile; hence, even mild steel bars can be used. When a pile is wholly embedded in soil having an undrained shear strength greater than 0.01 N/mm2, it is allowed to be designed as a short column and buckling verification can be neglected.

3.4.3.1. Longitudinal reinforcement in bored pile

Section 9.8.5 of Eurocode 1992-1-1:2004 deals with the detailing reinforcement of bored piles. It states that pile with diameter less than 600 mm should be provided with minimum reinforcement $A_{s,min}$ distributed along the periphery of the section. Table 2.1 gives the minimum reinforcement with respect to the pile section area.

Table 3. 2 Minimum reinforcement in pile section (BS-EN 1992 - 1-1_E 2004)

Pile cross section Ac	Minimum area of longitudinal reinforcement As, min
$Ac \leq 0.5\ m^2$	$As \geq 0.005Ac$
$0.5\ m^2 \leq Ac \leq 1\ m^2$	$As \geq 25\ cm^2$
$Ac > 1\ m^2$	$As \geq 0.0025Ac$

The minimum diameter for longitudinal bars is 16 mm, and we need at least 6 bars with a spacing not greater than 200 mm.

The design compressive resistance $R_{c,d}$ of reinforced length of cast in place pile is given by equation 3.14:

$$R_{c,d} = A_c f_{cd} + A_s f_{yd} \tag{3.14}$$

Where:

Ac: is the pile raft area

f_{cd}: is the design compressive resistance of concrete

As: is the longitudinal steel reinforcement

f_{yd}: is the design yielding resistance of steel reinforcement

3.4.3.2. Transversal reinforcement

The transverse reinforcement is calculated according to Eurocode 2 and the minimum diameter is 6 mm or one quarter of the longitudinal reinforcement. The minimum clear distance of transverse bars shall not be less than the clear distance as set out for the main reinforcement.

3.5. Pile installation

The installation process plays a vital role to the integrity of the pile and thus its performance, so it was of importance to look at the method used for the installation of the pile.

The bored piles construction entails two main steps after the setting out and stacking of the pile location done by a topographer with the help of a topographic instrument (Total station), the drilling phase (Demolition - Removal - Stabilisation) and the construction phase (Reinforcement cage - Casting - Curing).

3.5.1. Drilling phase

After identifying the pile location point (pile axis), the piling rig is aligned and verified to be perpendicular to the pile location point by the site assistants (verification was done with the use of a spirit level). Outlet casing was introduced (surface ground was not very stable) before starting with the drilling process. The drilling in this case was done using a drilling bucket connected to a Kelly bar and then to the drilling rig. The excavation was done with the help of a drilling fluid (bentonite) being poured into the borehole to stabilise the walls of the borehole from collapsing while excavation continues. The drilling fluid used was produced on site using a plant to ensure constant supply of the fluid during excavation. After excavation, the fluid is pumped from the bottom of the borehole back to the plant for recycling (de-sanding).

3.5.2. Construction phase

This phase consists of inserting the reinforcement cage after the excavation, casting of concrete and curing.

3.5.2.1. Inserting the reinforcement cage

The steel reinforcement cage, after assembly was inserted in the borehole using a service crane. As it is being lowered, concrete or plastic spacers are applied to the outside of the cage to ensure the designed concrete cover.

In order to guarantee sufficient concrete cover at the bottom of the pile, the cage was supported and suspended 15-20 cm from the bottom of the hole.

3.5.2.2. Casting the concrete

Once the cage has been inserted, the borehole is filled with concrete. To do this, a string of steel pipes (Tremie pipes) with an internal diameter of no less than 250 mm made up of 2 or 3 metre long sections which are connected to each other was lowered down the centre of the shaft until they reached the bottom of the hole. A funnel was placed at the top of the string and the concrete was poured into it. The concrete flows down the pipes and, when it reaches the bottom, it begins to fill the hole, rising back up. Thanks to the considerable difference in density between the two fluids, the slurry does not mix with the concrete but was forced up towards the surface where it was collected in special slurry pits, ready to be used again. As the concrete rises inside the borehole, the string was shortened to ensure that no more than 3-4 m of piping was immerged in the wet concrete at any one time. Once the concrete had reached the pre-established level, the pouring stops and the strings are removed completely.

3.5.2.3. Curing

Once the curing time of concrete has elapsed, an excavation is made around the foundation pile.

Pile heads are trimmed to leave the reinforcement bars only.

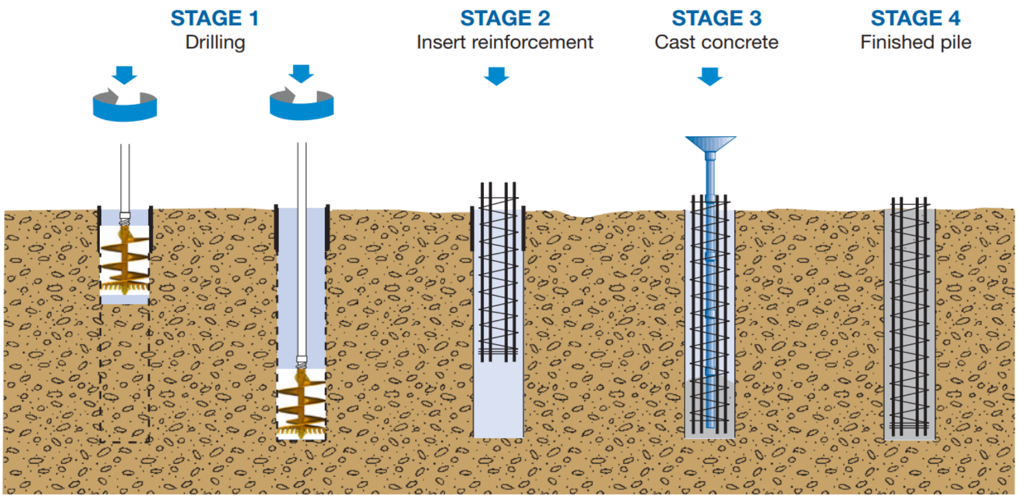


Figure 3. 1 Pile installation sequence (Soilmec, n.d)

3.6. LSIT method

This section gives in detail the process of the low strain integrity test method used on site. The method used was the Impedance test according to the NF P94-160-4 standard and the phase of the test witnessed was the process for data collection. The analysis and interpretation of the results was carried out later by the enterprise. The test method as described is applicable to the structural elements of deep foundations such as driven piles, augered piles and drilled shafts with the condition that these structural elements' must be receptive to low strains, regardless of its installation process.

3.6.1. Apparatus

The execution of the test required:

- A hand held hammer between 0.45 kg and 4.5 kg to produce the force (shock),
- a hand held concrete brush to sweep of concrete particles
- a trowel use to break off slight rough zones on pile head
- a stone hammer use to smoothen the pile head
- a speed sensor (accelerometer)
- Signal acquisition, display, recording and processing equipment (a portable laptop)
- an equipment for reproducing the recorded and processed signals
- a hydraulic pump for the extraction of water around pile head

Figure 3. 2 Tools used for the test

3.6.2. Operating mode

To prepare and execute the test, it was necessary to have the following information

- Identify the piles to be tested, this was done with the help of the layout plan of the piles presented in figure 4.7
- the date of casting of the different piles to be tested so as to ensure a minimum of 7 days before testing
- the geometrical characteristics of pile and method of installation
- a copy of the geotechnical report of the site for better understanding of the soil stratigraphy.

3.6.3. Pile head preparation

The preparation of the head of the pile had as aim to facilitate access to the pile head which was achieved by

- Dismantling of concrete hoops to obtain a plane surface for better testing with the help of the stone hammer or trowel
- Sweeping off of concrete debris from the pile head with the help of the brush
- Freeing the head of the pile from water which could tamper the results. This was done with the aid of a hydraulic pump

3.6.4. Testing process

After preparing the pile head, a receiver (accelerometer) is placed at the head by the help of a coupling product to ensure the reception of mechanical waves, the mechanical shock is created with the hammer perpendicularly to the pile surface. This shock gives rise to a transmitted wave and a reflected wave at the geometric and mechanical discontinuities and at the pile-ground interfaces. By means of the accelerometer the signal of the reflected wave is measured and the response is viewed on the PC which is connected to the accelerometer. The process is repeated for different points and 10 signals were registered for each pile tested.

3.6.5. Analysis of result and interpretation

According to ICE (1988), the result interpretation should be left to geotechnical engineers with thorough knowledge of wave propagation theory, soil mechanics and piling techniques. This part of the test was not witnessed, though a presentation of what was given in the report and the method according to NF P94-160-4 on how the final results could be arrived is made.

3.6.5.1 According to the report

The method combines the measurement (given by the hammer) with a mathematical processing of the signal obtained by reflection, and the evolution of the admittance as a function of the frequency is studied in order to measure the characteristic impedance. The analysis of the graph recorded on a tablet PC makes it possible to determine their depths. The homogeneity and quality of the concrete is appropriate and the defects are precisely located, including those at the point of the pile.

In homogeneous concrete, the speed of sound is constant, of the order of 4000 m/s. it drops rapidly in the presence of anomalies such as soil inclusion, cracks, segregation, etc.

3.6.5.2 Actual method by NF P94- 160-4

The distance from the point of impact to the point of reflection of the wave is calculated by analysing the signal.

An assumption of the planar wave speed in concrete is made, in the absence of any data, a default value of C = 4000 m/s is considered

The admittance v/F is calculated in the frequency domain.

A graph of v/F against frequency is plotted

Determine on the graph

- The variation of the frequency between two successive maxima or minima of the admittance of the reflected wave signal.
- The stiffness of the soil-pile system from the slope at the origin of the v/F curve in the frequency domain
- The characteristic admittance $(v/F)_c$ corresponding to the average value of successive minima and maxima of the admittance

Calculate the theoretical value of the characteristic admittance

$$(v/F) = (p_b CA)^{-1} \qquad 3.15$$

In the absence of data, the density of concrete is taken to be:

$$p_b = 2400\ kg/m^3$$

Calculate the distance of the reflection point:

$$L = 0.5\,C/\Delta f \qquad 3.16$$

3.7. Numerical method (MIDAS GTS NX)

The Finite Element (FE) software, Midas GTS NX, was chosen to study the behaviour of the pile under an axial load and to model and evaluate the effect of a defect in the pile. The process for the numerical method can be organised in the order, geometry which consist of building the model, mesh consisting of the material definition and creation of the FE mesh, analysis method whereby there is the definition of the boundary conditions loading which involved the attribution of load to the system, analysis which entails the control of the analysis to carry out and results consisting of the different forms of results possible

3.7.1. Geometry

The geometry which consist of building the model involve both the model of the soil and the pile.

3.7.1.1. Soil model

The soil was modelled as solid elements achieved with the use of the rectangle and extrusion tools.

3.7.1.2. Pile model

Piles in GTS NX can be modelled as

a. Solid element

Not very efficient in numerical modelling especially as it requires much time. It is very useful for in modelling special situations, like modelling low friction at certain portions of a pile due to reasons like cavity problems or partial collapse.

b. Embedded pile

Useful for large pile groups for instance pile supported embankment, or stabilising your slope with piles. The problem here is that no interface definition exist.

c. Beam element

The most rigorous and efficient way of modelling a pile is by using a beam element with or without interface.

In this study since the analysis involves a single pile subjected to different situations the solid element model was used which was obtained with the aid of the circle, extrusion and embed tools.

3.7.2. Material type

The material types used for the pile and soil materials was the Isotropic and the model type defined for each element. The models used are the elastic for pile and Mohr-Coulomb model for the soil layers

3.7.2.1. Elastic

A linear elastic model where the stress is directly proportional to the strain. The parameters involved are the Elastic modulus (E) and Poisson's ratio (v)

3.7.2.2. Mohr-Coulomb

The Mohr-Coulomb model is defined by an elasto-plastic behaviour. This behavioural assumption shows reliable results for general nonlinear analysis of the ground and widely used in simulating most terrain. This criterion is accurate within a limited range of confining pressure but as the range difference increases, the accuracy decreases. However, this criterion is often used because it is easy and displays considerably accurate results within the general confining pressure range. More to the elastic parameters, the internal friction angle (φ), cohesion (C’) are required.

3.7.3. Mesh

This step involves a detailed description of the behavioural characteristics and material parameters assigned to an element (ground/structure) that affects the analysis is included. The step is the most important modelling step and hence, it is necessary to understand the sections on element quality and material properties of the soil/structure. At this step we defined the material properties, pile/soil interface and generated the finite element mesh.

3.7.3.1. Pile/soil interface

The interface was generated as a plane element with the help of the interface wizard and considering some parameters from the manual with regards to the soil type around the pile shaft.

3.7.3.2. Finite element mesh

When the geometry model is fully defined the geometry has to be divided into finite elements in order to perform finite element calculations. The mesh generation process takes into account the soil stratigraphy as well all structural objects, load and boundary conditions. The hybrid mesher type was used in this study with mesh sizes of 0.2 used for the pile and 0.5 used for the soil layers.

3.7.4. Analysis method

This step at which the boundary conditions are defined, the condition used were the change property condition and the support condition.

3.7.4.1. Change property

A boundary condition aimed at changing a particular material to another during the construction stage analysis definition, used to signify replacement of soil with concrete during construction of pile.

3.7.4.2. Support

The support condition of the overall system was generated using the automatic condition mode.

3.7.5. Loading

The loading done on the system were of two nature, a self-weight attributed to the overall system and a surface (distributed) load acting on pile head.

3.7.6. Analysis

For the analysis to be carried out there was a definition of a construction stage set and then the analysis type before running the analysis. A stress analysis was used to define the construction stages which were the initial condition, pile installation and loading stages for the study. The definition of the water level was done during the definition of each step. The analysis type (construction stage) was defined from and the analysis launched by using the ¨perform tool¨.

Details of the different construction stages used can be given as,

3.7.6.1. Initial condition

In this stage which indicates the original state of the soil before the construction of the pile involved the activation of;

- Pile
- Different soil layers
- Rigid links
- Ground support
- Boundary condition for change of property
- Self-weight

3.7.6.2. Pile installation

This stage indicates the phase of construction of the pile and involved the deactivation of

- Rigid links
- Boundary condition for change of property

And the activation of the pile interfaces

3.7.6.3. Loading

Which indicates the application of the structures load carried by this pile, the process include the activation of the pressure load.

After the definition of each construction stage together with the water level, each stage is saved.

3.7.7. Results

After the analysis converges, the results of the analysis can be viewed in either a graphical form or through tables. Results such as displacements, solid stresses, solid strains, reactions can be accessed for each of the different construction staged defined before the analysis.

Conclusion

The aim of this chapter was to establish the procedures to follow in order to carry out the study. Firstly, a recognition of the site was done in order to assess the geographical location, its relief, the climatic conditions, the hydrology, and the geology. This is followed by a field trip which was essential mainly for the physical observations of the study area and a witness of the pile installation process and testing procedure. Afterwards, geometric and geotechnical data were collected to have the necessary parameters for the study. Then, an analytical design of the pile was done and as well the presentation of the software that was used for the study was done. Finally, the simulation procedure for the study was discussed in the last part. From these steps, the next chapter present the results obtained and their interpretation.

CHAPTER 4. RESULTS PRESENTATION AND INTERPRETATION

Introduction

In this part, the results are presented and interpreted as obtained according to the methodology adopted in chapter 3.The first section deals with the general presentation of the site focusing on its geographical location, climate, relief, vegetation, geology and hydrology. The second part focuses on the presentation of the project and the description of the study area under consideration, this is then followed by the presentation of the data collected. After which the results for the analytical design was presented and interpreted. Results from pile testing, numerical simulation and the prediction with respect to slight defect are interpreted. At the end the application of pile testing techniques in Cameroon was analysed.

4.1. General presentation of Douala

The presentation of the city of Douala is the subject of the first part; the city is presented geographically, the climate, relief, hydrology and hydrogeology, geological, demography and economic conditions in order to know the different conditions which generally have a major influence on the design of structures.

4.1.1. Geographical location

Douala is the largest city in Cameroon, the capital of Cameroon's Littoral region with latitude and longitude coordinates 4.061536 (4° 3' 41.5296" N), 9.786072 (9° 47' 9.8592" E) respectively. The city is located in the southwestern part of the country, right in the delta of the Wouri River. Home to central Africa's largest port and its major international airport, Douala International Airport (DLA), it is the commercial and economic capital of Cameroon and the entire CEMAC region comprising of Gabon, Congo, Chad, Equatorial Guinea, Central African Republic and Cameroon (Robert, 2020). Figure 4.1 presents the location of Douala.

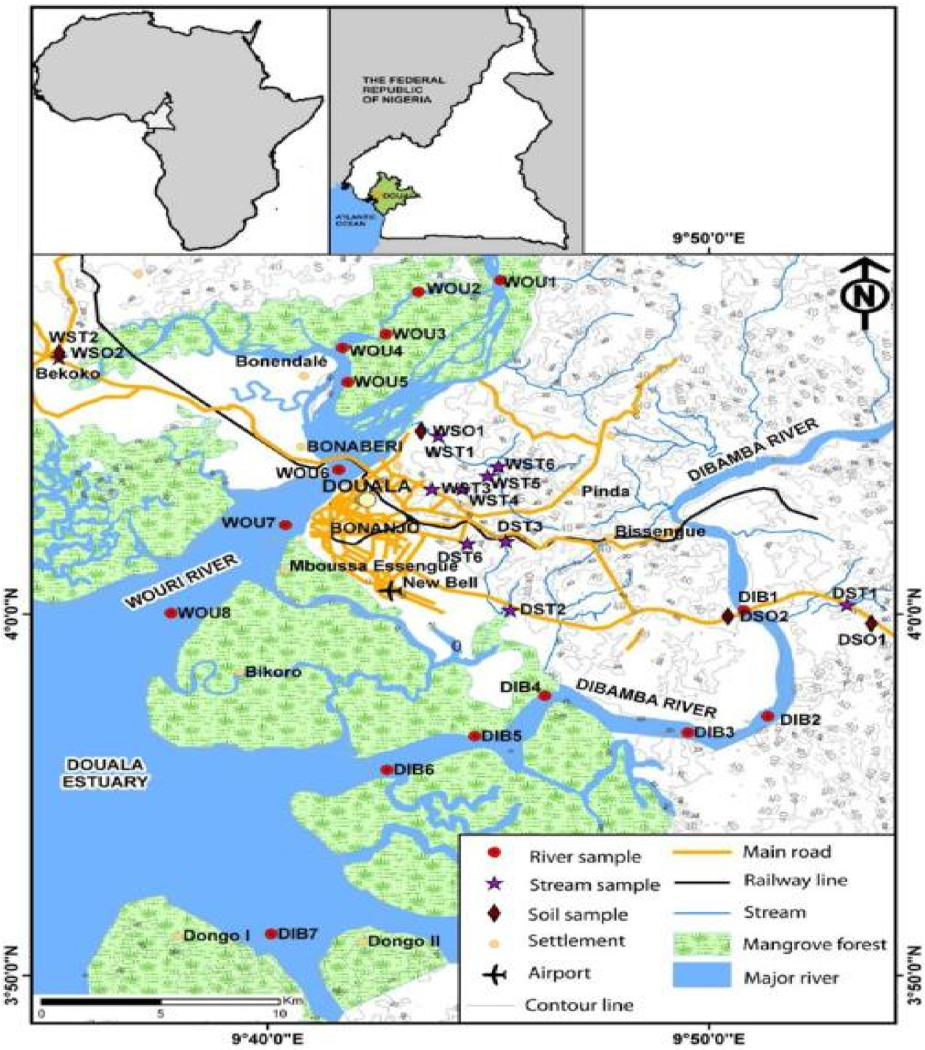


Figure 4. 1 Location of Douala (Robert, 2019)

4.1.2. Climate

The location of Douala permits an existing tropical monsoon climate, with relatively consistent temperatures throughout the course of the year, with dry season (four months) and heavy monsoon the rest of the year. According to the Holdridge life zones system of bioclimatic classification, Douala is situated in or near the subtropical wet forest biome. Douala typically features warm and humid conditions with a mean annual temperature of 26.7°C (Climatemps.com, 2009-2017) which varies by 3.2°C and an average humidity of 83%. Douala sees plentiful of rainfall during the course of the year, experiencing on average roughly 3,600 millimetres of precipitation per year. Its driest month is December, with an average of 28 millimetres of precipitation, while its wettest month is August, when on average nearly 700 millimetres of rain falls. (Douala Climate Normals 1961–1990, n.d.). Figure 4.2 gives an overview of the climate of Douala.

