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CRITICAL STUDY OF COMPACTION GROUTING AS A SOIL IMPROVEMENT METHOD IN THE DOUALA – BASSA INDUSTRIAL ZONE

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

Curriculum: Geotechnics

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DEDICATION

In memory of my Grand-Father,

Rev Samuel LOUOKDOM

(1941 – 2015)

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In the name of **GOD***, the ultimate source of knowledge and strength.*

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ACRONYMS

AASHTO	American Association of State Highway and Transportation Officials
ASCE	American Society of Civil Engineers
ASTM	American Society for Testing and Materials
CG	Compaction Grouting
CPT	Cone Penetration Test
CPTu	Cone Penetration Test – piezocone
CSW	Continuous Surface Waves
DEM	Division of Emergency Management
DIN	Deutsches Institut für Normung
DPH	Dynamic Probing Heavy test
EMI	Electro-Magnetic Induction
ERT	Electrical Resistivity Tomography
FBG	Fibre Bragg Grating
GPR	Ground Penetrating Radar
MASW	Multi – Channel Analysis of Surface Waves
OFS	Optic Fibre Sensor
SASW	Spectral Analysis of Surface Waves
SCPT	Seismic Cone Penetration Test
SPT	Standard Penetration Test
USA	United States of America
UV	Ultra violet
VES	Vertical Electrical Sounding

SYMBOLS

Cc	Coefficient of consolidation
CC	Coefficient of curvature
Cs	Coefficient of secondary consolidation or swelling index
CU	Coefficient of uniformity.
Cv	Coefficient of primary consolidation
CZ	Coefficient of dissymmetry
$\mathbf{e_0}$	Initial void ratio
$\mathbf{k_x}$	Permeability
LL	Liquid limit
PI	Plasticity index
Sr	Saturation degree
Su	Shear strength
γd	Dry weight
γs	Particles specific weight
γsat	Bulk density
$\boldsymbol{\sigma'_p}$	Pre-consolidation pressure
φ	Friction angle
ωc	Natural water content

ABSTRACT

The aim of this work is to study and improve the compaction grouting by evaluating its effectiveness and performance as an improvement method for soils located in the Douala – Bassa industrial zone. In fact, as many other soil improvement techniques, the verification of compaction grouting highly relies on the post-construction monitoring phase. The studied area has known some subsidence and sinkholes which occurred in four points in October 2018. To understand the causes of the phenomenon, investigation has been made through a structural damage survey, geotechnical (CPT, DPH and DST) and geophysical (ERT) tests to obtain soil stratigraphy, underground water level and soil parameters including friction angle and cohesion. To tackle this problem, the method consisted firstly of the design and the application of the compaction grouting, chosen among the various underpinning techniques to fill underground cavities detected, improve the soil bearing properties and reinforce the foundations. Secondly, the reliability assessment of the used monitoring techniques of injection parameters and ground uplifts during the grouting. Since monitoring can be used to determine any required maintenance of an important structure following a catastrophic geo-hazard, the present critical analysis of the grouting concluded that (1) the adequate monitoring of all the injection parameters and vertical displacements in each stage improve the compaction grouting performance, (2) the post-grouting density verification through direct and indirect methods permit to assess the compaction grouting effectiveness, and (3) the application of the new CG method can reduce the ground upheaval quantity by more than 90%. Hence, these results can be applied to establish better quality control and quality assurance programs of compaction grouting execution.

Keywords: sinkhole, soil improvement, compaction grouting, monitoring.

RESUME

L'objectif de ce travail est d'étudier et améliorer l'injection solide en évaluant son efficacité et sa performance comme méthode d'amélioration des sols situés dans la zone industrielle de Douala – Bassa. En effet, comme plusieurs autres techniques d'amélioration des sols, la vérification de l'injection solide repose fortement sur la phase d'instrumentation après l'exécution. La zone étudiée a connu des affaissements et des dolines qui se sont produits en quatre points en Octobre 2018. Pour comprendre les causes du phénomène, une investigation a été faite à travers une enquête sur les dommages structurels, des tests géotechniques (CPT, DPH et DST) et géophysiques (ERT) pour obtenir la stratigraphie du sol, le niveau d'eau souterrain et les paramètres du sol, notamment l'angle de friction et la cohésion. Pour faire face à ce problème, la méthodologie employée a consisté, premièrement, en la conception et l'application de l'injection solide, choisie parmi les différentes techniques de reprise en sous-œuvre pour remplir les cavités souterraines détectées, améliorer les propriétés portantes du sol et renforcer les fondations. Deuxièmement, l'évaluation de la fiabilité des techniques d'instrumentation utilisées pour contrôler les paramètres d'injection, les soulèvements du sol et les déformations sur les structures durant l'injection. Puisque l'instrumentation peut être utilisée pour déterminer tout entretien nécessaire d'une structure importante suite à une catastrophe imprévisible, la présente analyse critique de l'injection solide a abouti aux résultats selon lesquels (1) l'instrumentation adéquate de tous les paramètres d'injection, les déplacements de sols et les déformations structurelles à chaque étape améliore la performance de l'injection solide, (2) la vérification de la densité après l'injection par des méthodes directes et indirectes permet d'évaluer l'efficacité de l'injection solide, et (3) l'application de la nouvelle méthode d'injection solide peut réduire la quantité de soulèvement du sol de plus de 90%. Ces résultats peuvent être exploités pour établir de meilleurs programmes de contrôle qualité et d'assurance qualité de l'exécution de l'injection solide.

Mots-clés : doline, amélioration de sol, injection solide, instrumentation.

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INTRODUCTION

One of the most vital duties of an engineer is to preserve life and nature by using safe designs that take into account environmental standards and by monitoring the performance of structures following design criteria. Although all the design limit states have been verified, the underground conditions can modify soil hydraulic and bearing properties. This causes ground instability phenomena which is risky for structures. Therefore, finding the adequate soil improvement method regarding the in-situ soil conditions, the site access, the available technique and the economical aspect remains a challenge to geotechnical engineers. As many other existing soil improvement techniques, the compaction grouting aims to improve bearing capacity, compressibility and hydraulic properties of soils. Developed over the last 50 years, it means of densifying soft and weak soils under existing structures as well as a means to re-level the structure itself once settlement had occurred. It acts by compacting the soil and its surrounding with a denser grout. The technique relies on post-construction monitoring which can be used to determine any required structure maintenance following a catastrophic event. Numerous different techniques and instruments can be employed for such a purpose with different requirements producing different results.

Although its recent evolution, in Africa, geotechnical engineers are not well aware about the practice of compaction grouting which is highly experience based. The Douala – Bassa industrial zone in Littoral region, Cameroon, provided a good opportunity to evaluate design and execution procedures in CG. This first CG application in our country concerns the four studied areas which include roadways, residences, commercial buildings and chemical facilities such as storage tanks. Those areas made of soft clays and loose silty sands had been undergoing significant vertical displacements and deformations on surrounding structures. A CG program was designed to improve the bearing capacity, compressibility and hydraulic properties of these soils. As other soil improvement techniques, CG has no design standards and only relies on the failure risk control in post-construction phase. Since the CG is applied to soils beneath important infrastructures, the challenge is therefore to establish a quality assurance and control program which could make those affected structures less risky to collapse.

When dealing with soil improvement techniques such as CG, the huge challenges are:

- to obtain a good low-mobility grout mix rheology
- to control the ground collapse due to hydro-fracture during grouting
- to reduce the potential effects of ground uplift on structures
- to verify that the ground is well compacted or more densified

The main objective of this research work is to study and improve the CG design and execution method applied in the Douala – Bassa industrial zone, this by critically analysing good practice, technical instructions and literature historic cases. The objectives to attain by this research work are to:

- Design and execute on field the compaction grouting
- Verify the compaction grouting densification effectiveness
- Avoid the soil hydro-fracture and reduce ground uplift during grouting
- Monitor the compaction grouting effects during and after the injection

To answer these questions, the present critical analysis of the compaction grouting will be segmented in four chapters. The first chapter will deal with the soil formation processes, the sinkholes collapse schemes and the underground cavities detection techniques. Then, the next chapter will present the structural deformations and monitoring techniques, the underpinning techniques and all the theoretical and practical knowledge on compaction grouting. The chapter three will elaborate the methodology used to achieve this research work is detailed. Finally, the last chapter will present the collected data, the compaction grouting execution results, their limitations and the post – grouting monitoring proposals which will reveal that the treatment was successful.

CHAPTER 1: LITERATURE REVIEW

Introduction

To the civil engineer, soil is any un-cemented or weakly cemented accumulation of mineral particles formed by the weathering of rocks. The ground can be deformed by various hazards as sinkholes. Ground instabilities are depressions in the ground surface that form where subsurface drainage has removed rock or sediment to form a void (Gutiérrez et al., 2014). Such geohazard can be dangerous to life and property as it is unpredictable. It causes damages to existing infrastructures and produces the building foundation collapse; and in the worst cases, injury or loss of life. This chapter therefore deals with the formation processes, the detection methods of underground cavities and other karst features before and after the construction of any civil structures in order to help decision makers to perform the most appropriate and suitable programs of land use and development.

1.1. Soil formation processes

Civil engineers always deal with either soil or rock, and often both. Based on its formation, a soil can be subjected to chemical and/or physical alterations. Thus, to solve any geotechnical engineering problem, inorganic soils genesis is therefore a required knowledge.

1.1.1. Terminology on soils

Understanding soils, their chemical composition, their physical characteristics and their occurrence processes as well as possible is necessary before being used in construction.

1.1.1.1. Soil constituents

The typical soil consists of approximately 45% mineral, 5% organic matter, 20-30% water, and 20-30% air. These percentages are only generalizations at best. In reality, the soil is very complex and dynamic. The composition of the soil can fluctuate on a daily basis, depending on numerous factors such as water supply, cultivation practices, and/or soil type.

1.1.1.2. Formation factors

Soils form the upper part of the earth crust and its formation processes depend on various factors which can be classified as physic-thermal and chemical factors. Physical factors such as overburden stress variation, erosion, freezing, weathering, towing and thermal expansion of rocks reduce the particle sizes without any alteration of the original chemical of the parent rocks. Besides, the main agents responsible for chemical are hydration, carbonation and oxidation (Budhu, 2015). Often, chemical and physical weathering take place in concert.

1.1.1.3. Characteristics of soils

Characteristics of soils are multiple and depend on the studied parameters. Indeed, soil can be characterized following their physical aspect (fine- or coarse-grained soils), their minerals (clay minerals and quartz) and their chemical composition (carbonates, sulphides, silicates, oxides etc.). Also soils characteristics can be given by geotechnical uses (soft compressible and stiff soils) for construction purposes.

1.1.1.4. Types of soils

The soil types are generally defined based on soil textures. Since texture depends on the appearance or feel of a soil, the two common descriptive terms are sands and clays.

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. The response of fine-grained soils to loads, known as the mechanical behaviour, depends on the type of predominant minerals present.

1.1.1.5. Classification of soils

Depending on their origins, their sizes, their types, and their uses, soils have many classification systems. 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 1.1, four systems are presented. These are the Unified Soil Classification System (USCS), the American Society for Testing and Materials (ASTM) (a modification of the USCS system), the American Association of State Highway and Transportation Officials (AASHTO), and the British Standards (BS).

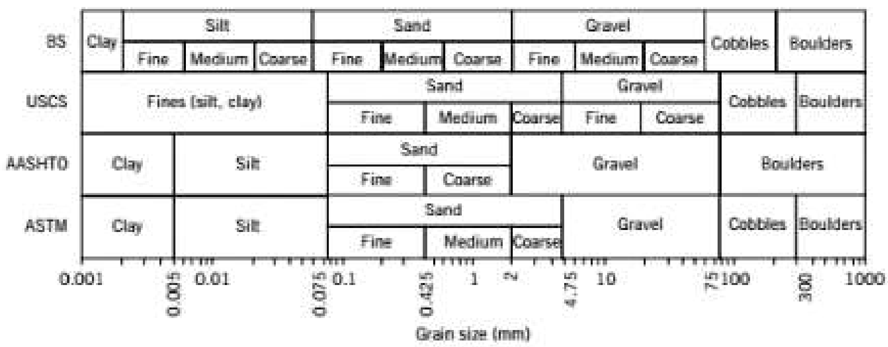


Figure 1.1. Comparison of 4 soils classification systems based on particle size (Budhu, 2015)

1.1.1.6. Uses of soils

Soils have different uses depending on humans and regions where they are located. Indeed in-exhaustively, they are mainly used for agricultural, for mining and civil engineering purposes as a bearing support and as a construction material.

1.1.1.7. Types of ground instability

Due to various causes and mechanisms, ground deformations can be induced or natural. Induced deformations are those caused by men activities whereas natural ones are not (Newton, 1981). Since those deformations are mostly unpredictable; they are therefore also called geohazards. They can be

- Landslide: is the movement of mass earth and/or mobile ground downslope.
- Sinkhole: is a sudden apparition of an underground cavity when the overburden stress is no longer supported.
- Subsidence: is a gradual downward settling or sudden sinking of the ground surface with little or no horizontal motion.
- Creep: is a long lasting slow and permanent deformation affecting loose material and rock.

1.1.2. Physical alteration process

Generally preceding chemical weathering, physical alteration could be erosion due to wind or rain or to alternate freezing and thawing which breaks parent rock masses down to small fragments.

1.1.2.1. Unloading

Powrie (2004) reported that it is a near-surface process and renders the rock mass more permeable to facilitate access for groundwater. Cracks and joints may form to depths of hundreds of meters below the ground surface when the effective confining pressure is reduced. Reduction in confining pressure may result from uplift, erosion, or changes in fluid pressure. Exfoliation is the spalling or peeling off of surface layers of rocks. Exfoliation may occur during rock excavation and tunnelling. The term popping rock is used to describe the sudden spalling of rock slabs as a result of stress release generally occurring in the deep tunnel excavation.