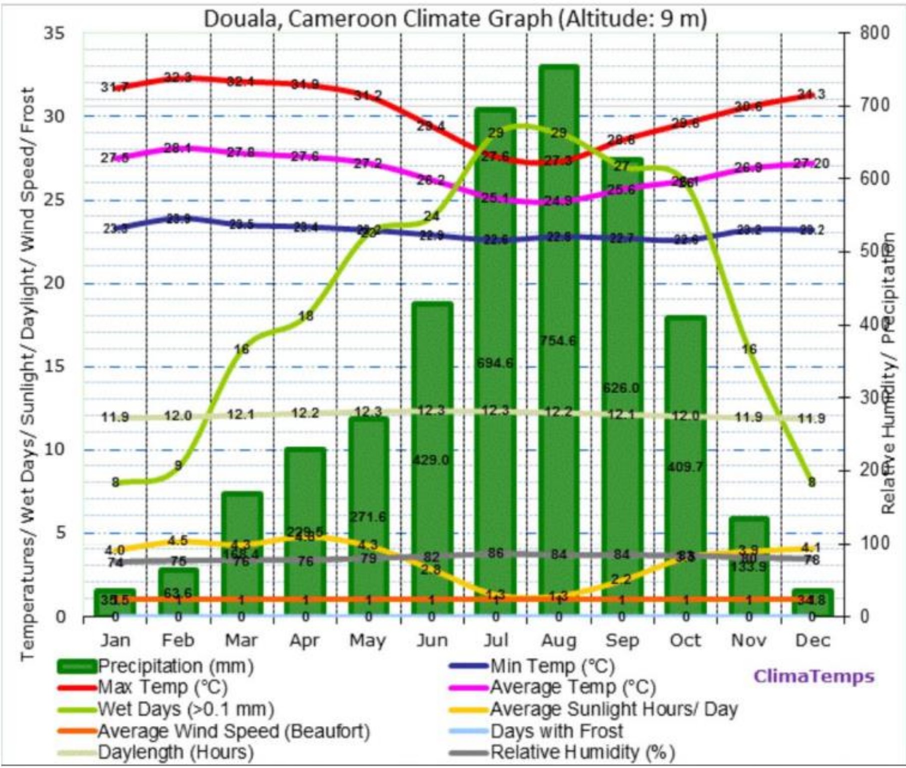


Figure 4. 2 Douala climate graph (climatemps.com, 2009-2017)

4.1.3. 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 (topographic-map.com) and has a morphology whose terrain 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. The coastal region is characterized by a short maritime facade in an arc, and formed by a succession of sedimentary plains, rivers and streams that flow into the Atlantic Ocean in a delta: beaches of golden sand lined with sandy beaches lined with coconut trees in Kribi.

4.1.4. Hydrology and hydrogeology

The geographic position of the Douala basin subjects the city to a relative high humidity due to the long rainy season which feeds the groundwater and streams, follows by a strong evaporation during short dry season. The hydrological system of Douala is very dense. The principal river is the Wouri, within Sanaga, Dibamba, Moungo and Nyong. The city is divided into many watersheds: Bonnes-courses, Epolo, Mbanya, Mbopi, Bologo, Ngoua, Lonmayagui, Kambo, TongoBassa and Beseke (Siaka, 2018). The groundwater regime depends on regional hydrogeology, influenced by the nature of the soil in place.

4.1.5. Geology

The Douala coastal basin is composed mainly of Cretaceous marine sandstones and limestones, which are 1000 to 2000 m thick. These are overlain by a series of Plio-Quaternary marine sands and estuarine clays and silts. As observed in Figure 4.3 the two aquifers are separated by mudstones of the Nkappa formation.

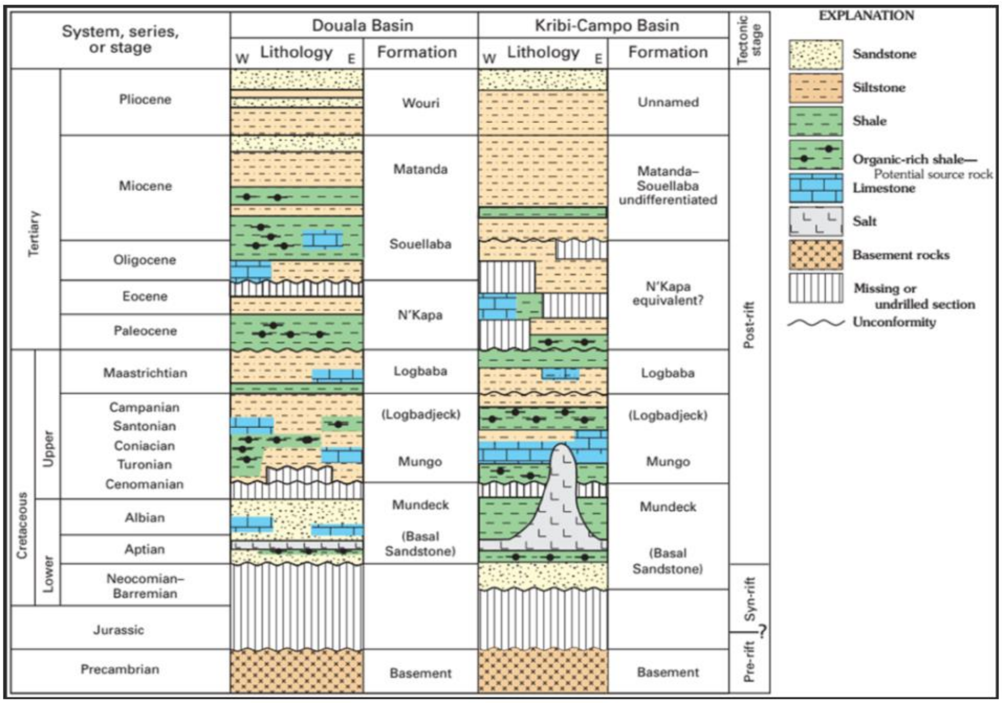


Figure 4. 3 Stratigraphy of the Douala basin (link.springer.com).

4.1.6. Demography

The town of Douala is known to be the most populated and largest town in Cameroon followed by Yaounde. It is spread over an area of 210 km² and had an estimated population of about 3.536 million inhabitants in 2019 (United Nations) hence having a population density of about 16838 inhabitants/km². Table 4.1 shows the population evolution of Douala indicating that the growth rate of this city has not gone over 5% over the past decade while, before, the increase of the population was greater. In the last decades the population can be said to be increasing in a decreasing manner.

Table 4. 1 Population Evolution of Douala from 2009 to 2019 (United Nations - World Population Prospects).

Douala - Historical Population Data		
Year	**Population**	**Growth Rate**
2019	3,536,000	3.63%
2018	3,412,000	4.69%
2017	3,259,000	4.72%
2016	3,112,000	4.71%
2015	2,972,000	4.72%
2014	2,838,000	4.68%
2013	2,711,000	4.71%
2012	2,589,000	4.73%
2011	2,472,000	4.70%
2010	2,361,000	4.70%
2009	2,255,000	4.74%

4.1.7. Economic activities

Douala is a city with a modest oil resource in Africa, but is in excellent agricultural condition, therefore it has one of best economies in Africa. However, it also faces some problems like other underdeveloped countries such as high dependence on civil service and bad climate (flood, tornado, storm …) that affect the business development.

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 (Avameg, Inc). While the aforementioned percentage is doubled for rural regions, poverty is a growing problem for Douala due to its steadily increasing population. Unlike the rural populations of Cameroon that can grow their own foods to lessen their expenses, Douala locals are disadvantaged by living in the port city where there are not many opportunities for monetary gain. Nevertheless, about 80% of industries in Cameroon are found in this area making it a highly industrialized town.

4.2. Presentation of the project

In this part a presentation of the project was done focusing on the description of the project area and describing the project itself.

4.2.1. Description of project area (Bonaberi industrial zone)

The industrial zone of Bonabéri (ZIBO) in Douala covers a total area of 192 ha (magzicameroun.com, 2021).This zone is a set of lots of land and infrastructure in Bonabéri where several industries (more than 70) and companies such as Cimencam are located. ZIBO is one of the two industrial zones of the City of Douala. The fields of activity covered are Steelworks, Food industry, Construction and public works, Breweries, Cement works, Car dealership, Fertilizers and pesticides, Metallurgy, Sawmill, Tannery, Waste treatment. It is located in the district of Douala IV. The management of the Douala-Bonabéri industrial zone is ensured by the Mission for the development and planning of industrial zones (Magzi) under the supervision of the Ministry of Mines, Industry and Technological Development (minmidt.cm, 2021).

Figure 4. 4 Bonabéri industrial zone (devseed.com, 2021)

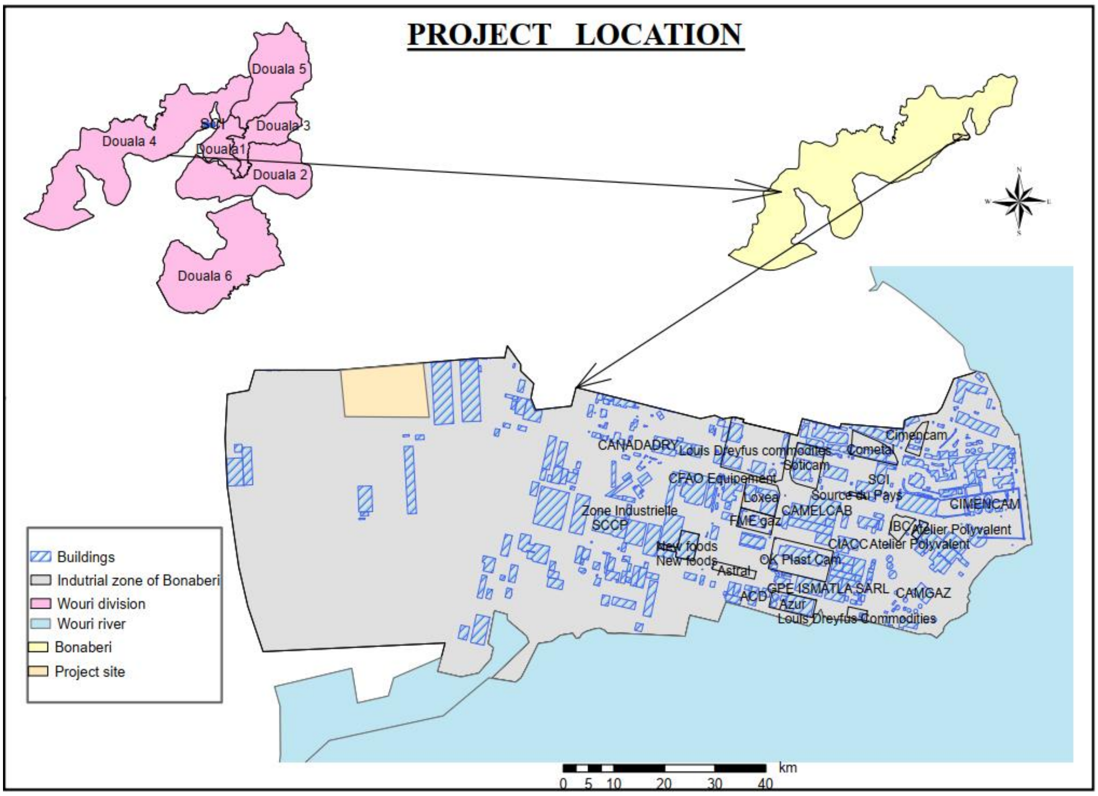


Figure 4. 5 Bonabéri industrial zone (Openstreetmap, 2021)

4.2.2. Project description

The project in this case entails the construction of an oil and cosmetics processing unit in the Magzi industrial zone in the city of Douala, littoral region. The project consists of the installation of product storage tanks for the unit. Initially the foundation opted to support the tanks was a raft foundation. During the construction of the raft and tank supports due to the poor nature (site made up of continental and fluviomarine boreholes, predominantly sandy and clayey, locally having under a layer of gravelly sand fill, a layer of compressible formations [mud, soft clay, loose sand…] sunk in water table whose level is at 50 cm dating from the tertiary and early quaternary periods) of the soil below, it was observed that the raft exhibited some differential settlement ranging from 11 cm to 25 cm in different areas. Due to this effect geotechnical studies were effected in order to remedy the situation. The investigations were made with aim not only to determine the nature of these soils, but also to calculate the permissible bearing capacity of the load-bearing soil and as well propose a solution. After the investigation, some solutions were being proposed and that which was implemented was the use of piles to transfer the loads of the structure to a deeper soil and thus limit the settlement. In construction of these piles, a quality control test (Low strain integrity test) was carried out after curing to verify the structural integrity of the piles, which is what these work is about. Figure 4.6 indicates head of a pile realised beneath the raft.

Figure 4. 6 Pile inserted beneath raft

4.3. Data collected

The data of the project obtained were the geometric and geotechnical data

4.3.1. Geometrical characteristics

The geometrical information obtained includes the geometrical characteristic of the piles in the zone of interest which consisted of 20 piles, all of similar characteristics that are;

- Diameter of 1 m
- Length of 12 m

Figure 4.7 presents the layout and coordinates (XY) in the zone

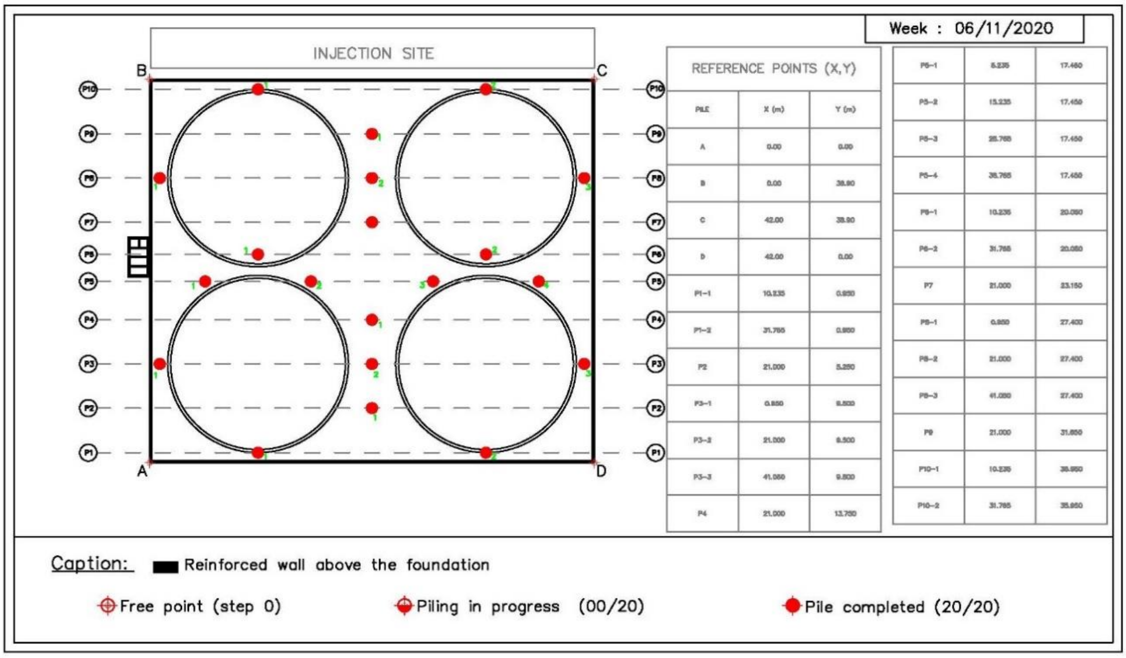


Figure 4. 7 Layout of piles

4.3.2. Geotechnical data

For the design of any geotechnical structure it is necessary to have the geotechnical data of the materials concerned before construction so as to decide on the type of structure to build (foundations, retaining walls, culverts etc....). These data are necessary to identify, classify and characterise the soil.

The survey and test data available from the geotechnical report were not completely useful as some information were lacking. Under these conditions, in order to make the proposed geotechnical models more reliable, further reconnaissance will be required.

4.3.2.1. Summary of available data

The soil recognition campaign carried out consisted of:

- 4 rotary-pressuremeter boreholes drilled to a depth of 20 m.
- Identification tests from the drilled samples.

Figure 4.8 presents the plan indicating the position of the boreholes in which the investigation were carried out which were governed by the settlement of the raft.

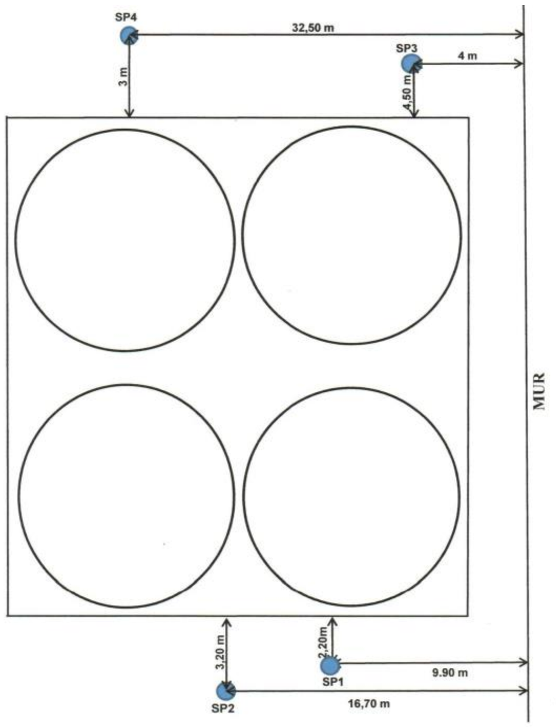


Figure 4. 8 Layout plan of Pressuremeter points

a. Soil stratigraphy

The Destructives boreholes done on the area give us the stratigraphy of the soil briefly presented in the Figure 4.9 consisting mainly of clay and sand.

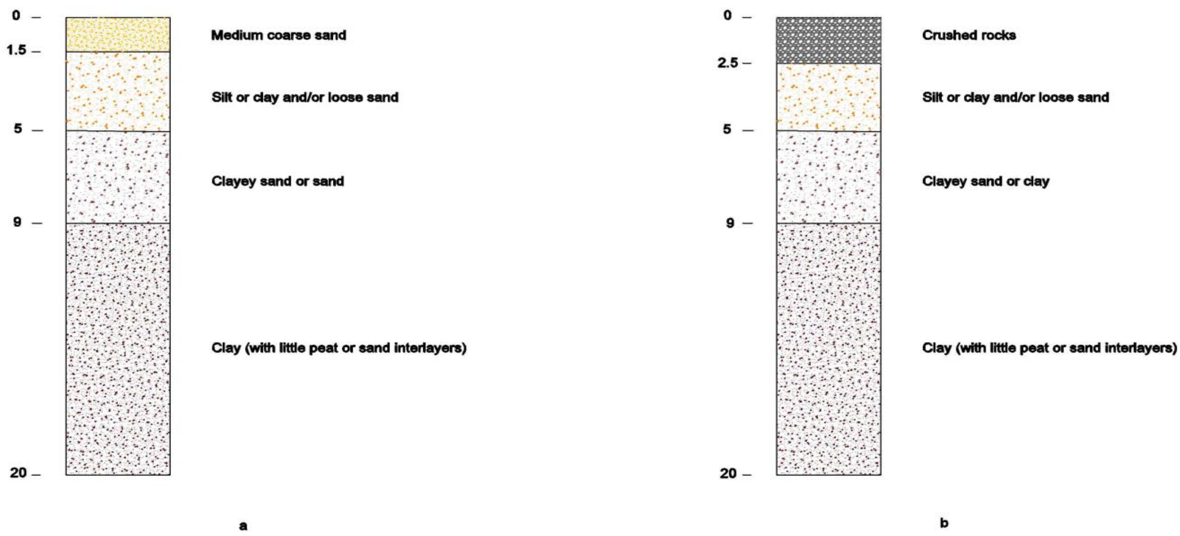


Figure 4. 9 Soil profile (a) initial stratigraphy (b) stratigraphy after soil improvement

b. Data from Pressuremeter test

Pressure testing was carried out to a depth of 20m for all four boreholes, with pressure measurements taken every metre and a summary of the results obtained are presented in Table 4.2.

Table 4. 2 Insitu test result (Source: Project document)

<table>
<tr><th rowspan="2">Depth</th><th colspan="2">SPR 1</th><th colspan="2">SPR 2</th><th colspan="2">SPR 3</th><th colspan="2">SPR 4</th></tr>
<tr><th>Pl</th><th>E</th><th>Pl</th><th>E</th><th>Pl</th><th>E</th><th>Pl</th><th>E</th></tr>
<tr><td>1
2
3
4</td><td>2.5 < pl < 3.5</td><td>21.26 < E < 34.3</td><td>3 < pl < 3.75</td><td>19.37 < E < 36.02</td><td rowspan="2">3.25 < pl < 7.25</td><td rowspan="2">32.7 < E < 129.36</td><td rowspan="3">2.5 < pl < 9.5</td><td rowspan="3">32.7 < E < 129.36</td></tr>
<tr><td>5
6
7</td><td rowspan="4">4.5 < pl < 9.5</td><td rowspan="4">19.67 < E < 124.41</td><td rowspan="3">4.5 < pl < 10.5</td><td rowspan="3">35.07 < E < 65.99</td></tr>
<tr><td>8
9
10
11</td><td rowspan="4">4.75 < pl < 12.75</td><td rowspan="4">67.96 < E < 247</td></tr>
<tr><td>12
13</td><td rowspan="3">5.5 < pl < 13.5</td><td rowspan="3">41.85 < E < 206.9</td></tr>
<tr><td>14</td><td rowspan="2">9.5 < pl < 13.5</td><td rowspan="2">63.57 < E < 353.78</td></tr>
<tr><td>15
16
17
18
19
20</td><td>9.25 < pl < 12.5</td><td>110.06 < E < 353.78</td></tr>
<tr><td></td><td colspan="2">3.27 < E/pl < 132.16</td><td colspan="2">5.2 < E/pl < 16.7</td><td colspan="2">10.06 < E/pl < 26.80</td><td colspan="2">6 < E/pl < 17.6</td></tr>
</table>

c. Ground water condition

The ground water level in the site of the project was found to be at 0.5 m from the ground surface and since the raft was of thickness 0.5 m, the ground water level for the piles coincides with the ground surface (submerged by water).

4.3.2.2. Material characteristic

a. Pile characterisation

In this section, it is given a brief description about the properties of the concrete and the steel reinforcement bars used for the pile.

The pile used in this project is constructed using the technology of drilled or bored pile (with soil removal).

Table 4. 3 Concrete characteristics

Property	Value	Unit
Class	B30	-
fck	30	N/mm^2
fcm	38	N/mm^2
fctm	2.9	N/mm^2
fctd	1.4	N/mm^2
E	33000	N/mm^2
ν	0.2	-
G	10748.5	N/mm^2
γ	25	KN/m^3
Cnom	0.07	m

Table 4. 4 Reinforcement characteristics

Property	Value	Unit
Class	B400A	-
fyk	400	N/mm^2
fyd	348	N/mm^2
E	210000	N/mm^2
ν	0.3	-

b. Summary of soil parameters

After the reconstitution of the soil layers in our work area was done to improve the strength of the soil, from the test results together with some documentary research (literature, project data of neighbouring sites), a summary of all the soil parameters that was used in all our analysis are defined in the Table 4.5.

Table 4. 5 Summary of soil parameters

Parameters	Soil thickness	Soil unit weight			Frictional angle	Dilantancy angle	Cohesion		Elastic modulus	Poisson's ratio	At rest pressure coef
Soil layer	H(m)	Yd (KN/m³)	Ysat (KN/m³)	Y (KN/m³)	φ' (°)	ψ (°)	C' (kPa)	Cu (kPa)	E (kPa)	ν	k_0
Crushed rocks	2			20	41	11	0	0	15 000	0.3	0.34
Silt or Clay and/or loose sand	2.5			19	30		0		20 000	0.25	0.6
Clayey sand or clay	4			19	30		5	36	15 500	0.3	
Clay (little peat or sand interlayers)	11	16.5	19		23		30	35	10 000	0.35	

4.4. Analytical calculation of pile

Information obtained from the project report with regards to the load transfer, together with the Eurocodes permitted the analytical calculation of the foundation. A geotechnical and structural design of an axially loaded pile was made using the limit state design method.

4.4.1. Bearing capacity calculation of pile

The bearing capacity was calculated with the help of an excel file mounted, taking into consideration the different design approaches 1, 2 and 3 of Eurocode 7 part 1 and a summary of the results presented in Table 4.6 and Table 4.7 using the procedure described in section 3.4.2.1

Table 4. 6 Short term analysis

ULS verification in SHORT TERM CONDITION						
Design Approach			1-C1	1-C2	2	3
Actions						
Load (kN)	123.2	Ld (kN)	166.32	123.2	166.32	123.2
Selfweight (kN)	226.19	Wd (kN)	305.36	226.19	305.36	226.19
total applied vertical load, Qd (kN)			**471.68**	**349.39**	**471.68**	**349.39**
Base Resistance, Rb						
cu,d (kPa)			35.00	25.00	35.00	25.00
σv (kPa)			372.50	372.50	372.50	372.50
Rb (kN)			544.08	472.22	544.08	472.22
Base resistance of design, Rbd (kN)			**435.27**	**295.14**	**494.62**	**472.22**
Shaft Resistance, Rs						
layer 1		cu,d (kPa)	0.00			
		As (m2)	6.28			
		Rs1 (kN)	**0.00**			
layer 2		cu,d (kPa)	0.00			
		As (m2)	7.85			
		Rs2 (kN)	**0.00**			
layer 3		cu,d (kPa)	36.00	25.71	36.00	25.71
		As (m2)	5.65			
		Rs3 (kN)	**158.34**	**113.10**	**158.34**	**113.10**
layer 4		cu,d (kPa)	35.00	25.00	35.00	25.00
		As (m2)	11.00			
		Rs4 (kN)	**134.70**	**96.21**	**134.70**	**96.21**
total shaft resistance, Rsu (kN)			293.03	209.31	293.03	209.31
Shaft resistance of design, Rsd (kN)			**293.03**	**161.01**	**266.39**	**209.31**
Total resistance of pile, Q (kN)			**728.30**	**456.14**	**761.02**	**681.53**
Verify, Rg/Qd > 1			1.544	1.306	1.613	1.951

With respect to the result of the verification in ULS in short term condition, all the approaches were verified and the second combination of design approach 1 was seen to be the most critical approach among all in this case.

Table 4. 7 Long term analysis

ULS verification in LONG TERM CONDITION						
Design Approach			1-C1	1-C2	2	3
Actions						
Load (kN)	123.2	Ld (kN)	166.32	123.2	166.32	123.2
Selfweight (kN)	226.19	Wd (kN)	305.36	226.19	305.36	226.19
total applied vertical load, Qd (kN)			**471.68**	**349.39**	**471.68**	**349.39**
Base Resistance, Rb						
φ'd (°)			38.00	32.01	38.00	32.01
σ'v (kPa)			137.87	137.87	137.87	137.87
Rb (kN)			1080.86	1080.86	1080.86	1080.86
Base resistance of design, Rbd (kN)			**864.68**	**675.53**	**982.60**	**1080.86**
Shaft Resistance, Rs						
layer 1		K = K0	0.384	0.470	0.384	0.470
		tanδ	0.781	0.625	0.781	0.625
		Rs1 (kN)	**19.23**	**18.81**	**19.23**	**18.81**
layer 2		K = K0	0.546	0.623	0.546	0.623
		tanδ	0.510	0.408	0.510	0.408
		Rs2 (kN)	**25.10**	**22.89**	**25.10**	**22.89**
layer 3		K = K0	0.546	0.623	0.546	0.623
		tanδ	0.510	0.408	0.510	0.408
		Rs3 (kN)	**64.26**	**58.61**	**64.26**	**58.61**
layer 4		K = K0	0.658	0.720	0.658	0.720
		tanδ	0.364	0.291	0.364	0.291
		Rs3 (kN)	**76.04**	**66.60**	**76.04**	**66.60**
total shaft resistance, Rsu (kN)			184.62	166.92	184.62	166.92
Shaft resistance of design, Rsd (kN)			**184.62**	**128.40**	**167.84**	**166.92**
Total resistance of pile, Q (kN)			**1049.30**	**803.93**	**1150.43**	**1247.77**
Verify, Rg/Qd > 1			2.225	2.301	2.439	3.571

With respect to the result of the verification in ULS in long term condition, all the approaches were verified and the second combination of design approach 1 was seen to be the most critical approach among all.

From the results, looking at the differences in the shaft and base resistances in the different conditions and approaches, it can be seen that the base resistance has a greater contribution to the overall resistance of this pile and so can be referred to as an end bearing resistance pile.

4.4.2. Settlement calculation

In order to very SLS of the foundation, a settlement analysis of the pile was made using the method of Randolph and Wroth as described in section 3.4.2.2 and a summary of the analysis presented in Table 4.8.

Table 4. 8 Settlement analysis of pile

Calculation of the overall pile stiffness		
Pile diameter	D0 (m)	1.00
Shear modulus	G1 (Mpa)	5.77
	G2 (Mpa)	8.00
	G3 (Mpa)	5.96
	G4 (Mpa)	3.70
Pile length	Lp (m)	12.00
Pile segment	Ls1 (m)	2.00
	Ls2 (m)	2.50
	Ls3 (m)	4.00
	Ls4 (m)	3.50
Base Stiffness	**Kb (MN/m)**	**11.40**
Shaft Stiffness 1	Ks1 (MN/m)	22.81
Shaft Stiffness 2	Ks2 (MN/m)	39.54
Shaft Stiffness 3	Ks3 (MN/m)	47.15
Shaft Stiffness 4	Ks4 (MN/m)	25.63
Shaft Stiffness	**Ks (MN/m)**	**135.13**
Overall pile stiffness	**Kpile (MN/m)**	**146.52**
Settlement	**S_i(mm)**	0.841

With respect to the result obtained it was noticed that since the Randolph and wroth method considers the soil to be elastic, the settlement obtained here is the elastic settlement (immediate settlement) realised just after the end of the construction without having enough time for the dissipation of the pore water pressure and thus little or negligible consolidation of the soil thereby a very small value of displacement (settlement).

4.4.3. Structural design of pile

Considering the maximum load transmitted to the pile according to the different design approaches considered, a brief structural design was made to evaluate the structural capacity of the pile.

$Q = 471.68\ kN$

The cross-sectional area of the pile is: $A_c = 0.785\ m^2$

This leads to a minimum reinforcement of $A_{s,min} = 2500\ mm^2$ which corresponds to 8 Φ20. The reinforcement proposed is 9 Φ20 spaced at 300 mm. The design compressive resistance obtained using equation 3.14 is:

$$R_{cd} = 14413455.1\ MPa > \frac{Q}{A_c} = \frac{471.68 \times 10^3}{785} = 600.9 MPa.$$

For the transversal reinforcement, 6 mm bar is recommended at a spacing of 400 mm reduced by a factor of 0.6 near lapped joints.

4.5. Pile testing results

The quality of the PIT results is a direct function of the operator's familiarity with the system and experience with pile foundations, e.g., factors such as pile surface preparation for attachment of the sensors, use of certain hammer weight for certain pile size, data processing, etc., can readily influence the results if their contributions are not recognized (Massoudi et al, 2004). The results obtained was through documentation from the project report.

4.5.1. Curves of test results

After the data from the test (reflectograms) were collected, they were then sent for analysis and interpretation by a specialist Engineer using advanced PIT software and then a report was made on the final results. Presented here are the time domain results of 4 piles.

4.5.1.1. Pile number 1

Three signals were recorded per pile for a better analysis of the integrity as recommended by the norm. Shown in Figure 4.10.