1.1.2.2. Thermal expansion and contraction

The effects of thermal expansion and contraction range from creation of planes of weakness from strains already present in a rock to complete fracture. Repeated frost and insolation may be important in some desert areas. Fires can cause very rapid temperature increase and rock weathering.

1.1.2.3. Crystal growth and frost action

The crystallization pressures of salts and the pressure associated with the freezing of water in saturated rocks may cause significant disintegration. Many talus deposits have been formed by frost action. However, the role of freeze–thaw in physical weathering has been debated. The rapid rates and high amplitude of temperature change required to produce necessary pressure have not been confirmed in the field.

1.1.2.4. Colloid plucking and organic activity

The other soil physical alteration processes are colloid plucking and organic activity.

- Colloid plucking: the shrinkage of colloidal materials on drying can exert a tensile stress on surfaces with which they are in contact.
- Organic activity: the growth of plant roots in existing fractures in rocks is an important weathering process. In addition, the activities of worms, rodents, and humans may cause considerable mixing in the zone of weathering.

1.1.3. Chemical decomposition

The chemical decomposition caused by either near-surface (chemical weathering) or deep-seated (hydrothermal alteration) processes can change the mineralogy of parent rock (Fell et al, 2015).

1.1.3.1. Mineralogical changes of rocks

Chemical weathering is mainly caused by groundwater (with acid, alkali or other aqueous solutions) access at very slow rates to low-porosity rocks. Then, the chemical reactions involved include carbonation, oxidation, reduction, hydrolysis and dissolution producing new minerals, some of which are soluble. The reactions details are published by workers like Selby (1993). In the case of more porous rocks, groundwater can also enter through inter-granular pores, decompose minerals and disintegrate rocks within few months or years of exposure (Table 1.1).

Table 1.1. Rocks susceptibility to weathering: common minerals & igneous rocks (Fell et al, 2015)

GROUPS	MINERALS	WEATHERING EFFECTS
Carbonates	Calcite	Readily soluble as acidic waters
	Dolomite	Soluble as acidic waters
Evaporites	Gypsum, Anhydrite, Halite	Highly soluble
Sulphides	Pyrite and various other pyritic minerals	Weather readily to form sulphates, sulphuric acid and limonite
Clay minerals	Chlorite	Weathers readily to other clay minerals and limonite
	Vermiculite, Illite	Weather to form kaolinite or montmorillonite
	Montmorillonite	Weathers to kaolinite
	Kaolinite	Stable (softens on wetting)
Oxides	Haematite	Weathers to limonite
	Ilmenite, Limonite	Stable
Basalt, Dolerite, Gabbro	Olivine	Highest susceptibility to weathering
Andesite, Diorite	Augite, Hornblende, Soldic feldspar, Biotite	
Rhyolite, Granite	Muscovite, Quartz	Lowest susceptibility to weathering

Fell et al. (2015) reported that hydrothermal alteration is a deep-seated decomposition of some igneous rocks partly or wholly by gases or waters heated by magma. It involves the chemical breakdown of some primary minerals to form secondary ones, which are generally weaker and less stable in water. Some important chemical weathering processes are: hydration, hydrolysis, chelation, oxido-reduction, dissolution and carbonation.

1.1.3.2. Hydration

Practically all chemical weathering processes depend on the presence of water. Hydration is the forerunner of all the more complex chemical reactions, many of which proceed simultaneously. Huggett (2011) stated it occurs when minerals absorb water molecules on their edges and surfaces, or, for simple salts, in their crystal lattices, without otherwise changing the chemical composition of the original material. For instance, if water is added to anhydrite, which is calcium sulphate $CaSO_4$, gypsum $CaSO_4.2H_2O$ is produced.

$$CaSO_4 + 2H_2O \rightarrow Ca^{2+} + SO4^{2-} + 2H_2O \quad (1.1)$$

The water in the crystal lattice leads to an increase of volume, which may cause hydration folding in gypsum sandwiched between other beds. Under humid mid-latitude climates, brownish to yellowish soil colours are caused by the hydration of the reddish iron oxide hematite to rust-coloured goethite.

$$Fe_2O_3 + H_2O \rightarrow 2FeO(OH) \quad (1.2)$$

The taking up of water by clay particles is also a form of hydration. It leads to the clay's swelling when wet. Hydration assists other weathering processes by placing water molecules deep inside crystal structures.

1.1.3.3. Hydrolysis

Probably the main process of chemical weathering, is the reaction where water splits into hydrogen ions (H^+) and hydroxyl anions (OH^-) and reacts directly with the silicate minerals in rocks. The small size of the ion enables it to enter the lattice of minerals and replace existing cations, commonly potassium (K^+), sodium (Na^+), calcium (Ca^{2+}), or magnesium (Mg^{2+}). The released cation then combines with the hydroxyl anion. The hydrolysis reactions of some minerals, examples of Anorthite:

$$Ca\text{-}silicate + H^+ - O\,H^- \rightarrow H\text{-}silicate + (Ca^{2+} + 2OH^-)\ (basic) \quad (1.3)$$

Orthoclase feldspar, $KAlSi_3O_8$, is as follows:

$$2KAlSi_3O_8 + 2H^+ + 2OH^- \rightarrow 2HAlSi_3O_8 + 2(K^+ + OH^-) \quad (1.4)$$

As water is absorbed into feldspar, kaolinite is often produced by weathering of silicate minerals as the associated ions such as silica, sodium, potassium, calcium, and magnesium are lost into solution. Hydrolysis will not continue in the presence of static water. The feldspar products aluminosilicic acid $HAlSi_3O_8$ and potassium hydroxide KOH are unstable and then react further. The potassium hydroxide is carbonated to potassium carbonate, K_2CO_3, and water,

$$H_2O + 2KOH + H_2CO_3 \rightarrow 2K^+ + CO_3^{2-} + 2H_2O \quad (1.5)$$

The potassium carbonate so formed is soluble in and removed by water. The aluminosilicic acid obtained reacts with water to produce kaolinite, $Al_2Si_2O_5(OH)_4$ (a clay mineral), and silicic acid H_4SiO_4,

$$2HAlSi_3O_8 + 9H_2O \rightarrow \varphi Al_2Si_2O_5(OH)_4 + 2Si(OH)_4. \qquad (1.6)$$

The silicic acid is soluble in and removed by water leaving kaolinite as a residue, a process termed desilication as it involves the loss of silicon. If the solution equilibrium of the silicic acid changes, then silicon dioxide (silica) may be precipitated out of the solution:

$$H_4SiO_4 \rightarrow 2H_2O + SiO_2 \qquad (1.7)$$

Weathering of rock by hydrolysis may be complete or partial (Pedro, 1979). Complete hydrolysis or allitization produces gibbsite. Partial hydrolysis produces either 1:1 clays by a process called monosiallitization, or 2:1 and 2:2 clays through a process called bisiallitization.