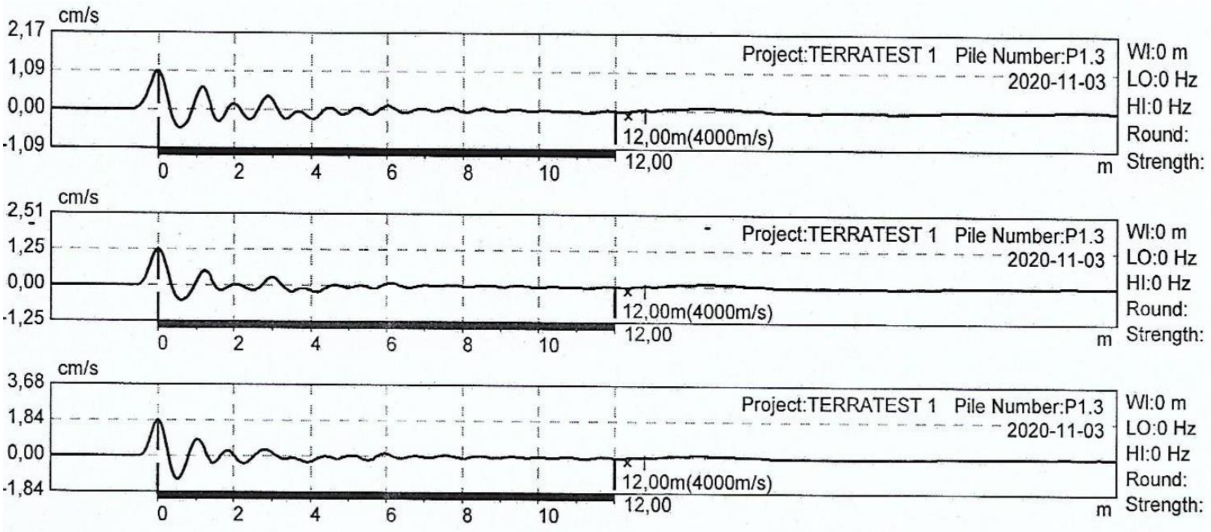


Figure 4. 10 Signals of pile number 1

4.5.1.2. Pile number 2

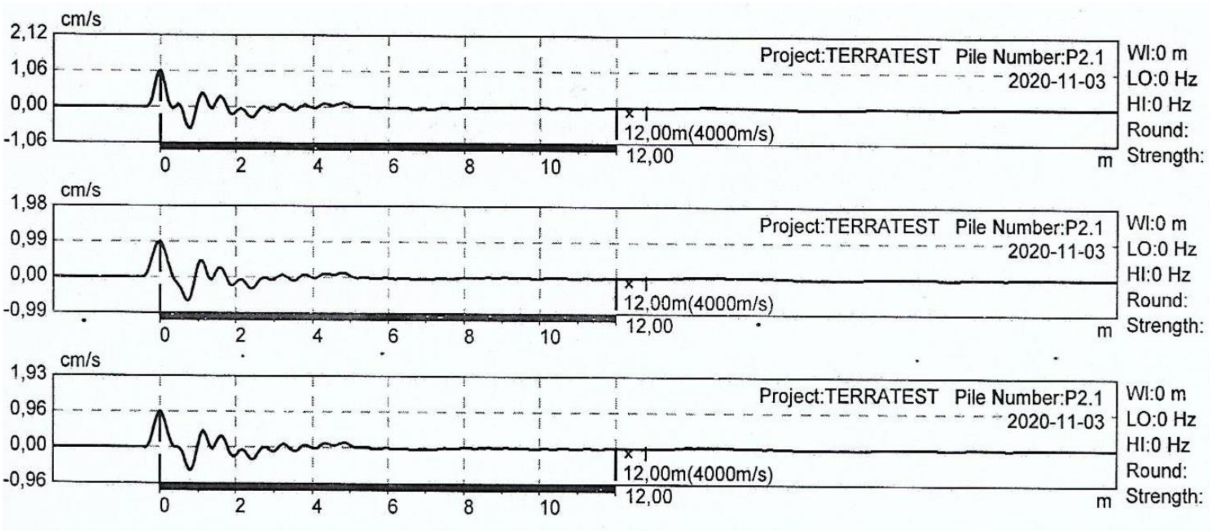


Figure 4. 11 Signals of pile number 2

4.5.1.3. Pile number 3

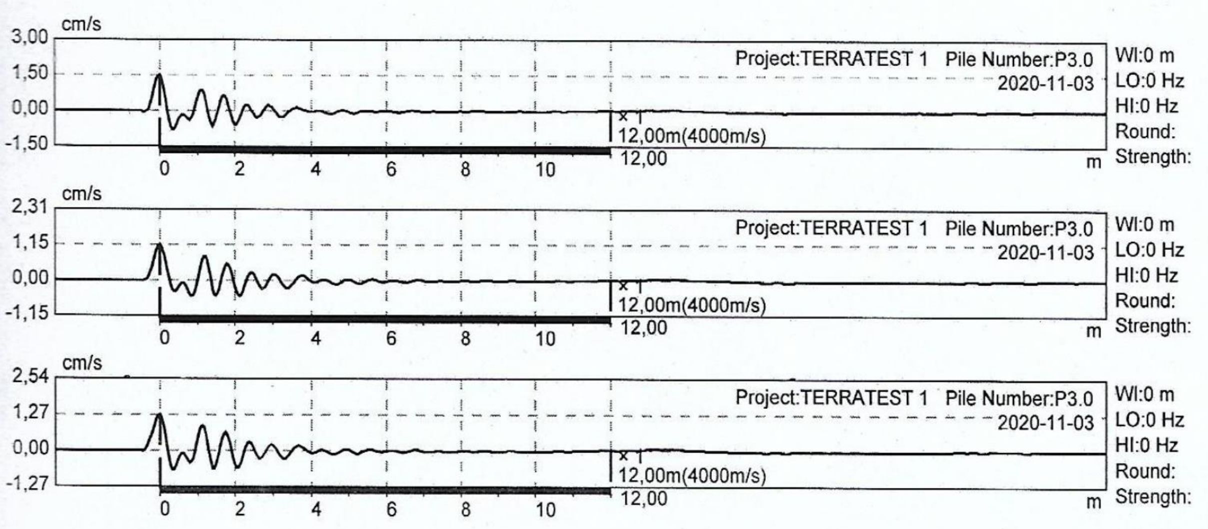


Figure 4. 12 Signals of pile number 3

4.5.1.4. Pile number 4

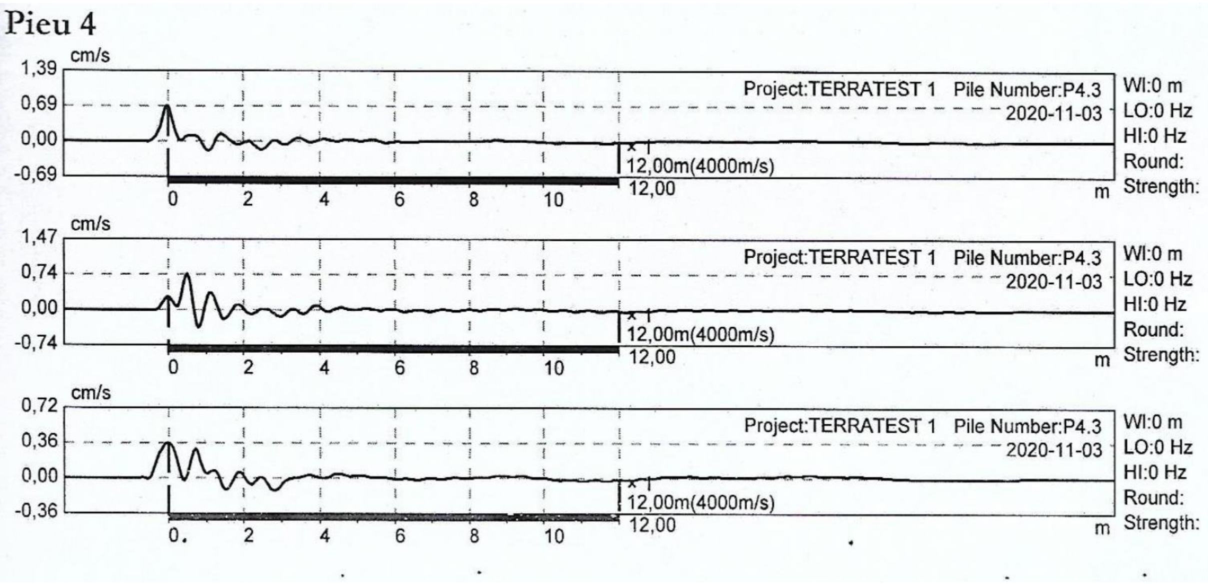


Figure 4. 13 Signals of pile number 4

4.5.2. Interpretation

After the analysis of the signals were done, the interpretation of the signals were then made and presented. Table 4.9 and Table 4.10 presents the details of the interpretation of pile number 3 and 4 respectively.

Table 4. 9 Pile number 3

Sequence number	Pile number	Pile type	Pile Diameter (mm)	PL (m)	Concrete class	V (m/s)	Pile quality
1	3	Round	1000	12.00	B30	4000	Complete pile
2	3	Round	1000	12.00	B30	4000	Complete pile
3	3	Round	1000	12.00	B30	4000	Complete pile

Integrity of the pile with no anomalies and probably homogeneous concrete

Table 4. 10 Pile number 4

Sequence number	Pile number	Pile type	Pile Diameter (mm)	PL (m)	Concrete class	V (m/s)	Pile quality
1	4	Round	1000	12.00	B30	4000	Complete pile
2	4	Round	1000	12.00	B30	4000	Complete pile
3	4	Round	1000	12.00	B30	4000	Complete pile

Integrity of the pile with no anomalies and probably homogeneous concrete.

A summary of the different signals analysed is presented in Table 4.11

Table 4. 11 Summary table of results

Pile number	Pile quality	Integrity validation	Homogeneity/heterogeneity
1	No defect	Validated	Homogeneous concrete

2	No defect	Validated	Homogeneous concrete
3	No defect	Validated	Homogeneous concrete
4	No defect	Validated	Homogeneous concrete

From all the analysis made from the different signals, it was concluded that the different piles were of good integrity and a homogeneity of the concrete was noted.

4.6. Numerical prediction of pile behaviour

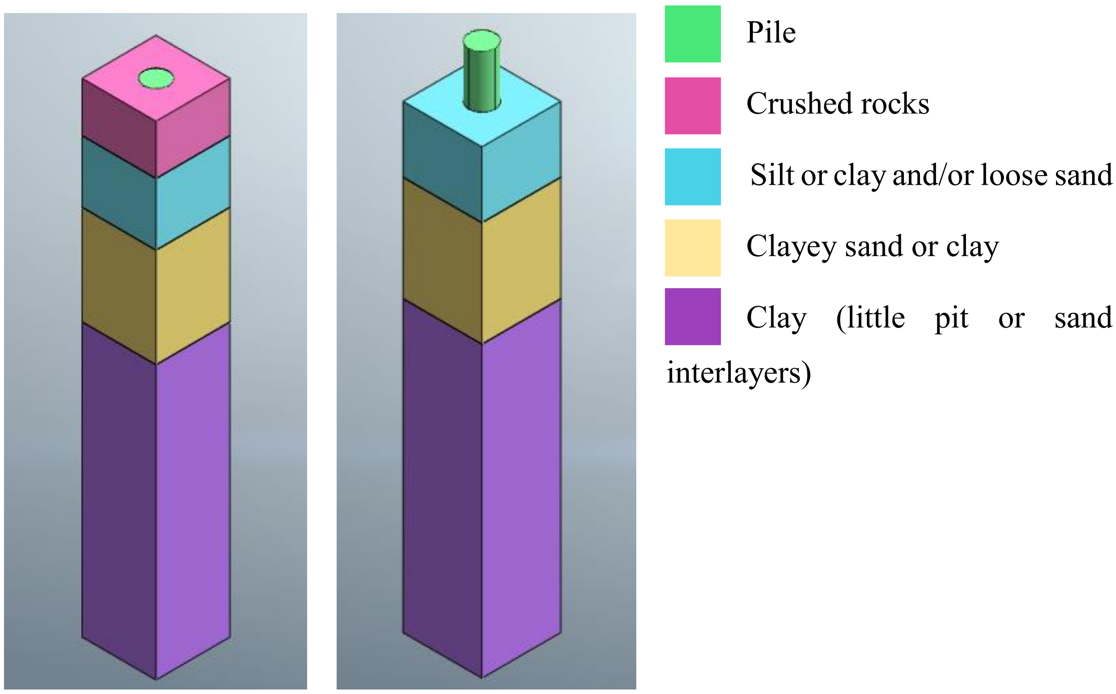
Predicting the behaviour of a pile is very important for both design and verification of the pile and as well the control of the quality (performance) of the foundation. This section deals with numerical analysis of an axially loaded pile executed with the software MIDAS GTS NX. The procedure previously detailed in part 3.7 were implemented to analyse the pile, carry out a parametric analysis and simulate the effect of a defect on the pile.

4.6.1. Analysis of pile

The pile was analysed considering two models, one with an interface and the other without any interface defined between soil and pile surface. The two models were all evaluated and the differences identified.

4.6.1.1. Finite Element model and construction phases

The finite element model has been made in MIDAS GTS NX using a 3D model. As detailed in section 3.7, Figure 4.14 presents the model of the system adopted for the analysis. The ground water table is located at the soil surface, this implies the influence of water is taken into consideration along the pile length.



a b

Figure 4. 14 Model (a) complete soil and pile (b) pile in soil without a soil layer

4.6.1.2. Material properties

The linear elastic model based on Hooke's law of isotropic elasticity is used for the pile, in this study the soil is modelled using the Mohr coulomb model as the soil parameters obtained could only permit the use of this model. Table 4.12 present the properties of the materials used for the analysis.

Table 4. 12 Properties and parameters used

Name	crushed rocks	Silt or Clay and/or loose sand	Clayey sand or clay	Clay (little peat or sand interlayers)	Pile
Material	Isotropic	Isotropic	Isotropic	Isotropic	Isotropic
Model type	Mohr coulomb	Mohr coulomb	Mohr coulomb	Mohr coulomb	Linear elastic
General					
Elastic modulus (E)	1.50E+04	2.00E+04	1.55E+04	1.00E+04	3.30E+07
Poisson's ratio (v)	0.3	0.25	0.3	0.35	0.2
Unit weight	20	19	19	16.5	25
Ko	0.34	0.6	-	-	-
Non-Linear					
Cohesion (C)	0	0	5	30	-
Frictional angle	41	30	30	23	-

4.6.1.3. Mesh

Generating a proper Finite Element mesh is a very important intermediate step between the definitions of the geometry and the construction stages. In order to have a smooth and accurate calculation the finite element mesh has to be of good quality, that is to say, the elements should be regular without being excessively long and thin. For the accuracy of the calculation, the elements should be small enough, especially in those areas where significant changes in stress or strain can be expected during the analysis (plaxis2DCE-V21.01-02-reference). In this study a hybrid mesher of sizes 0.2 and 0.5 were used for the pile and soil respectively.

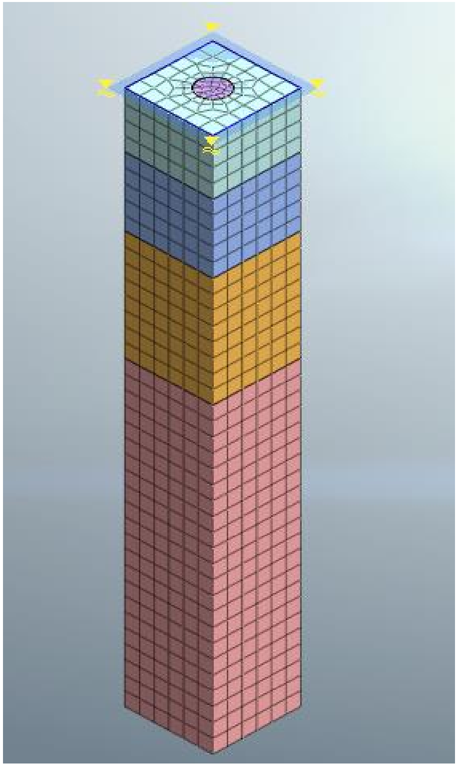
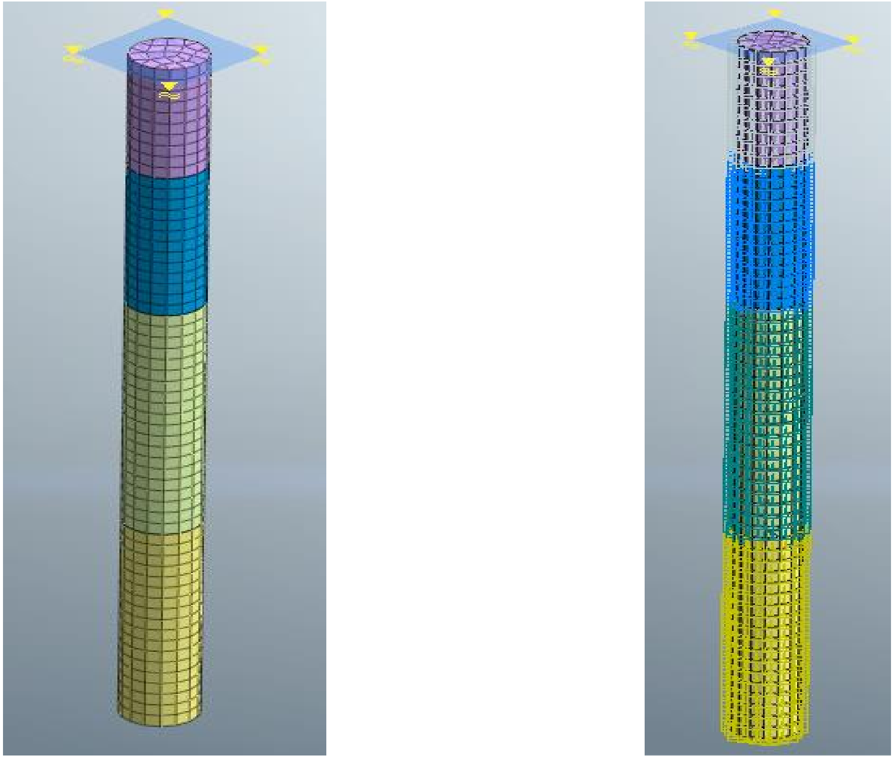


Figure 4. 15 Mesh generated

4.6.1.4. Interface

For the model including the interface, with the help of the interface wizard, an interface was generated for each soil layer following the recommendations of the manual.



(a) (b)

Figure 4. 16 Interface definition (a) no interface (b) interface defined

4.6.1.5. Calculation steps

The construction stage analysis type was used in this work considering three different construction stages, initial condition, pile construction and loading which are clearly detailed in section 3.7.2.5. Figure 4.17 presents the definition of the initial condition construction stage.

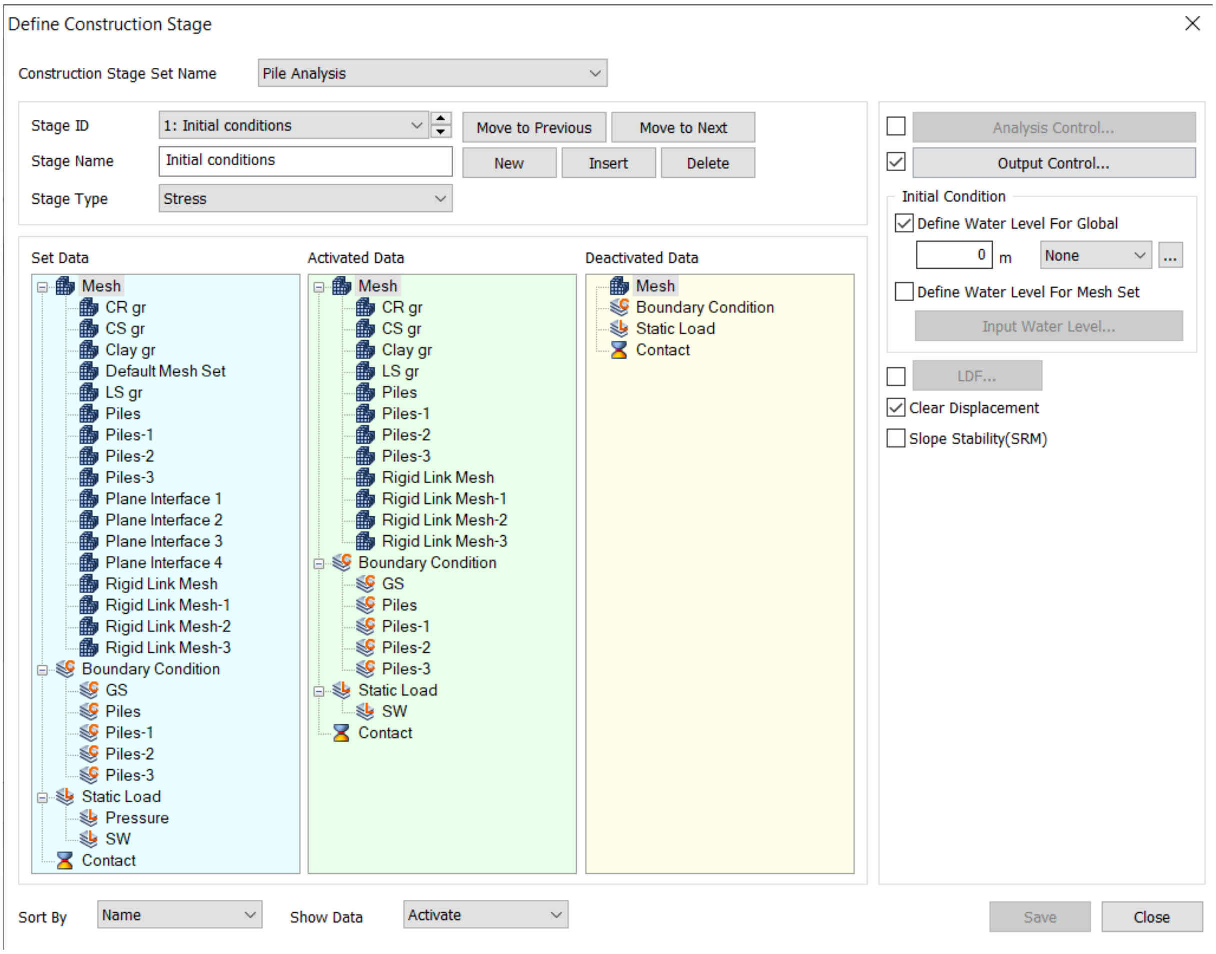


Figure 4. 17 defining construction stages

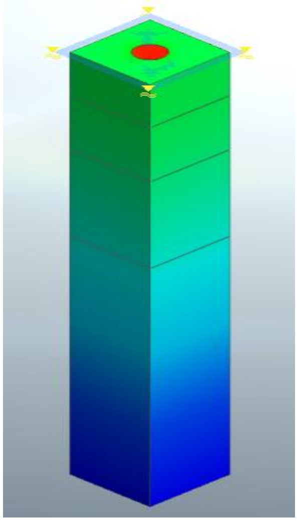
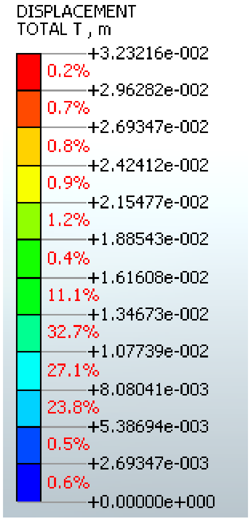
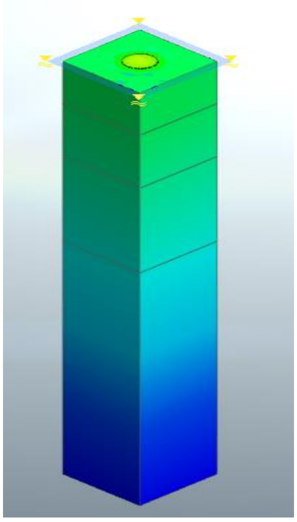
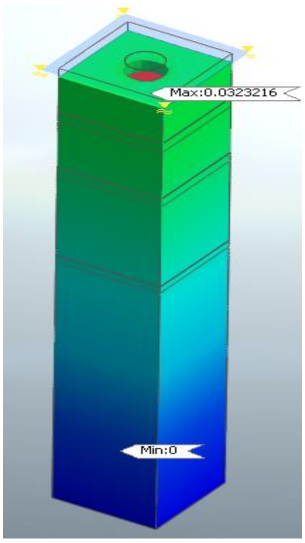
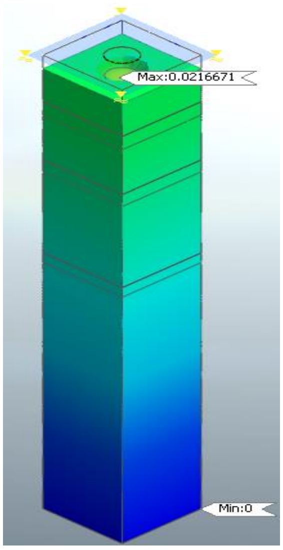
4.6.2. Analysis Results