1.1.3.4. Chelation

It is the complex removal of metal ions and in particular ions of aluminium, iron, and manganese, from solids by binding with such organic acids as fulvic and humic acid to form soluble organic matter– metal complexes. The chelating agents are in part the decomposition products of plants and in part secretions from plant roots. Chelation encourages chemical weathering and the transfer of metals in the soil or rock. It helps to drive hydrolysis reactions. For example, muscovite chelation chemical equation is:

$$K_2[Si_8Al_2]Al_4O_{20}(OH)_4 + 6C_2O_4H_2 + 8H_2O \rightarrow 2K^+ + 6C_2O_4Al^+ + 6Si(OH)_4 + 8OH^- \qquad (1.8)$$

Oxalic acid ($C_2O_4H_2$), the chelating agent, releases $C_2O_4^{2-}$, which forms a soluble complex with Al^{3+} to enhance dissolution of muscovite. Ring-structured organic compounds derived from humus can act as chelating agents by holding metal ions within the rings by covalent bonding.

1.1.3.5. Oxidation and reduction

It is the loss of electrons by cations, and reduction is the gain of electrons. Both are important in chemical weathering. Oxidation weathering chiefly affects minerals containing iron, though such elements as manganese, sulphur and titanium may also be oxidized. Most important oxidation products depend on dissolved oxygen in the water. The oxidation of pyrite is typical of many oxidation reactions during weathering (Keller, 1957):

$$2FeS_2 + 2H_2O + 7O_2 \rightarrow 2FeSO_4 \qquad (1.9)$$

$$2H_2SO_4 + FeSO_4 + 2H_2O \rightarrow Fe(OH)_2 + H_2SO_4 \quad \textit{(hydrolysis)} \qquad (1.10)$$

The sulphuric acid H_2SO_4 formed in these reactions rejuvenates the process. Oxidation of $Fe(OH)_2$ gives:

$$4Fe(OH)_2 + O_2 + 2H_2O \rightarrow 4Fe(OH)_3 \qquad 2Fe(OH)_3 \rightarrow Fe_2O_3 \,.\, nH_2O \text{ (limonite)} \tag{1.11}$$

It may also drive the hydrolysis of silicates and weather limestone to produce gypsum and carbonic acid. Many iron minerals weather to iron oxide (Fe_2O_3, hematite). The red soils of warm, humid regions are colored by iron oxides. Oxides can act as cementing agents between soil particles. Reduction reactions, which are of importance relative to the influences of bacterial action and plants on weathering, store energy that may be used in later stages of weathering.

1.1.3.6. Dissolution

The process called solution or dissolution, involves the dissociation of the molecules of mineral salts in water, which is a very effective solvent into their anions and cations and each ion becomes surrounded by water. It is a mechanical rather than a chemical process, but is normally discussed with chemical weathering as it occurs in partnership with other chemical weathering processes.

Solution is readily reversed – when the solution becomes saturated some of the dissolved material precipitates. The saturation level is defined by the equilibrium solubility, that is, the amount of a substance that can dissolve in water. Once a solution is saturated, no more of the substance can dissolve. Minerals vary in their solubility. The most soluble natural minerals are chlorides of the alkali metals: rock salt, halite NaCl or potash salt KCl.

$$NaCl,\ 2H_2O \rightarrow Na^+ + Cl^- \tag{1.12}$$

These are found only in very arid climates. Gypsum $CaSO_4.2H_2O$ is also fairly soluble. Quartz SiO_2 has a very low solubility. The solubility of many minerals depends upon the number of free hydrogen ions in the water, which may be measured as the pH value.

1.1.3.7. Carbonation

The formation of carbonates, which are the salts of carbonic acid H_2CO_3. Carbon dioxide CO_2 dissolves in natural waters to form carbonic acid. The reversible reaction combines water with carbon dioxide to form carbonic acid, which then dissociates into a hydrogen ion and a bicarbonate ion.

$$H_2O + CO_2 \Leftrightarrow HCO_3^- + H^+ \tag{1.13}$$

Carbonic acid attacks minerals, forming carbonates. Carbonation dominates the weathering of calcareous rocks (limestone, dolomite) where the main mineral is calcite or calcium carbonate $CaCO_3$. Calcite reacts with carbonic acid to form calcium hydrogen carbonate ($Ca(HCO_3)_2$) that, unlike calcite, is readily dissolved in water.

$$CaCO_3 + CO_2 + H_2O \Leftrightarrow Ca^{2+} + 2HCO_3^- \quad (1.14)$$

This is why some limestone is so prone to solution. Limestone made of calcite and dolomite is one of the calcareous rocks that weather most quickly especially in humid regions. The carbonation of dolomitic limestone proceeds as follows:

$$CaMg(CO_3)_2 + 2CO_2 + 2H_2O \rightarrow Ca^{2+} + 2HCO_3^- + Mg^{2+} + 2HCO_3^- \quad (1.15)$$

The dissolved components can be carried off in water solution. They may also be precipitated at locations away from the original formation. This equation summarizes a sequence of events starting with dissolved carbon dioxide (from the air) reacting speedily with rainfall water to produce carbonic acid, which is always in an ionic state (1.13).

Carbonate ions from the dissolved limestone react at once with the hydrogen ions to produce bicarbonate ions (1.14). This reaction upsets the chemical equilibrium in the system, more limestone goes into solution to compensate, and more dissolved carbon dioxide reacts with the water to make more carbonic acid. In response, carbon dioxide diffuses from the air to the water, which enables further solution of limestone through the chain of reactions. Carbonation is also a step in the complex weathering of many other minerals, such as in the hydrolysis of feldspar (1.4).

Carbonic acid H_2CO_3 which speeds chemical weathering, is also produced by the roots of plants, by insects that live in the soil, and by the bacteria that degrade plant and animal remains structures.

1.2. Sinkholes formation processes

Naturally formed cavities mainly develop in geological areas made of rocks prone to dissolution by water or due to underground water fluctuations, whereas artificial cavities result from various civil engineering projects such as mining works, subsurface drainage pipes or tunnel excavation.

1.2.1. Underground cavities expansion

Sinkholes are predominantly due to the collapse of underground structures such as natural rock caves, man-made cavities and inhomogeneities in the stratigraphy.

1.2.1.1. Hydrogeological processes

Hydrogeologically, sinkholes formation are of four types based upon rate and process.

i. Sudden – collapse sinkhole

Collapse sinkholes are formed when the ceiling of an underground cavity can no longer support the overlying weight, resulting in an abrupt collapse of the overburden into the cavity, thereby forming a hole at land surface. Groundwater can provide buoyant support for the bridging overburden sediments. Fluctuations of aquifer water-levels near the rock-overburden boundary can lead to either a weakening of bridging sediments or a loss of buoyancy, or both collapse of the cavity roof either by time or by aquifer water-level fluctuations results in the formation of a sinkhole (DEM, 2017).