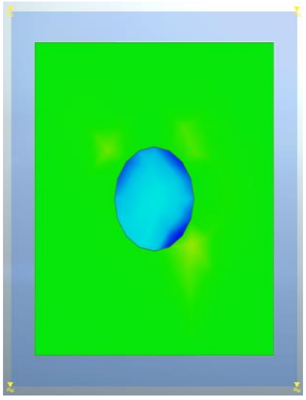
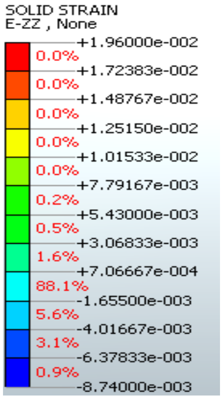
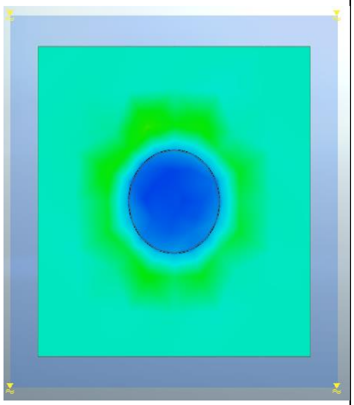
After performing the different analyses on both models, different results could be obtained to characterise the performance of the pile. In this part some of the most essential of the results were presented either graphically or extracting tables from which curves were plotted.

4.6.2.1. Graphical results

Here are presented some screen shots of the results from the model. Table 4.13 presents for both models the results of the total vertical displacement, deformed shape and vertical strain.

Table 4. 13 Results of two pile models

Result parameter	Interface		No interface
Total vertical displacement (T)			
Deformed shape			

Strain (E-ZZ)			

From these graphical details a few observations were made such as

- The pile modelled with an interface indicates higher settlements (3.23 > 2.166 cm) as compared to the pile modelled without an interface. This can be explained by the fact that since the interface serves as a clear definition of the interaction pile/soil, an absence of the interface implies a more rigid connection between the pile shaft and the soil and so more friction between the pile and the soil beyond the real nature thus a reduction of the total settlement witnessed.
- From the deformed shape and strain results, still due to the overestimation of the connection between soil and pile due to lack of interface, it is noticed a great deformation of the soil around the pile which is very less in the model with an interface.

4.6.2.2. Curves from result tables

Result concerning a particular vertical profile of the pile and of the soil around pile was extracted and from which a total displacement against depth was plotted. Figure 4.18 indicates the curves of both models along a vertical profile on the soil around pile and Figure 4.19 indicates the curves of both models along the pile.

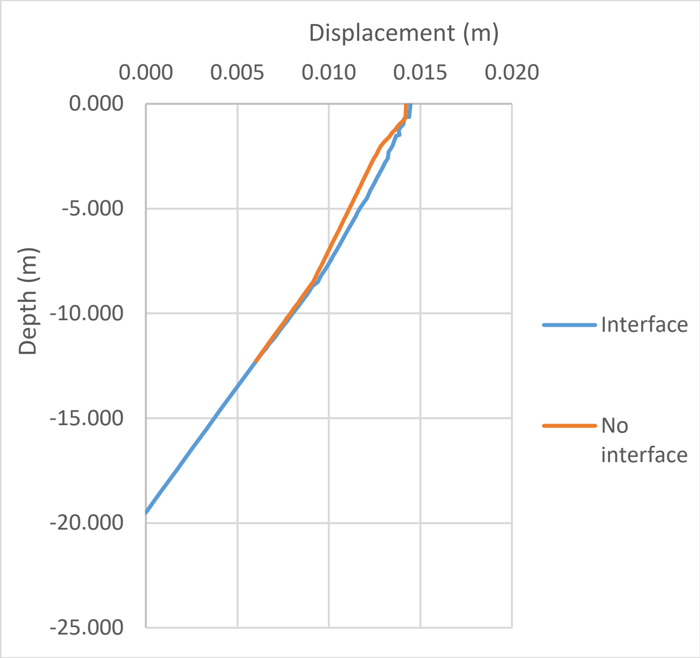


Figure 4. 18 Displacement against depth of soil profile

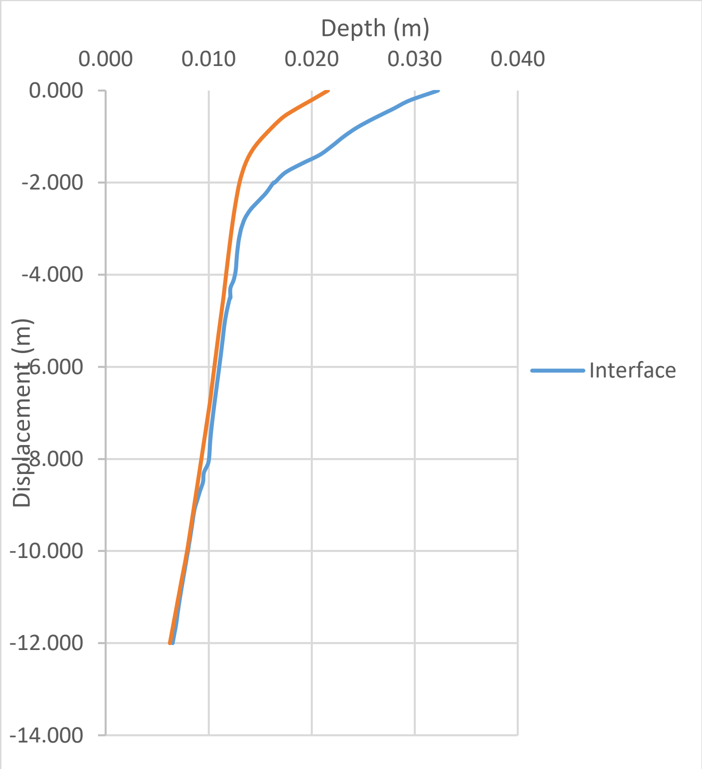


Figure 4. 19 Displacement against depth of pile

Looking at the curves, it can be noticed that for the pile, the displacements are maximum at the head (ground surface) and decreases down the pile with the pile model with interface having higher displacements than the model without interface.

For the deformation of the soil around the pile, the difference can be noticed from a depth of 1-9 m with the model with interface being of higher displacement.

Though with the differences noticed from graphical results or curves, both models satisfy both the ULS (model converges) and SLS ($T < 0.05D = 5$ cm) conditions.

4.6.3. Parametric analysis

A parametric study of the pile was carried out to check the influence of some soil parameters of the most influencing soil layer (the bearing soil) and the parameters varied are, the internal angle of friction, the elastic modulus and the cohesion. The behaviour of the pile assessed after varying these parameters. The model used for these analyses is the pile model with interface element defined.

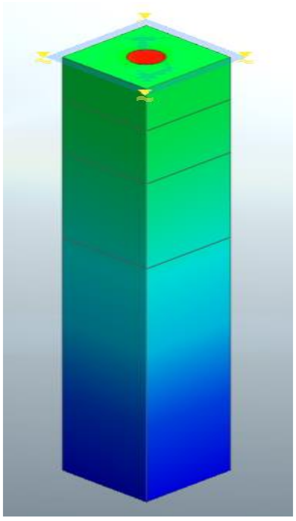
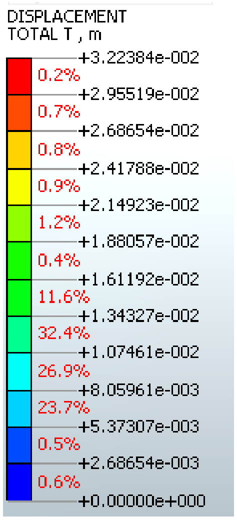
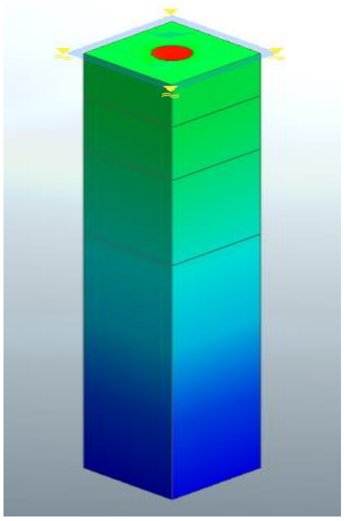
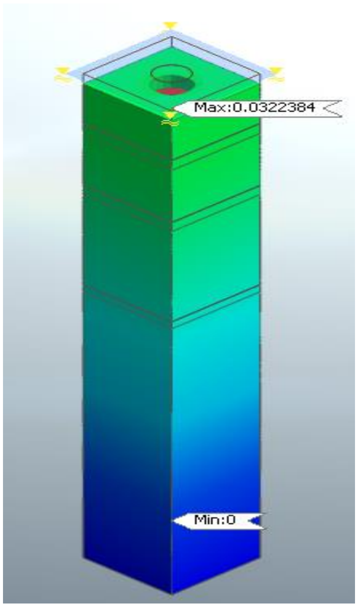
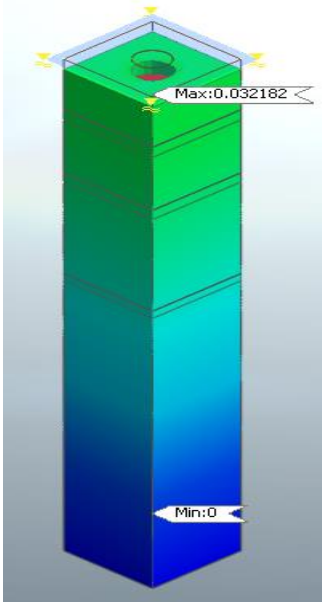
4.6.3.1. Variation of the angle of friction

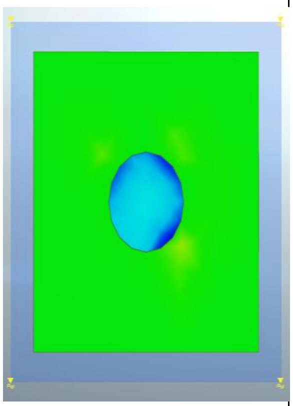
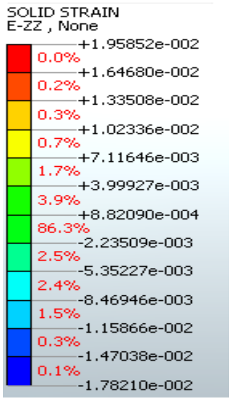
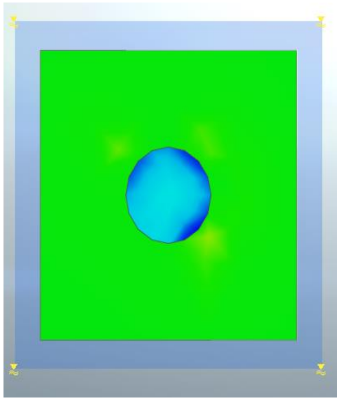
The influence of the variation of the internal angle of friction was studied considering a variation of ± 5 in accordance with the parameter range of the soil type from the literature.

a. Graphical results

Here are presented some screen shots of the results from the model. Table 4.14 presents for both models the results of the total vertical displacement, deformed shape and vertical strain.

Table 4. 14 Results of parameter variation models

Result parameter	φ + 5		φ - 5
Total vertical displacement (T)			
Deformed shape			

<table>
<tr>
<td>Strain
(E-ZZ)</td>
<td></td>
<td>
</td>
<td></td>
</tr>
</table>

From these results it can be identified there is a very little or negligible variation in the behaviour of the pile.

b. Curves from result tables

Result concerning a particular vertical profile of the pile and of the soil around pile was extracted and from which a total displacement against depth was plotted. Figure 4.20 indicates the curves of both models together with the initial case along a vertical profile on the soil around pile and Figure 4.21 indicates the curves of both models together with the initial case along the pile.

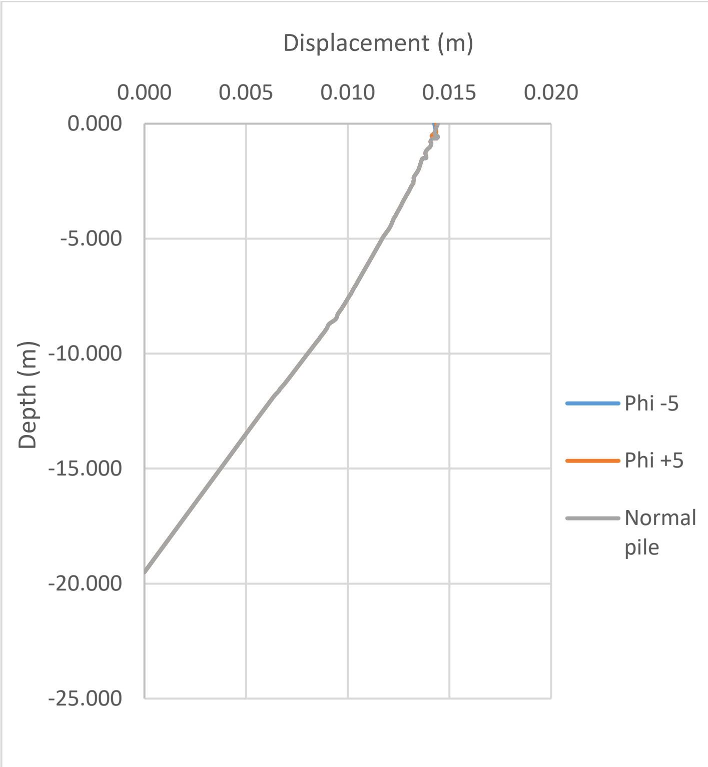


Figure 4. 20 Displacement against depth of soil profile

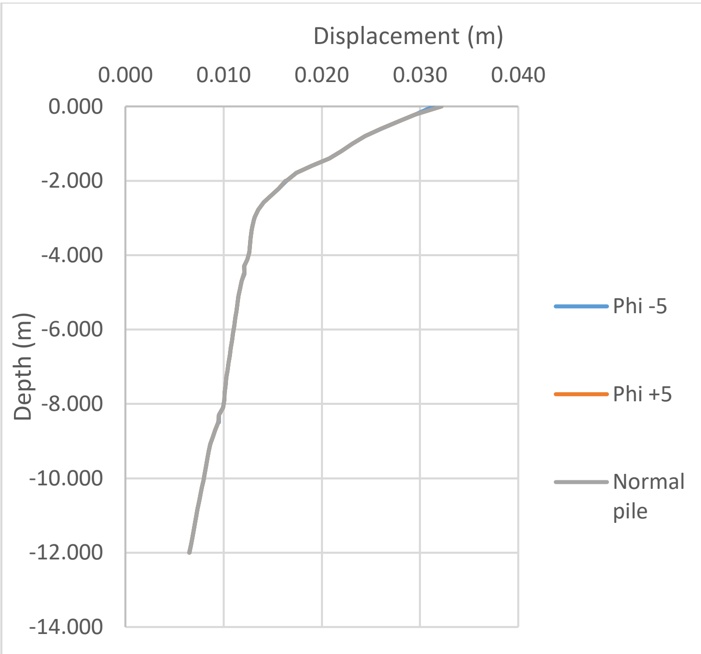


Figure 4. 21 Displacement against depth of pile

Looking from the different curves it is noted that the different curves superpose one another thereby signifying no considerable change in the behaviour of the pile by the variation of the friction angle of the bearing soil layer.

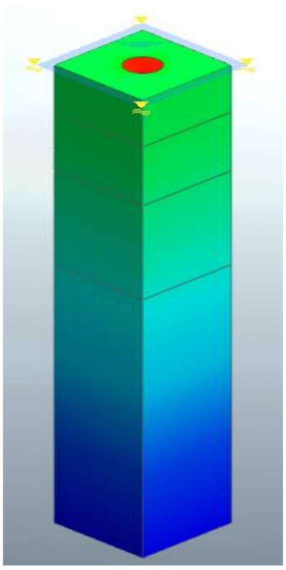
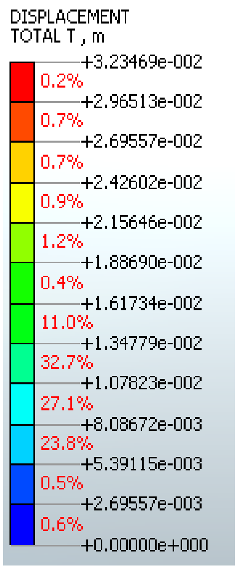
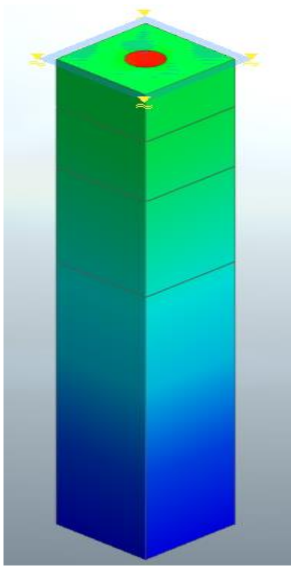
4.6.3.2. Variation of the cohesion

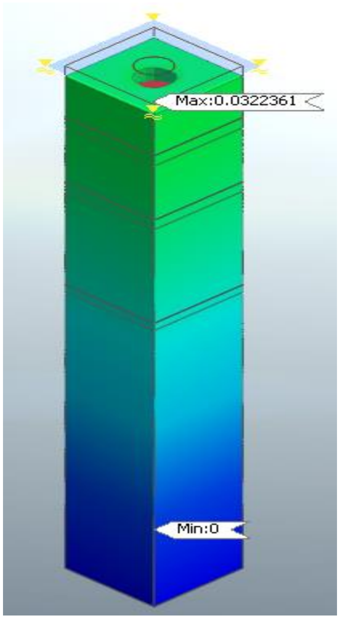
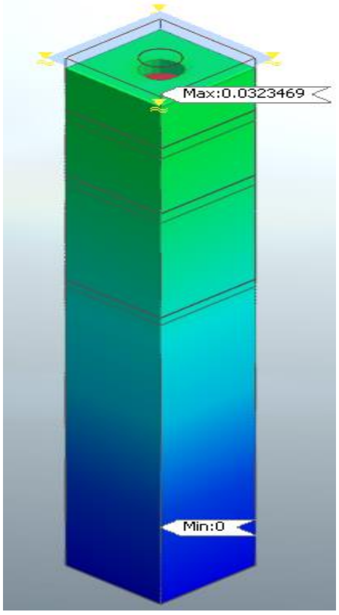
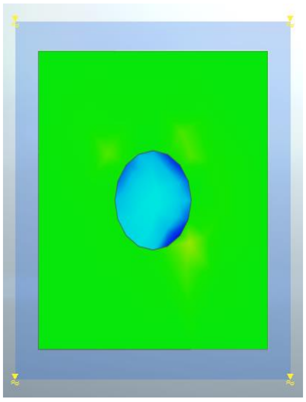
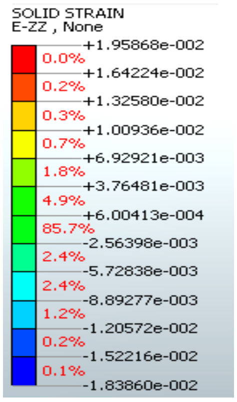
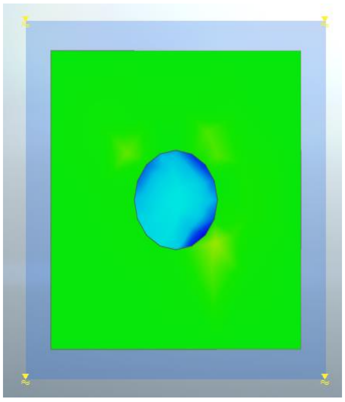
The influence of the variation of the cohesion of the soil was studied considering a variation of ± 10 in accordance with the parameter range of the soil type from the literature.

a. Graphical results

Some screen shots of the results from the model. Table 4.15 presents for both models the results of the total vertical displacement, deformed shape and vertical strain.

Table 4. 15 Results of parameter variation models

<table>
<tr><td>Result parameter</td><td colspan="2">C - 10</td><td>C + 10</td></tr>
<tr><td>Total vertical displacement (T)</td><td></td><td>
</td><td></td></tr>
</table>

Deformed shape		
Strain (E-ZZ)	 	

With these graphical details it is noticed a very slight difference in the behaviour of the pile.

b. Curves from result tables

Result concerning a particular vertical profile of the pile and of the soil around pile was extracted and from which a total displacement against depth was plotted. Figure 4.22 indicates the curves of both models together with the initial case along a vertical profile on the soil around pile and Figure 4.23 indicates the curves of both models together with the initial case along the pile.

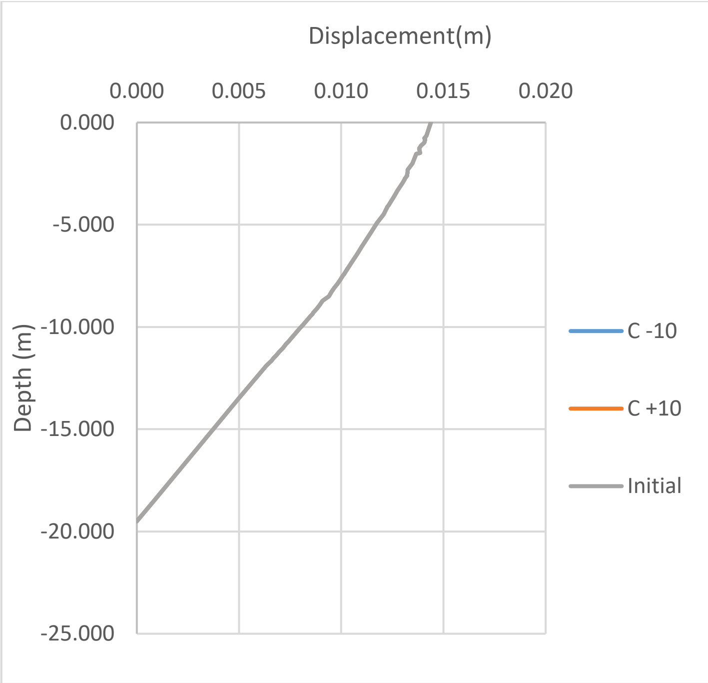


Figure 4. 22 Displacement against depth of soil profile

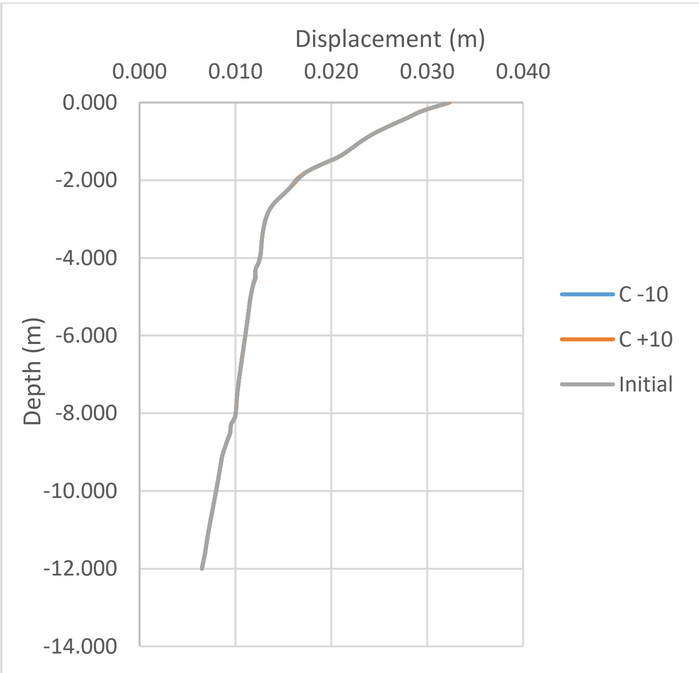


Figure 4. 23 Displacement against depth of pile

From all the different result, it is seen that the variation has very little or negligible effect on the behaviour of the pile as all the curves superpose one another.

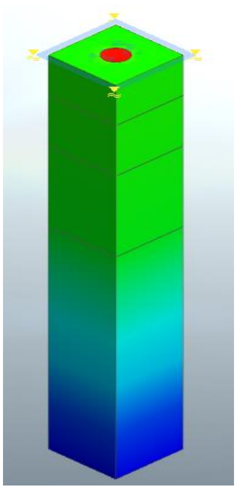
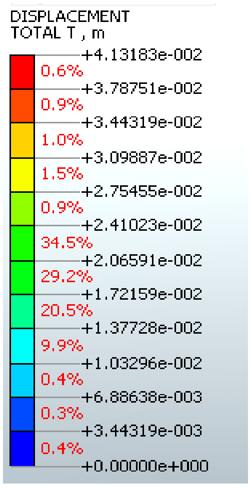
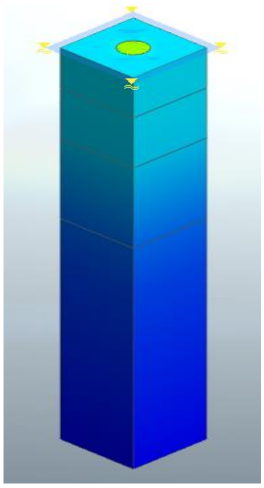
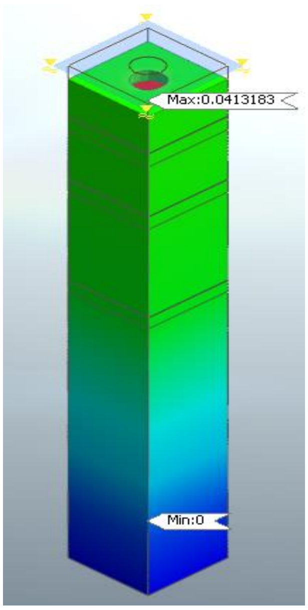
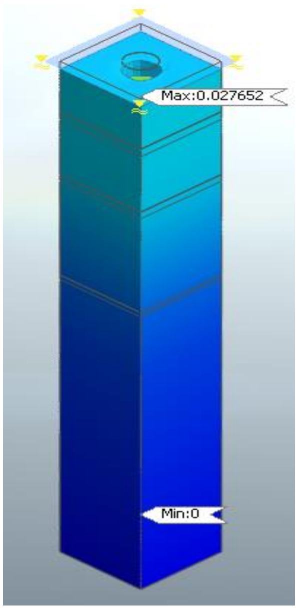
4.6.3.3. Variation of the Elastic modulus

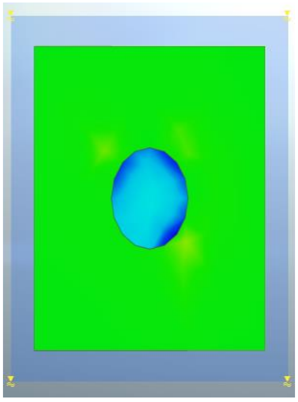
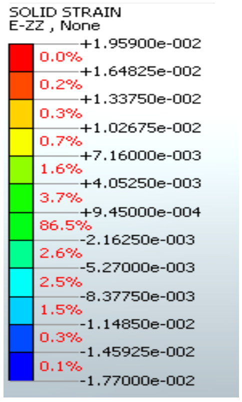
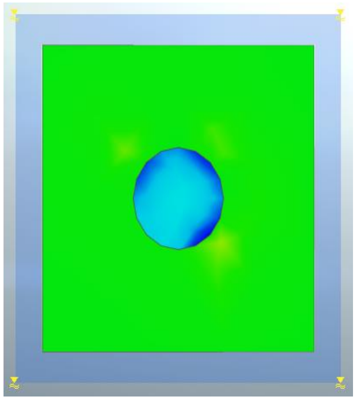
The influence of the variation of the elastic modulus was studied considering a variation of 0.5*E and 2*E of the soil.

a. Graphical results

Here are presented some screen shots of the results from the model. Table 4.16 presents for both models the results of the total vertical displacement, deformed shape and vertical strain.

Table 4. 16 Results of parameter variation models

<table>
<tr><td>Result parameter</td><td colspan="2">0.5*E</td><td>2*E</td></tr>
<tr><td>Total vertical displacement (T)</td><td></td><td></td><td></td></tr>
<tr><td>Deformed shape</td><td colspan="2"></td><td></td></tr>
</table>

Strain (E-ZZ)			

From these results it can be identified there is reduction of the displacement value as we increase the elastic modulus of the soil and an increase as we decrease the elastic modulus which can be explained by the equation of Christian and Carrier (1978) of settlement against modulus of elasticity which is an inverse relation.

$$S_i = \frac{Bq}{E}(1 - \nu^2)I_s \qquad 4.1$$

b. Curves from result tables

Result concerning a particular vertical profile of the pile and of the soil around pile was extracted and from which a total displacement against depth was plotted. Figure 4.24 indicates the curves of both models together with the initial case along a vertical profile on the soil around pile and Figure 4.25 indicates the curves of both models together with the initial case along the pile.

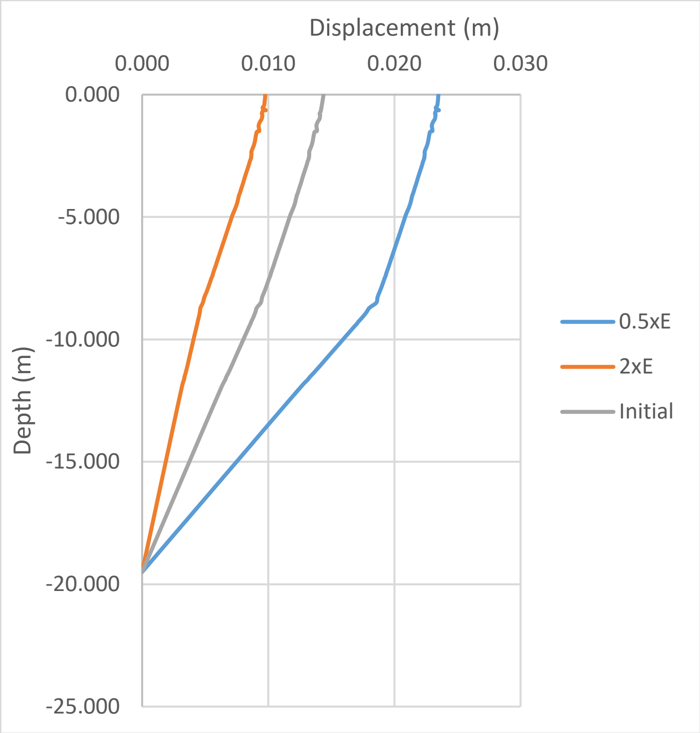


Figure 4. 24 Displacement against depth of soil profile

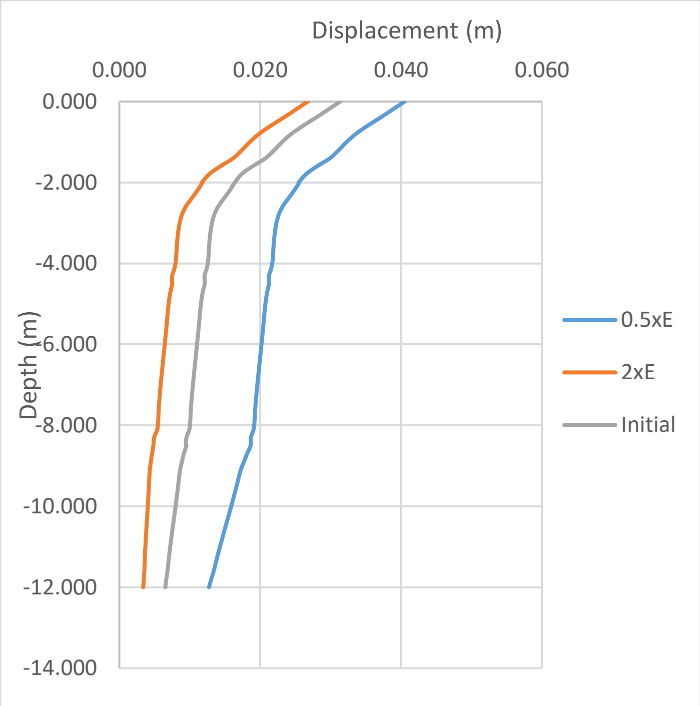


Figure 4. 25 Displacement against depth of pile

From all the result details it is well noted that the displacement of both pile and soil increases with the model of 0.5*E and decreases for the model of 2*E this can be related to the variation of the strength of the soil.

Though with the differences noticed from graphical results or curves, both models satisfy both the ULS (model converges) and SLS ($T < 0.05D = 5$ cm) conditions.

4.6.4. Effect of a defect on the pile

In this part even though we had from the low strain test results that the piles were all in conformity with structural integrity, a study of the influence of a supposed defect on the pile to verify the behaviour of the pile under certain default conditions were made and evaluated. The evaluation made were with respect to the position of the defect on the pile and the size of the defect.

4.6.4.1. Effect of the position of a defect

The position of the defect considered were at the midway of the different soil layers to understand the influence of each soil layer in the case of this defect.

a. The model

The model used was obtain by a similar procedure as in section 3.7 with a slight addition of cutting out a small volume from the pile solid. This volume is considerably small as the aim was to model a defect and evaluate the influence on the pile behaviour. Information on the volume of the defects is presented in Table 4.17

Table 4. 17 Size of the different elements used

Element	Volume (m^3)	%
Pile	9.425	-
5 cm inside void	0.114	1.21%
7 cm inside void	0.126	1.34%
Torous void	0.108	1.15%
Inclusion	0.221	2.34%

Figure 4.26 presents the different models used for the study of the defect position which can be

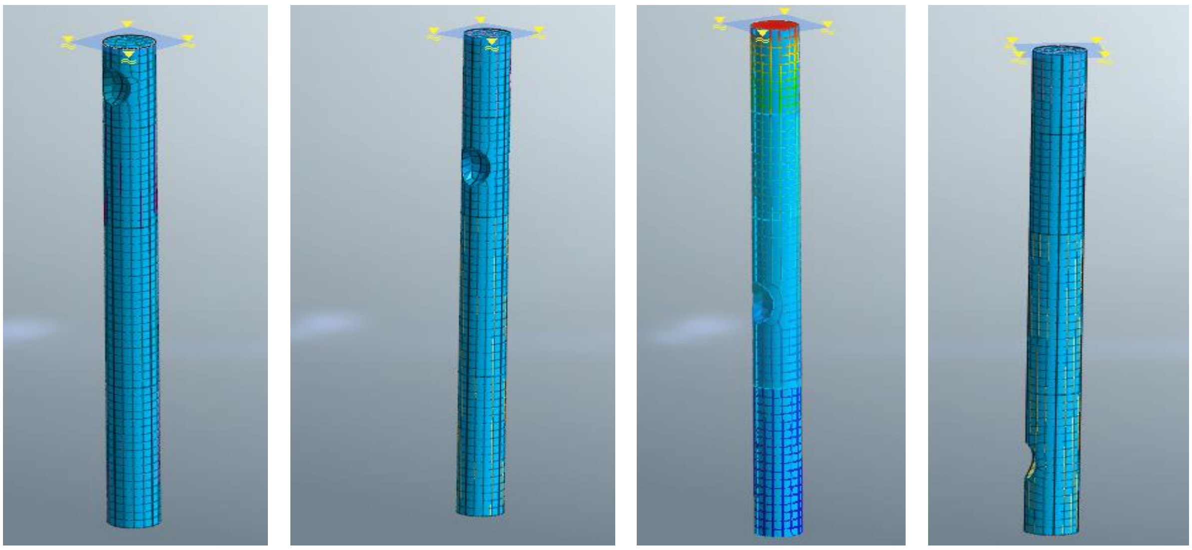


Figure 4. 26 Models of defective pile

used to simulate a case like that in Figure 4.27 which may have resulted due to dry chunks of concrete jamming with the reinforcement cage, creating a large air-filled void (Amir, 2015).