Delle et al. (2004) reported that this failure process appears to be very frequent due to the typical geological configuration in Apulian karst, which consists of stratified limestone with sub-horizontal bedding, intensely affected at various depth by paleokarst and active karst processes.

Figure. 1.2 configuration often produces, as result of the underground cave evolution, multiple slab breakdown deposits at the base of the main underground caverns. Soluble bedrocks including carbonates (limestone, dolomite) and evaporites (gypsum, halite) tend to be dissolved by water surface runoff over time. Most dissolutions occur in thick-bedded bedrocks. However, such dissolutions can also occur in thin-bedded bedrocks and even in lenses and are driven by one or more of the chemical reactions such as carbonation of limestone – (1.14), carbonation of dolomite – (1.15), hydration of gypsum – (1.1) and dissolution of halite – (1.12) on the bedrock surface where the water is mildly acidic (Zhou & Lei, 2017).

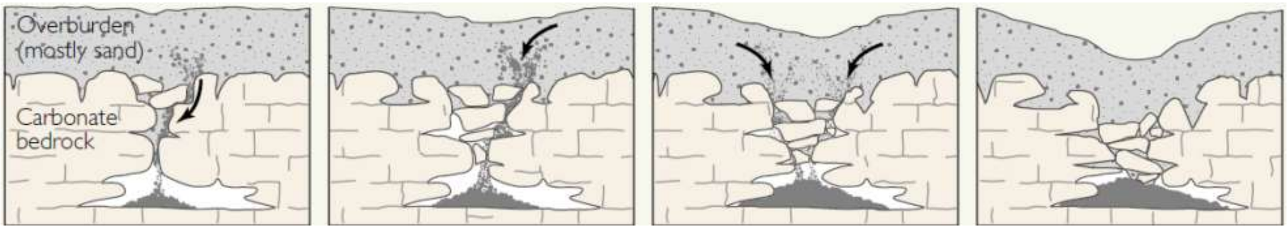


Figure 1.2. Illustration of cover-collapse sinkhole formation (modified from Tihansky, 1999)

Mixing of waters with different geochemical properties increases the power of dissolving rocks. In 1969, White & White reported that collapse sinkhole may continue to expand for months after the failure; can range in diameter and depth from less than a foot to hundreds of feet and finally poses a definite risk to loss of both property and life because it is a rapidly and abruptly forming sinkhole when the roof above fails, it sinkholes. Any change in loading above the top of the dome is distributed over the cavern walls and the additional loading would increase the shear along the walls, leading to collapse of the cavity roof.

ii. Ravelling / subsidence sinkhole

Cover – subsidence sinkhole is the classical solution process of doline formation in karst environment and a slow ravelling of sand into cavities in the underlying lime-rock, similar to sand running through an hourglass, which results in subsidence of the ground surface (Figure 1.3). This type of sinkhole can occur over a few hours, a few days, or a few weeks. They form as the overburden slowly migrates down into the fissures and cavities in the underlying rock. The result of a subsidence sinkhole is a depression in the land surface (DEM, 2017).

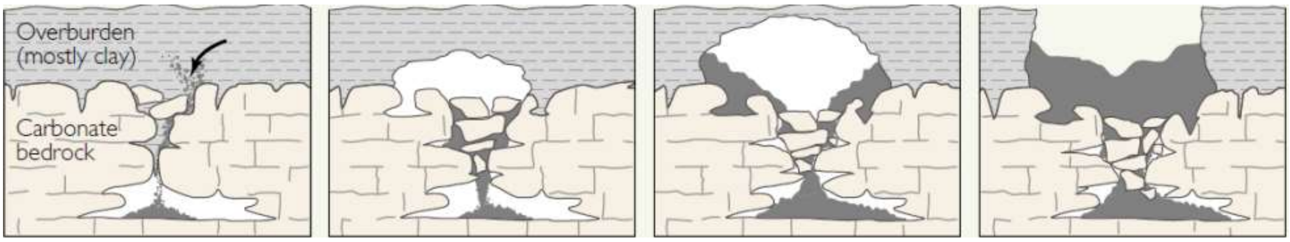


Figure 1.3. Illustration of subsidence sinkhole formation (modified from Tihansky, 1999)

Direct exposure of carbonate rocks facilitates the initiation of karst drainage, and its later evolution through sinkholes at the main points of water infiltration (Waltham & Fookes, 2003). The more intensely corroded zones, even due to weathering processes in the limestone rocks, begin to obtain topographic expression as solution dolines.

Soil is eroded into solution-enlarged cracks in the underlying bedrock as water percolates downward through the soil overburden, leading to voids in the soil. As the void becomes larger, the hydraulic conductivity becomes greater, thereby enlarging the "catchment area" of the void and accelerating the erosion/ravelling process. Under this mechanism, the rock cracks or channels need not be large. An opening as small as a few centimetres may be the exit for an erosion-ravelling dome several meters high if the water and the suspended solids can be readily transported into the network of solution-enlarged conduits (Zhou & Lei, 2017).

Han & Hwang (2017) reported that sinkholes of this type are generated in sandy soil where no viscosity is present in the surface layer and it occurs mainly at a place where underground water is close to the ground. The subsidence occurs gradually rather than suddenly so sinking occurs gradually as well. In the Italian Gargano Promontory areas for example, the relative vicinity and the elevated overall density together with the absence of larger compound sinks are all elements favouring the origin through slow and progressive development of solution process rather than rapid collapse. In addition to the Gargano, other areas in the Murge Plateau are also characterized by a high number of sinkholes originated by solution (Delle et al., 2004).

Referring to the DEM 2017 report, subsidence sinkhole in Florida, USA is a slow forming sinkhole that is created when sediment is slowly washed (ravelled) downward into existing small fissures, fractures, cavities, and conduits in the sediments or carbonate rocks below; forms over a period of months to millions of years; pose little to no risk to loss of life, but they can pose a risk to property over extended periods time.

iii. Dissolution / solution sinkholes

In a dissolution sinkhole on Figure 1.4, the process of subsidence is not visible in many cases and dissolution in the shallow surface layer progresses rapidly due to changes in the surface water flow thereby causing erosion that ultimately extends to the bedrock (Han & Hwang, 2017). These sinkholes are probably the most difficult to be detected with very often, no precursor signs.