Figure 4. 27 An obviously defective pile (Amir, 2015)

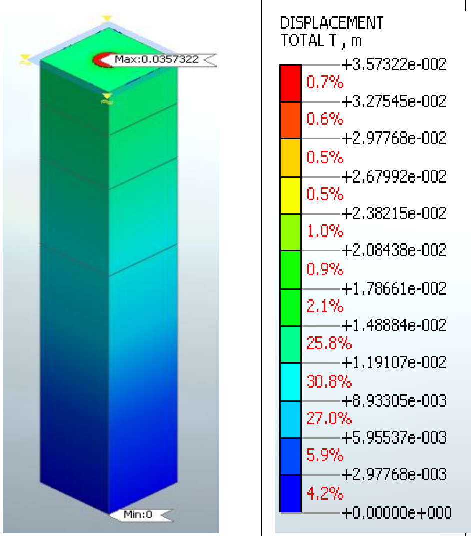
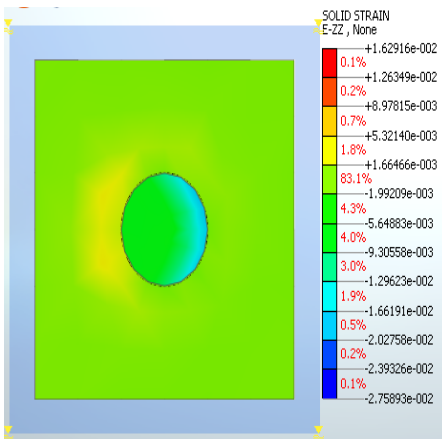
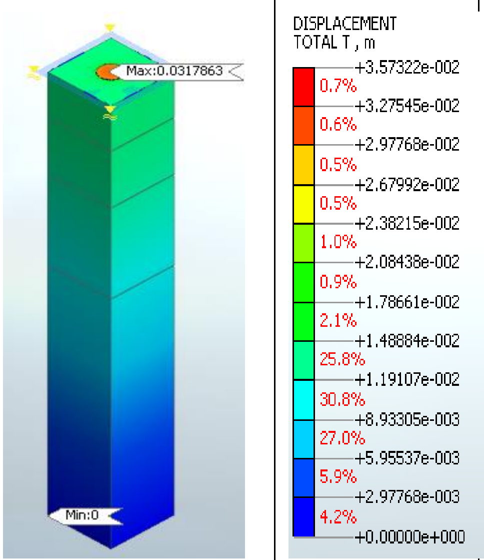
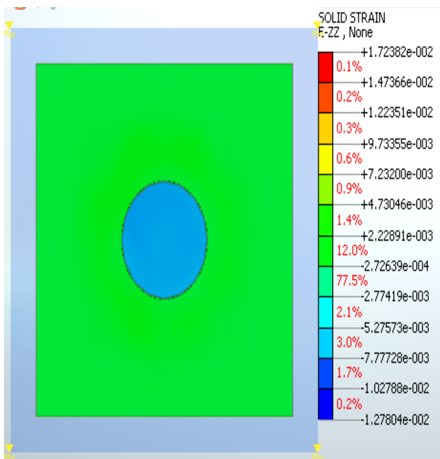
b. The results

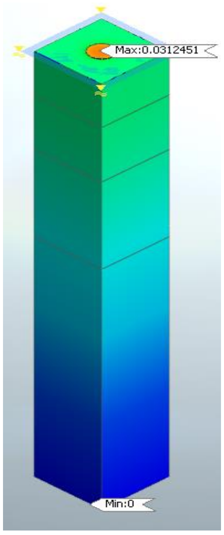
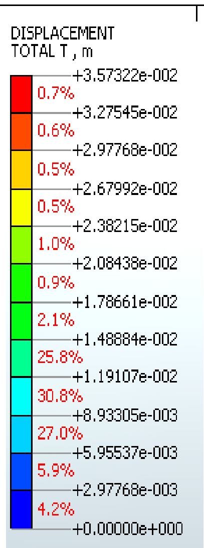
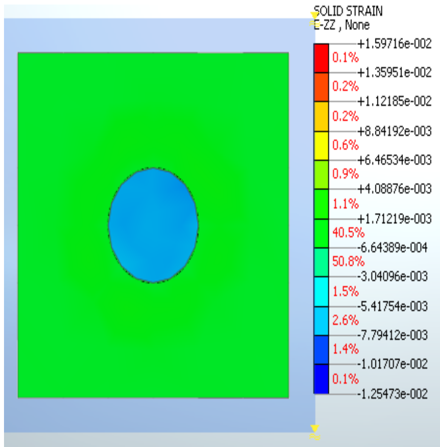
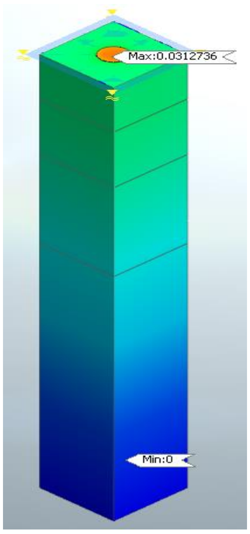
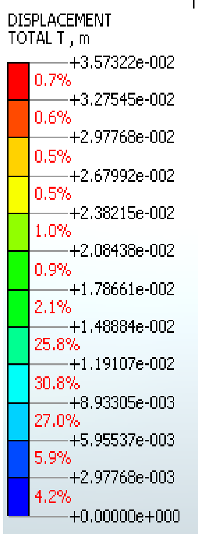
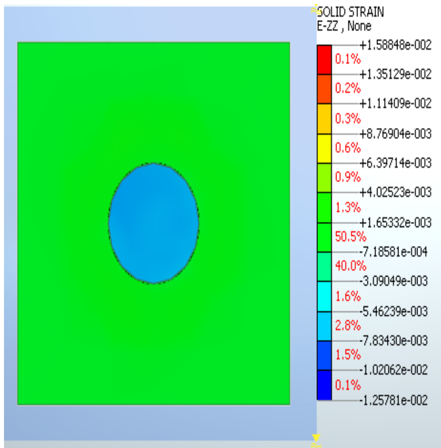
After performing the analyses, different results could be obtained to characterise the performance of the pile. The most essential of the results were presented either graphically or extracting tables from which curves were plotted.

i. Graphical results

Here are presented some screen shots of the results from the models. Table 4.18 presents for the different models the results of the total vertical displacement and vertical strain of the defect at different soil layers, the defect presented is that of volume 0.126 (1.34 % of pile volume).

Table 4. 18 Results of four pile models

<table>
<tr><th>Result parameter</th><th>Total vertical displacement (T) deformed shape</th><th>Strain (E-ZZ)</th></tr>
<tr><td>First layer</td><td>
</td><td>
</td></tr>
<tr><td>Second layer</td><td>
</td><td>
</td></tr>
</table>

<table>
<tr>
<td>Third layer</td>
<td>
</td>
<td>
</td>
<td>
</td>
</tr>
<tr>
<td>Fourth layer</td>
<td>
</td>
<td>
</td>
<td>
</td>
</tr>
</table>

The graphical details show a defect present at the first layer (crushed rocks) is the most critical (highest displacement) and the least critical is at the second layer (loose sand).

ii. Curves from result tables

Result concerning a particular vertical profile of the pile and of the soil around pile was extracted and from which a total displacement against depth was plotted. Figure 4.28 indicates the curves of the different models along a vertical profile on the soil around pile and Figure 4.29 indicates the curves of both models along the pile.

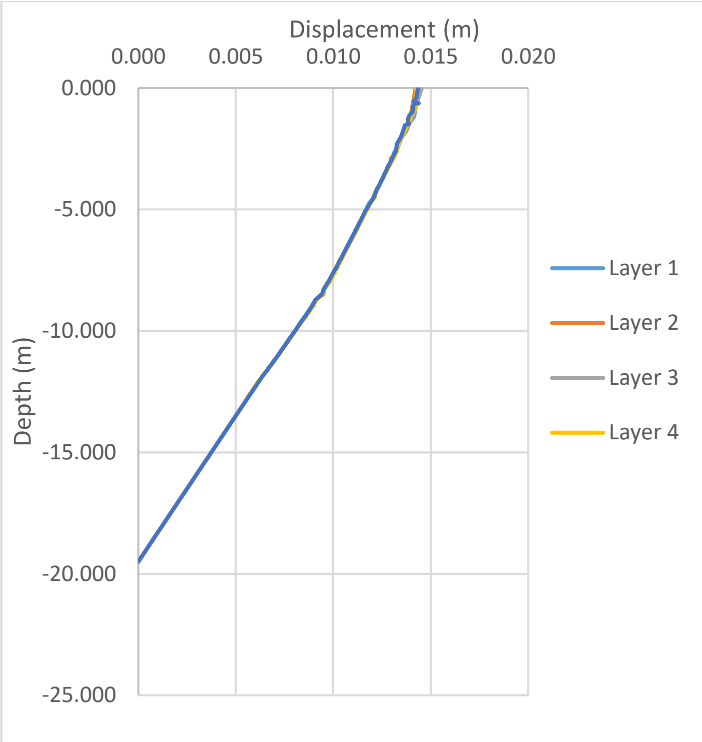


Figure 4. 28 Displacement against depth of soil profile

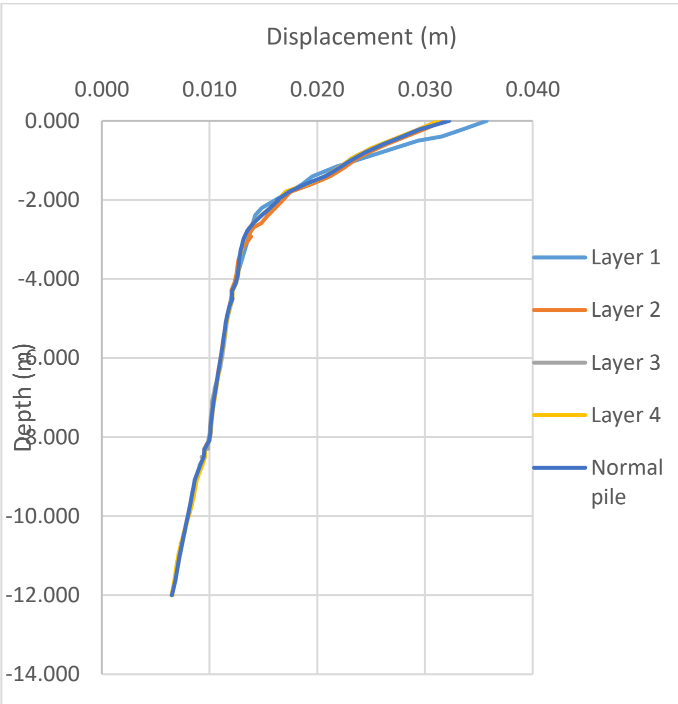


Figure 4. 29 Displacement against depth of pile

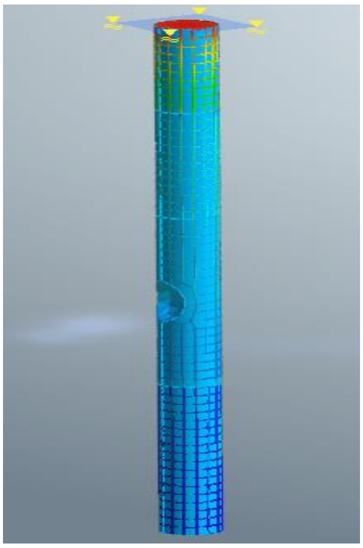
With the curves it can as well be noted that a defect at first layer has the highest of the displacement which can be explained as since this layer with highest friction angle material (41°), it contributes most to the resistance to movement of the pile and thus a loss of some contact of this material with the pile surface reduces the friction offered by the soil to the pile so a higher displacement of the pile than normal.

4.6.4.2. Effect of the type of defect

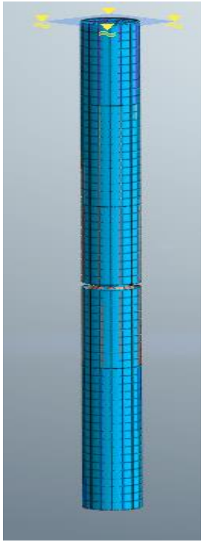
The different types of defect considered are, a void at the side of the pile, a void around the pile and an inclusion in the pile. The location of the defect considered is at the soil layer with maximum contact with the pile surface (Clayey sand)

a. The model

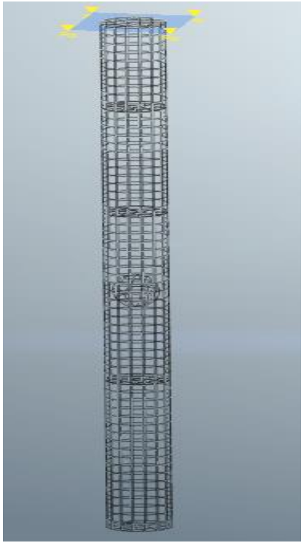
The model used was obtain by a similar procedure as in section 3.7 with a slight addition of cutting out a small volume from the pile solid, or replacing with another material a small volume of the pile solid. These volumes were considerably small as the aim was to model a defect and evaluate the influence on the pile behaviour. Information on the volume of the defects is presented in table 4.17. Figure 4.30 presents the different models used for the study of the defect type where Figure 4.30(a) can be used to simulate a case like that in Figure 4.27,



(a)



(b)



(c)

Figure 4. 30 Model for type of defect (a) void on the side of pile, (b) void all round pile (c) inclusion in pile solid

Figure 4.30(b) can be used to simulate a case like that in Figure 4.31 which may have resulted from an inflow of water or the use of slurry. Figure 4.30(c) can be as a result of inflow of soil into the pile during concreting

Figure 4. 31 A bored pile cast with bentonite (Amir, 2015)

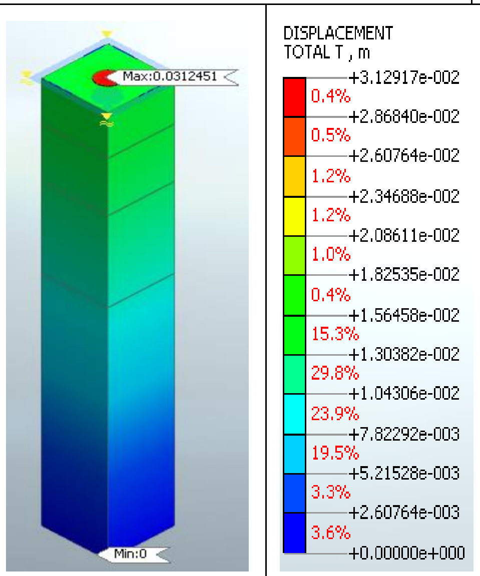
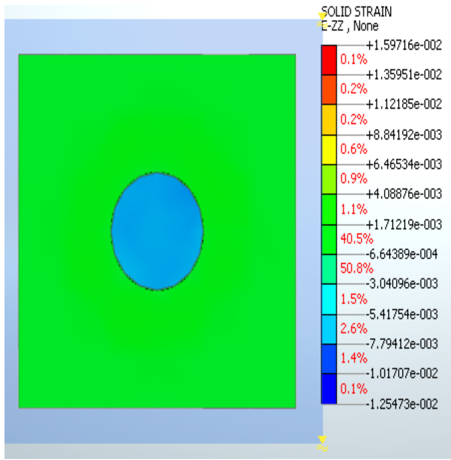
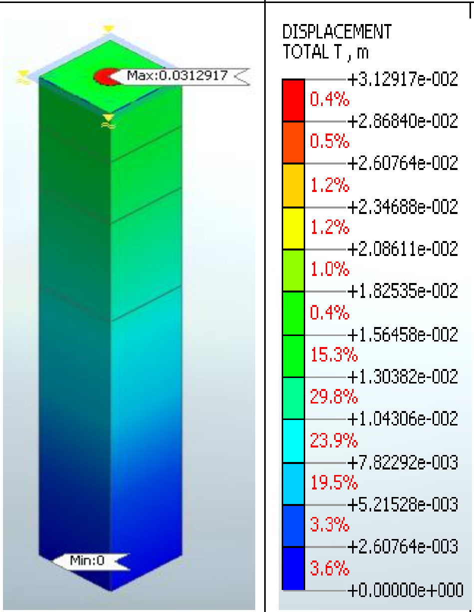
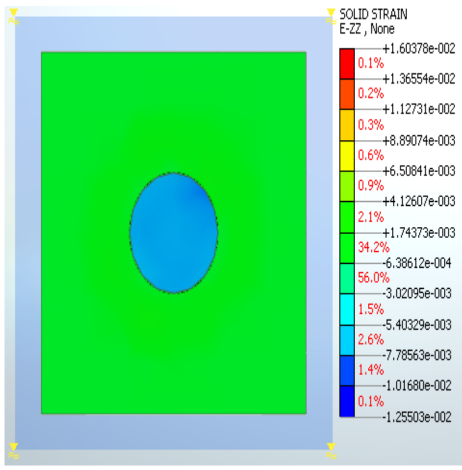
b. The results

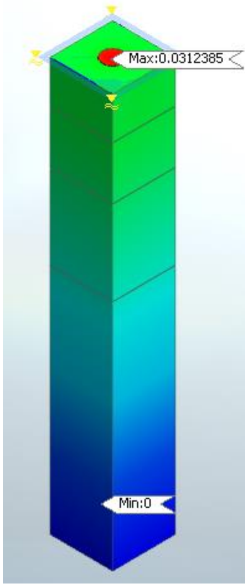
After performing the analyses, different results could be obtained to characterise the performance of the pile. The most essential of the results were presented either graphically or extracting tables from which curves were plotted.

i. Graphical results

Here are presented some screen shots of the results from the models. Table 4.18 presents for the different models the results of the total vertical displacement and vertical strain of the defect at the particular soil layer.

Table 4. 19 Results of three pile models

Result parameter	Total vertical displacement (T) deformed shape	Strain (E-ZZ)
0.126 m^3 void		
0.108 m^3 void		

<table>
<tr>
<td>0.221 m^3 inclusion</td>
<td>
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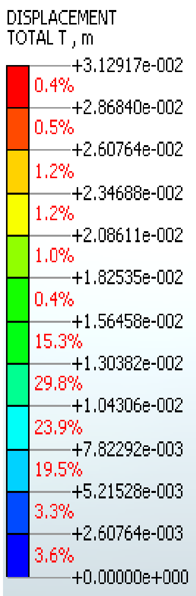
<td>
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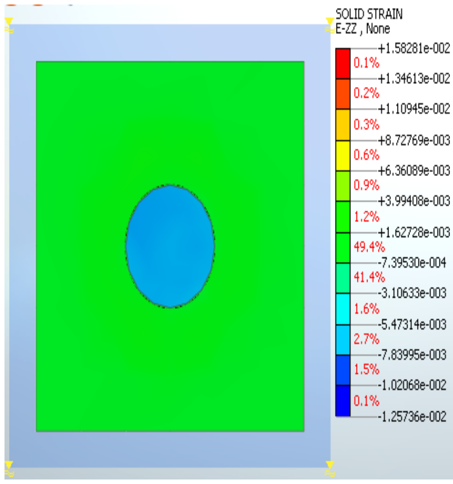
<td>
</td>
</tr>
</table>

From the graphical details, all defect types show very similar behaviours with almost negligible differences in displacement and strain values.

ii. Curves from result tables

Result concerning a particular vertical profile of the pile and of the soil around pile was extracted and from which a total displacement against depth was plotted. Figure 4.32 indicates the curves of the different models along a vertical profile on the soil around pile and Figure 4.33 indicates the curves of both models along the pile.

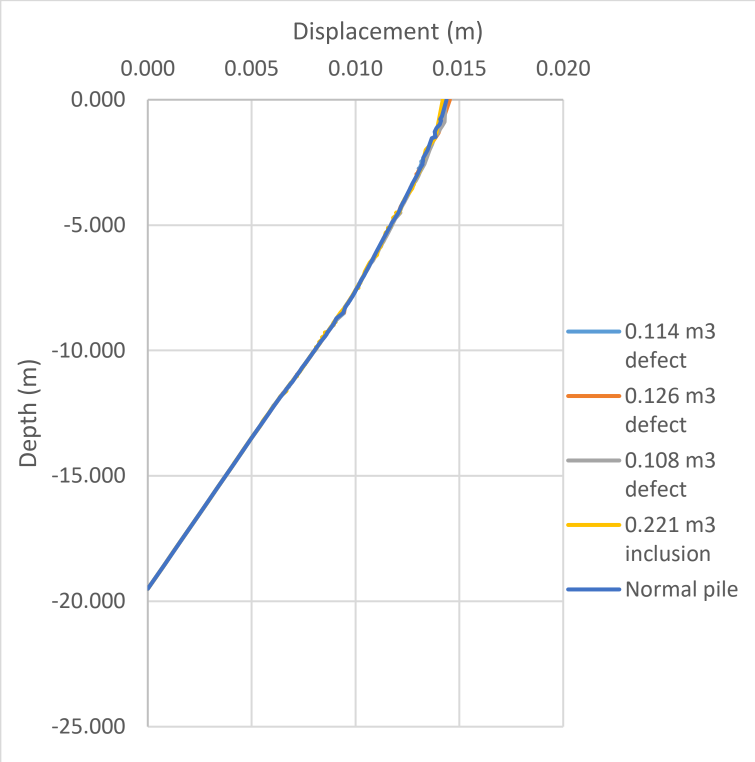


Figure 4. 32 Displacement against depth of soil profile

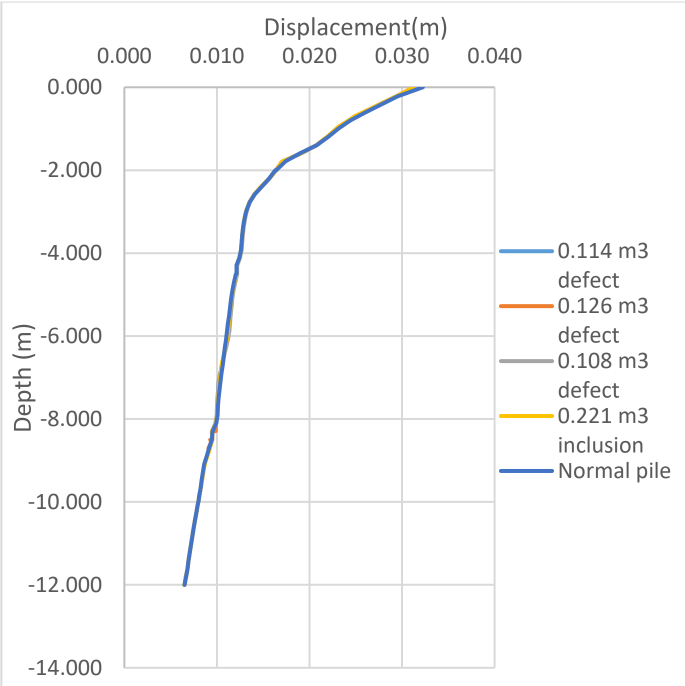


Figure 4. 33 Displacement against depth of pile

Comparing to the normal pile condition, it is shown by the curves that these selected defect have no considerate influence on the pile behaviour as the curves all superpose one another.

To indicate clearly the details at the tip of Figure 4.32 a zoom of the displacement was made with the aid of a bar chart (Figure 4.34) can be obtain to bring out the slight difference at a smaller scale that occurred at the soil profile considered.

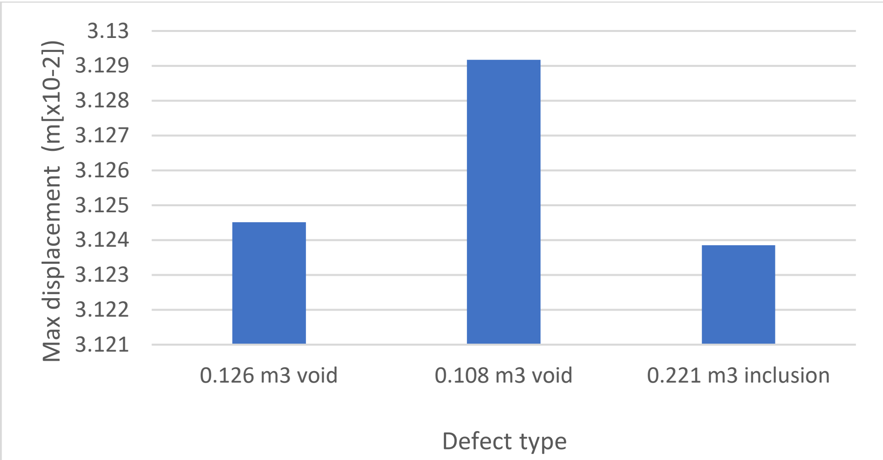


Figure 4. 34 A zoom detail of soil profile detail

This detail indicates the highest displacement obtained with a void of 0.108 m3 extracted from the pile around the shaft which can be noted to be the most critical defect that the pile can experience as there is a great loss of contact between the soil and the pile shaft so a less resistance to settlement as compared to other defects though of slightly higher volumes. A void as well is more detrimental than an inclusion which is a replacement of concrete by soil.

4.7. Application of pile testing in Cameroon

In this part it was of aim to evaluate the applicability of the pile testing techniques in Cameroon and this was approached by verifying the frequency of use of techniques and possibly the costs.

To achieve this a survey was carried out on different projects in which a pile foundation was adopted and some interview questions asked to some experts in pile foundations with regards to the testing techniques. Table 4.19 presents a summary of some different projects in the nation in which pile foundations were adopted and the type of testing carried out on these projects (information presented in French to indicate the exact state from which it was collected).

From the results obtained throughout this survey, the major techniques used are the low strain integrity test and the cross hole sonic logging tests, these are the two preferred due to their availability, cost, prescription by clients, and local experience. The pile load tests are rarely used due to their costs and technology involve for the application.

Table 4. 20 Some different pile projects in Cameroon and testing method used

Projet	Ville	Type de pieu	Type du sol	No Existant	No Testé	Geometrie	de charge	Essai d'integrité	Prix Estimé par pieu
Pont sur le fleuve Lom à Tourake	EST	Pieux forés	argileux, Schisteux	22	22	D = 1 m , L =22 m	Non	Auscultation Sonique	-
Pont sur la LOBO	SUD	Pieux forés	roche altéré	-	-	D = 0.5 m , $12 \leq L \leq 22$ m	-	-	-
CONSTRUCTION D'UNE MAÏSERIE	CENTRE	Pieux forés	argileux, Schisteux et roche altérée	15	15	D = 1 m , $17 \leq L \leq 19$ m	NON	Auscultation Sonique et par Impedance	500 000 fcfa/pieu
TRAVAUX D'AMENAGEMENT DE LA ROUTE LENA-SENGBE-LOT 5 ET SENGBE- TIBATI-LOT 6 OUVRAGE 17	ADAMAOUA	Pieux forés	argileux, Schisteux et roche altérée	12	12	D = 1 m , $6 \leq L \leq 15$ m	NON	Auscultation Sonique et par Impedance	500 000 fcfa/pieu
TRAVAUX D'AMENAGEMENT DE LA ROUTE LENA-SENGBE-LOT 5 ET SENGBE- TIBATI-LOT 6 OUVRAGE 17	ADAMAOUA	Pieux forés	argileux, Schisteux et roche altérée	6	6	D = 1m , $6 \leq L \leq 16$m	NON	Auscultation Sonique et par Impedance	500 000 fcfa/pieu
TRAVAUX D'AMENAGEMENT DE LA ROUTE LENA-SENGBE-LOT 5 ET SENGBE- TIBATI-LOT 6 OUVRAGE 16	ADAMAOUA	Pieux forés	argileux, Schisteux et roche altérée	6	6	D = 1 m , $6 \leq L \leq 16$ m	NON	Auscultation Sonique et par Impedance	500 000 fcfa/pieu
TRAVAUX D'AMENAGEMENT DE LA ROUTE LENA-SENGBE-LOT 5 ET SENGBE- TIBATI-LOT 6 OUVRAGE 15	ADAMAOUA	Pieux forés	argileux, Schisteux et roche altérée	6	6	D = 1 m , $7 \leq L \leq 14$ m	NON	Auscultation Sonique et par Impedance	500 000 fcfa/pieu
CONSTRUCTION D'UN RESERVOIR DE 10 000 M^3 AU DEPOT SCDP DE NSAM-YAOUNDE	CENTRE	Pieux forés	argileux, Schisteux et roche altérée	8	8	D = 1 m , $11 \leq L \leq 15$ m	NON	Auscultation Sonic et par Impedance	500 000 fcfa/pieu
Projet de l'extension d'usine agroalimentaire	LITTORAL	Pieux forés	argileux, peu sableux	152	68	D = 0.8 m , L =18 m	Non	Auscultation Sonique et PIT	-
Projet de Création usine de cosmétique Bonaberi	LITTORAL	Pieux forés	argileux	71	71	D = 1 m , L =12 m	Non	Essais PIT	-
Projet de l'extension d'usine brassicole	LITTORAL	Pieux forés	argileux, sableux	60	60	D = 0.8 m , L =23 m	Non	Auscultation Sonique et PIT	-
Construction du stade de Japoma	LITTORAL	Pieux forés	Argile sableuse	2000	2000	D = 0.8 m , $9 \leq L \leq 22$ m	Non	Essais PIT	-
Batchenga-Ntui avec la construction d'un pont sur le fleuve Sanage à Nachtigal	CENTRE	Pieux forés	roche altérée	60	60	D = 1000 mm , $4.5 \leq L \leq 10$ m	Nom	Auscultation sonique	70620/ml

Conclusion

The main objective of this chapter was to present and interpret the results of the analytical and numerical design of a pile, the quality control test results and interpretation, the prediction of the behaviour of the pile in a defective condition and the analysis of the applicability of pile testing techniques in Cameroon. To attain this a general presentation of the site focusing on its geographical location, climate, relief, vegetation, geology and hydrology was made. The second part focuses on the presentation of the project and the description of the study area under consideration, this was then followed by the presentation of the data collected. After which the results for the analytical design was presented and interpreted. Results from pile testing, numerical simulation and the prediction with respect to slight defect were interpreted. Finally the application of pile testing techniques in Cameroon was analysed.

GENERAL CONCLUSION

Pile foundations which are foundations used for very complex and important structures happens to be very critical in terms of their performance which should always be ensured making it important to carry out a quality control of these foundations.

The main objectives of this work was to perform a quality control of a pile foundation and to assess the influence of a slight defect on the pile foundation. In order to achieve these objectives, a review on pile foundation being the most used of deep foundations was carried out along with the study of the soil as an important element in foundation engineering, the design methods of this foundation and their testing. This was followed by the presentation of the research methodology used in order to conduct the study. Finally, following the methodology, the result obtain from the methodology was presented and interpreted.

However, the methodology used in this study consisted of making a general site recognition, a visit to the site for the collection of data, witnessing the pile installation process as well as the pile testing method, carrying out an analytical pile design and then a numerical simulation with the help of the special FEM software MIDAS GTS NX.

Concerning the different results obtained the conclusions (observations) derived from the study are:

- The analytical design made indicates the bearing capacity of the pile was verified for all the design approaches as well as the settlement and structural design, with a proposed possible reinforcement of 9Φ20 spaced at 300mm apart for the longitudinal bars and 6 mm wire mesh spaced at 400mm for transversal bars reduced by a factor of 0.6 near lapped joints.
- The numerical design made indicates the pile was verified with a remark to consider the pile-soil interaction (interface) during design as the model without an interface indicated an over estimation of the connection between pile and soil thus reducing the settlement real settlement value.
- The parametric analysis made indicated the variation of the friction angle and cohesion with the model had a negligible variation of the pile behaviour where as a

variation of the elastic modulus had a significant variation of the pile behaviour which can be explained with the fact that as the stiffness of the soil increases the more its resistance and thus less deformation.

- The pile integrity test results (LSIT) carried out on site indicates all the piles were made of homogeneous concrete with no anomalies and thus are of good quality.
- The study of the influence of a slight defect indicated concerning the location of a defect, a defect closest to the pile head is most critical than those far away from pile head and concerning the type of defect in a particular location, a void around the shaft is most critical than a soil inclusion of void on one corner of the shaft though of slightly larger volume.
- The study of the influence of the defect did not compromise the capacity of the pile as the settlements were less than the limit (5 cm) and could be explained by the fact that the defect volumes considered were very small to influence the capacity of the pile according to the research of Zhussupbekov et al (2021) which says field test results confirm that piles with defects over 20% of pile volume have a low bearing capacity and are not allowed for exploitation.
- The survey results for pile testing techniques in Cameroon indicates the major techniques used here in Cameroon are the LSIT and the CSL, which are the 2 preferred due to their availability, cost, prescription by clients, and local experience. It was noted the pile load test is rarely used due to the cost and technology involve for the application.

In order to ensure an objective continuity of this study, a few perspectives for future research are being proposed:

- The Mohr-Coulomb model, which is considered as a first approximation of the soil behaviour, is applied in this study. So, advanced soil models such as the hardening soil model can be used to simulate the soil layers to better capture the real behaviour of these materials.
- More parameters should be taken into account for parametric analysis aimed at calibration of a model

- Seismic loading application also can be included in the future study in other to determine the effect of the earthquake on the foundation.
- Numerical prediction of the pile load test from a given integrity test result interpretation

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APPENDICES

Appendix 1 Partial factors on actions (γ_F) or the effect of actions (γ_E)

Action		Symbol	Set	
			A1	*A2*
Permanent	Unfavourable	γ_G	1,35	1,0
	Favourable		1,0	1,0
Variable	Unfavourable	γ_Q	1,5	1,3
	Favourable		0	0

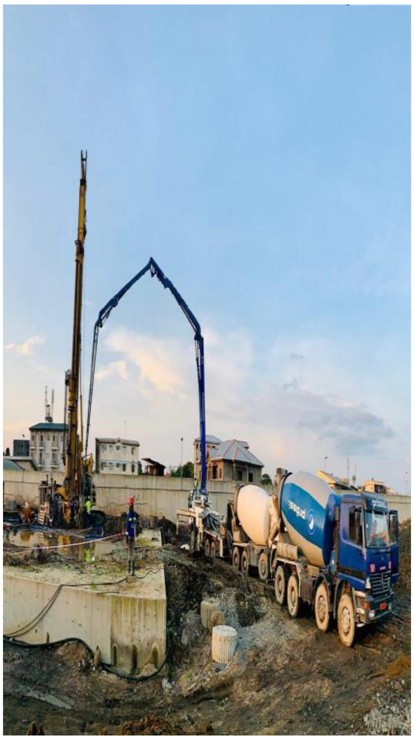
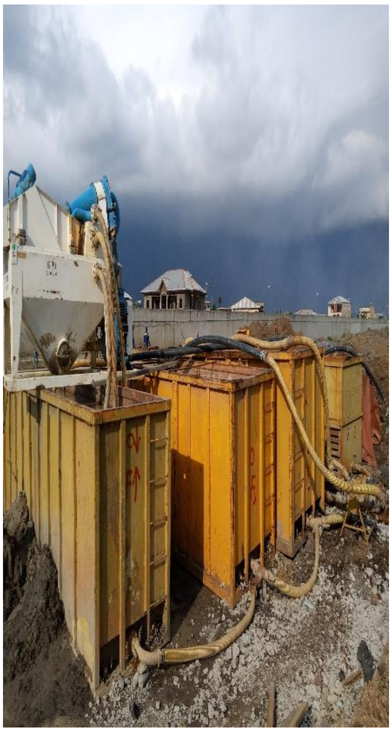
Appendix 2 Partial resistance factors (γ_R) for bored piles

Resistance	Symbol	Set			
		R1	*R2*	*R3*	*R4*
Base	γ_b	1,25	1,1	1,0	1,6
Shaft (compression)	γ_s	1,0	1,1	1,0	1,3
Total/combined (compression)	γ_t	1,15	1,1	1,0	1,5
Shaft in tension	$\gamma_{s;t}$	1,25	1,15	1,1	1,6

Appendix 3 Formulae of settlement from in-situ test

Test	Formula	Authors
CPT	$s = C_1 C_2 q \sum_{i=1}^{n} \frac{I_{z,i}}{E_i} \Delta z_i$	Schmertmann et al.
SPT	$s = C_1 C_2 C_3 [(q - 2\sigma'_{v0}) B^{0.7} I_c]$	Burland &Burbidge

Appendix 4 Installation images

Appendix 5 Testing

Appendix 6 Courbe du Ménard