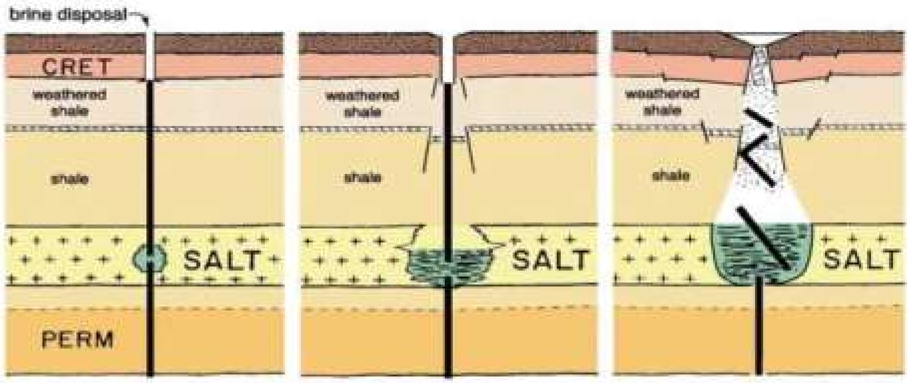


Figure 1.4. Dissolution sinkhole (modified from Walters, 1978)

The complex stratigraphy at the Italian site of Castellana-Grotte, reconstructed from borehole examination, showed the presence of a cavity filled with detritus and, in the most surficial part, anthropogenic deposits; likely, erosion within the succession produced differential settlements and the sinkhole development at the land surface (Zezza, 1976). It is interesting to note that the site is located within a larger area, which is the lowest part of a karst valley: there, man strongly changed the original configuration, altering the natural drainage and clogging many ways of underground infiltration for water. The Castellana-Grotte example once more puts into evidence the fragility of karst environment, and the difficulty to correctly evaluate the likely consequences that human impact might have in this territory (Delle et al., 2004).

In some cases, where the soluble rocks are covered by other non-soluble and poorly consolidated deposits, the development of a cavity may trigger erosional process even in the cover, until a sinkhole is formed at the land surface: dropout dolines (Figure 1.5). This was the case for the sequence at Marina di Lesina, where gypsum rocks are overlain by poorly cemented sands, and a number of sinkholes formed from 1993 to 2000 (Delle et al., 2004). The sinkholes started to develop in the evaporites and then transmitted in the above sands, following the typical modality of the mantled karst (Guitierrez et al., 2002).

Figure 1.5. Dissolution of dolomite bedrock in Lyttelton Quarry (Council for Geoscience, 2011)

iv. Rock / cap-rock – collapse sinkhole

Water level drop may lead to hydro-fracturing in soil following Anikeev (1999) who proposed a simple hydro-fracturing criterion that is controlled by the ratio of soil cohesion to the loss of buoyancy. Two main mechanisms can be distinguished (Figure 1.6) depending on their collapse mode and both occur in overlying non-soluble rock such as sandstone that is underlain by soluble rocks. The contacts between the soluble and non-soluble rocks can be favourable locations to karst development and mechanically, the formation processes of cap-rock sinkholes are similar to the impact of underground mining on the land surface.

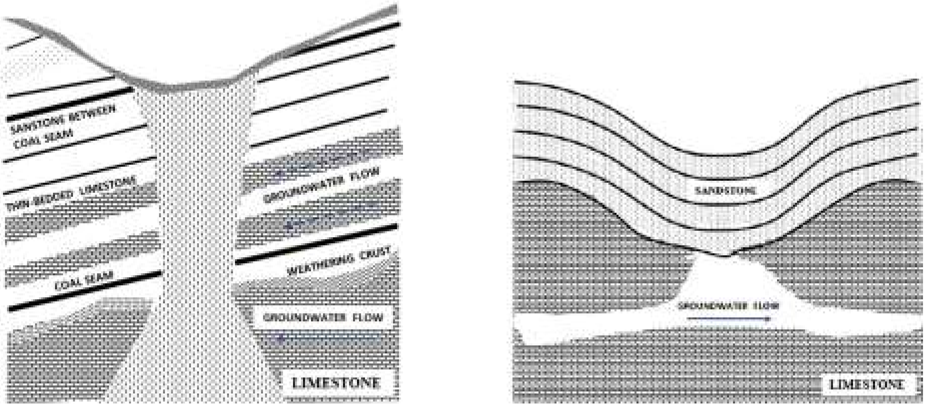


Figure 1.6. Site models of sinkhole – Caprock collapse (L) sagging (R) (Zhou & Lei, 2017)

The first type is the cap-rock collapse (Figure 1.6L) where the compound dissolution of gypsum and limestone may contribute to their formation in which the limestone is dissolved by both HCO^{3-} and $SO4^{2-}$. Gypsum can produce H_2S by sulphate-reducing bacteria (desulfovibrio). The collapses can be episodic. For the second type (Figure 1.6R), the sagging collapse, caprocks are mechanically weak and cannot preserve the karst voids as they are enlarged by dissolution and broad bedrock subsidences occur as a result of gradual sagging (Zhou & Lei, 2017).

1.2.1.2. Internal erosion

Collapses resulting from underground pipes leakage are result of two mechanisms which are the piping process and saturation. Saturation causes loss of cohesion of residual clays and also causes loading due to the addition of the weight of water (Newton, 1981).

Sudden collapse of the ground like a pitfall has been increasing nowadays in urban roads. It is often caused by an underground cavity expansion without being noticed. A collapse happens when the cavity has reached the surface ground. Many cavities are found close to underground structures. Some of those cavities are caused due to the breakage of buried structures such as sewer pipes. Soil is flowed out from cracks of pipes with water. The Figure 1.7 is one supposition which shows water flow concentrated at the gaps between the ground and buried structures and soil is drained through this "water pathway" and restart on another path until collapse (Sato & Kuwano, 2015).

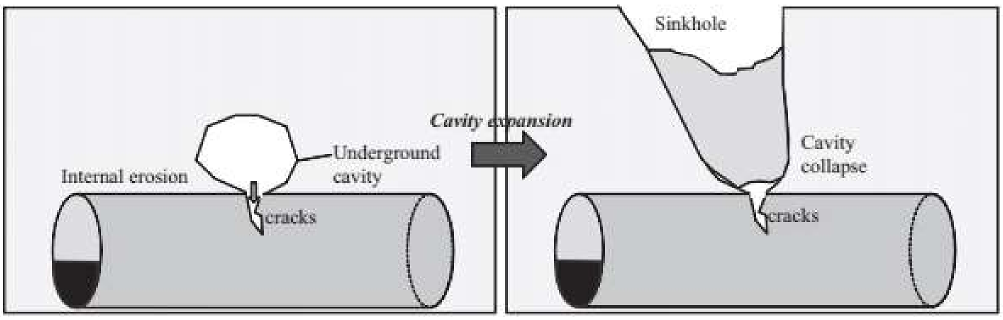


Figure 1.7. Internal erosion from cracks in pipes (Sato & Kuwano, 2015)

As upper soil is introduced to defective segments of aging conduits, a space can be formed in the lower part of the pavement on Figure 1.8. The pavement can maintain its stiffness to some extent so it can bear the subsidence within a range due to internal forces until a sudden collapse takes place when a limit has been crossed. Since the sink ground occurs up to the upper surface of the buried conduits, the depth varies according to the locations of buried pipes (Han & Hwang, 2017). On Figure 1.8, a relatively shallow depth ground collapse is formed in urban or industrial areas.

Figure 1.8. Sinkhole due to leakage in Pretoria, South Africa (Council for Geoscience, 2011)

1.2.1.3. Underground mines

Fasani et al. (2013) reported that large parts of Rome are exposed to a geological hazard caused by the degradation of an underground network of man-made cavities that were mainly excavated within the quaternary volcanic deposits. Underground mines on Figure 1.9 constitute quite complex networks of galleries in large sectors of the city of Rome.

Figure 1.9. Man-made caves at Villa De Sanctis, Rome - Italy (Fasani et al., 2013)

Underground excavations for coal mining purpose may create cavities in the subsurface due to deformations and displacements of the overlying strata, the extent of which depends on the magnitude of the in-situ stresses, mining induced stresses, void size, immediate roof characteristics and presence of geological discontinuities. Gradually, these movements work up to the surface to form a depression on the ground surface which is commonly referred to as subsidence (Figure 1.6) similar to caprock collapse (Sahu & Lokhandeb, 2015).

1.2.1.4. Tunnel excavation

If a tunnel is excavated in a place where the cover value is small and structures are positioned nearby, stress conditions in nearby ground can change then (Figure 1.10), surface subsidence and deformation of adjacent structures can occur due to the occurrence of loosening of the ground around the tunnel (Han & Hwang, 2017).

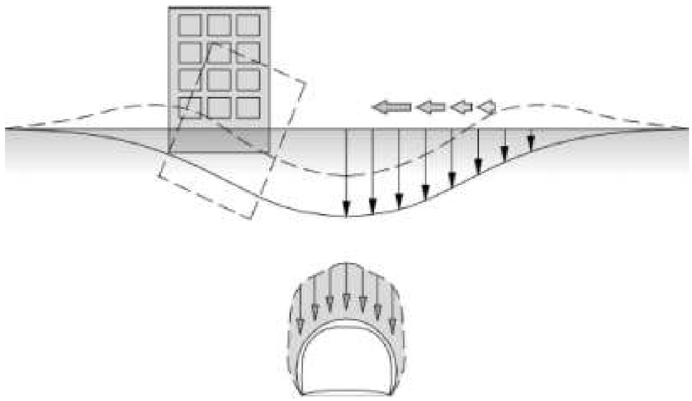


Figure 1.10. Behaviour of nearby Ground due to tunnel excavation (Han & Hwang, 2017)

1.2.1.5. Soil compaction

Emplacement of weight on thinned roofs of existing cavities in residual clay or on those of shallow bedrock cavities can cause their failure. The occasional collapse beneath heavy equipment is probably attributable to this cause. Differential compaction caused by the weight of a structure on unconsolidated deposits overlying the irregular surface of the top of bedrock also results in subsidence and foundation problems (Newton, 1981).

Very occasionally, subsidence sinkholes can happen due to loads increase on unconsolidated overburden deposits where underground cavities exist. Han & Hwang (2017) reported that underground cavity expansion can be caused due to defective compaction of backfill materials during construction of structures on upper soil layers.

1.2.2. Underground water disturbances

Zhou & Lei (2017) reported that water activities are probably the most important factor that causes sinkholes. Water functions as an enabling and trigger agent in the sinkhole formation.

1.2.2.1. Acidic water infiltration

Collapse also can result where surface water gains access to uncased or unsealed holes created by drilling, augering or coring. The piping process is generally responsible. The same process may be responsible for collapses that occur at drainage wells. Collapses caused by the impounding of drainage occur, in part, in the same manner as those resulting from diversions of drainage. The impounding of water results in the saturation and loss of cohesiveness of unconsolidated deposits overlying bedrock openings. This, accompanied by loading resulting from the weight of impounded water, can result in the collapse of overlying deposits into the bedrock opening and a draining of the impoundment.

If the impoundment is located where the water Table is below the top of bedrock and openings at the surface are interconnected with those in bedrock, a collapse can result from the piping process (Newton, 1981).

1.2.2.2. Dewatering scenarios

Foose (1953), in the first investigation of this type sinkhole activity, associated the occurrence of sinkholes with pumping and a subsequent decline in the water table. He determined that their formation was confined to areas where a drastic lowering of the water table had occurred, that their occurrence ceased when the water table recovered, and that the shape of collapses indicated a lowering of the water table and withdrawal of support. The development of sinkholes in South Africa is associated with pumping and creation of cones of depression and determined that sinkhole and subsidence problems increased where the water table was lowered.

These mechanisms, based on studies in Alabama are (1) loss of buoyant support to roofs of cavities or caverns in bedrock previously filled with water and to residual clay or other unconsolidated deposits overlying openings in the top of bedrock, (2) increase in the velocity of movement of ground water, (3) increase in the amplitude of water-level fluctuations, and (4) movement of water from the land surface to openings in underlying bedrock where recharge had previously been largely rejected because they were water filled. Pumping results in water-level fluctuations greater in magnitude than those occurring under natural conditions. The repeated movement of water through openings in bedrock against overlying unconsolidated deposits causes repeated addition and subtraction of buoyant support to them and repeated saturation and drying. Both result in the downward migration of the deposits that creates or enlarges cavities in them (Newton, 1981).

A load applied to an aquifer where ground water is distributed is shared by both the skeleton of the aquifer and the ground water. A change in volume in the skeleton does not occur in circumstances where ground water inflow and outflow are equivalent but when the ground water level is reduced due to the excessive collection of ground water. Then the load applied to the ground water is transferred to the skeleton so that the skeleton contracts due to the increase in load. As a result, particles are re-arranged due to plastic deformation and pore water flows to the outside so that the storage capacity in the aquifer is reduced gradually and the ground subsides permanently (Han & Hwang, 2017).

Figure 1.11. Dewatering induced sinkhole in South Africa (Council for Geoscience, 2011)

The natural process of induced sinkhole is drastically accelerated by groundwater level drawdown or ‘dewatering’. In the Bapsfontein area of South Africa for example, which had, prior to 2003, largely been unknown for sinkhole formation, the recorded rapid lowering of the water Table during that time led to some 28 sinkholes; one of the largest (> 50 m diameter) of these is pictured in Figure 1.11.

1.2.2.3. Heavy rainfalls

Groundwater is an aggravating factor in karstic sinkhole activity as it exacerbates infiltration, percolation, soil saturation and drainage (Luu, 2020). Moreover, karst cover soil is particularly hazardous in the case of intense rainfall and floods. Heavy rainfall within a short period has the ability to trigger formation of sinkholes in three ways. First, heavy rainfall adds additional weight to the overburden sediments above a cavity potentially causing failure of the cavity ceiling. Second, flood water from heavy rainfall naturally collects in low lying areas and infiltrates into the ground. Should a cavity be present below ground at that location, the weight of the flood water and accelerated infiltration may cause failure of the cavity ceiling. The third mechanism relates to an area that has sustained extended rainfall such that the overburden sediments become saturated and soft. Heavy rainfall can cause accelerated additive weakening of the overburden sediments causing failure of the cavity ceiling forming a sinkhole (DEM, 2017).

Water level fluctuation accelerates the void formation and enlargement in overburden soil. A rise in the groundwater level from below the soil–rock interface to above the interface will increase the degree of saturation of the soil, decreasing the soil strength. A subsequent drop of that elevated groundwater level is accompanied by the loss of the buoyancy support, which may initiate the acceleration of ravelling processes (Zhou & Lei, 2017).

Rainfall recharges the overburden strata, which decreases the strength of the rocks. This phenomenon increases the pore pressure, which can trigger roof fall. Recharge of the overburden can also increase erosion of the weak and weathered sandstones due to movement of water along closely spaced joints and faults which can also result in the formation of sinkhole on surface (Sahu & Lokhandeb, 2015).

Underground water flows from higher to lower places (water head difference) in a manner similar to surface water, and erosion occurs through the movements of underground water from higher to lower places creating continuous erosion which forms a cavity and ultimately a sinkhole develops (Figure 1.12). A sinkhole can also be created when the flow of surface water changes. When moisture content is increased due to changes in surface water flow in the soil ground where moisture content is low, pore water pressure is increased thereby reducing the effective stress to collapse the ground after excessive loads are exerted (Han & Hwang, 2017).

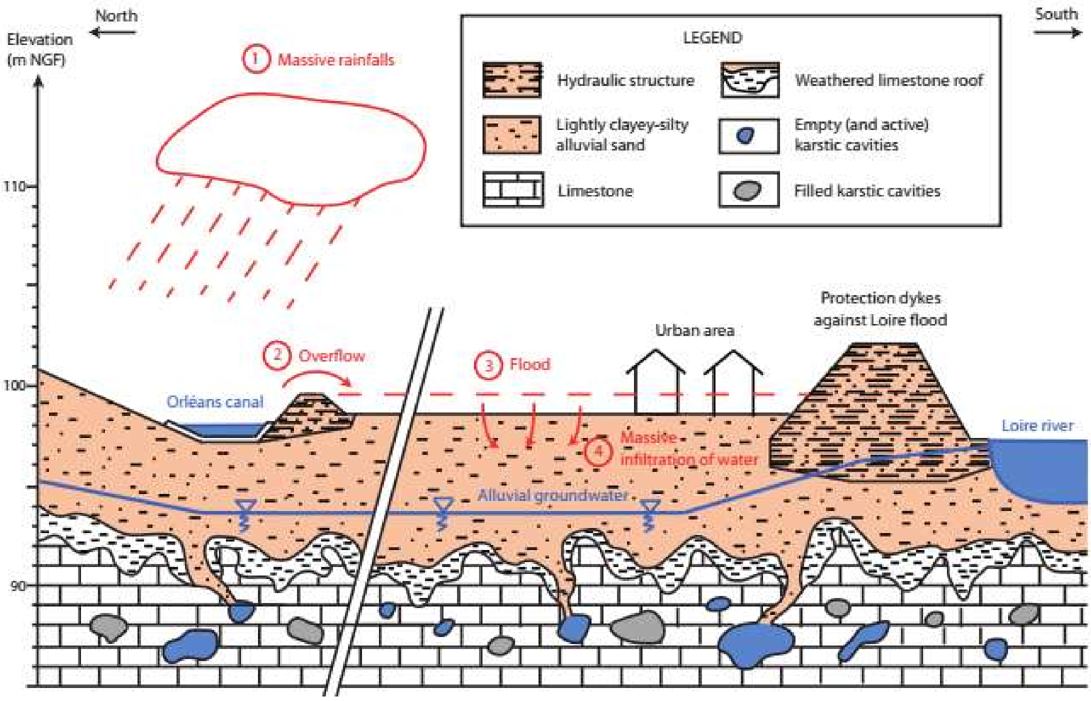


Figure 1.12. Geological cross section through the Loire floodplain in Chécy area and sketch of the cascading effects caused by the 2016 flood (Luu, 2020)

1.2.2.4. Dynamic liquefaction

Shocks or vibrations resulting from blasting can also cause or contribute to the failure of roofs of cavities in bedrock and unconsolidated deposits. About four percent (4%) of collapses identified in Missouri have been attributed to this cause.

Earthquakes can also suddenly increase the frequency of the occurrence of sinkholes. Earthquakes initiated significant liquefaction that drastically changed the ground surface. This is one of the major effects of earthquakes. Sinkhole subsidence can occur in various ways during an earthquake. Movement that occurs along faults can be horizontal or vertical or have a component of both. As a result, a large area of land can subside drastically during an earthquake (Sahu & Lokhandeb, 2015).

1.3. Underground cavities detection

An engineering subsoil model oriented to the problem of cavity risk assessment should quantify the geometry of the cavity network and properties that are of use to soil characterization (Fasani et al., 2013).

1.3.1. Superficial investigations

Scientists explore subsurface to stratify soils in order to detect underground cavities in calcareous rocks and evaluate areas of ravelling or which may be a sign of a potential sinkhole.

1.3.1.1. Geotechnical tests

Geotechnical researchers developed a lot of in-situ tests where the most used are SPT and CPT.

i. Exploratory drilling

It consists of drilling by driving a hollow cylinder into the ground with continuous rock sampling. This type of drilling is done only when it is necessary to know precisely the rock, its structure or to make physical and chemical tests. It is a slow operation; the progression is limited by the size of the cylinder of the corer or of the drill string (Mouici et al., 2017).

Previously the single method of searching for underground workings has been by drilling holes. Exploratory drilling is very expensive and laborious but necessary to drill a grid with holes up to 40 m deep 5-6 m apart. In addition, drilling does not guarantee high reliability of information, since data of individual observations are insufficient for generalizing conclusions for the investigated region (Zaderigolova, 1973).

ii. Standard penetration test

The Standard Penetration Test (SPT) is undoubtedly the most common method of soil exploration for foundation design. It is an invasive test that not only provides information from which soil strength can be estimated, but also provides a physical sample that can be visually inspected or used for laboratory classification and it is sensitive to operator and equipment variability. At any rate, the general concept of penetration resistance and the hands on soil sample recovery make it the choice of many designers (Gunaratne, 2014).

The SPT is described by the ASTM D-1586-99. This standard defines the appropriate manner in which the test should be conducted, which involves drilling techniques, penetration and sampling methods, proper equipment, and the reporting of results. The test procedure shown in Figure 1.13 consists of driving a standard 50mm outside diameter thick walled sampler into the ground at the bottom of a borehole using the repeated blows of a 63.5kg hammer falling freely through 760 mm. The SPT N-value is the number of blows required to achieve a penetration of 300 mm, after an initial seating drive of 150 mm (Robertson, 2006).

The ravelling of sinkhole has been studied and supported using field exploration tests within and around sinkholes just after surface collapse by Foshee & Bixler (1994). SPT borings found zones of very low N-value blow counts as well as zones where the drilling equipment advanced through the soil without any hammering. These soft anomaly zones of sandy material above limestone rocks are considered to be due to ravelling or subterranean erosion. Conditions like these found in close proximity to where sinkhole subsidence or collapse has occurred are also encountered within other areas of known karst geology suggesting a relationship between sinkhole activity and these soft zones (Shamet et al, 2017).

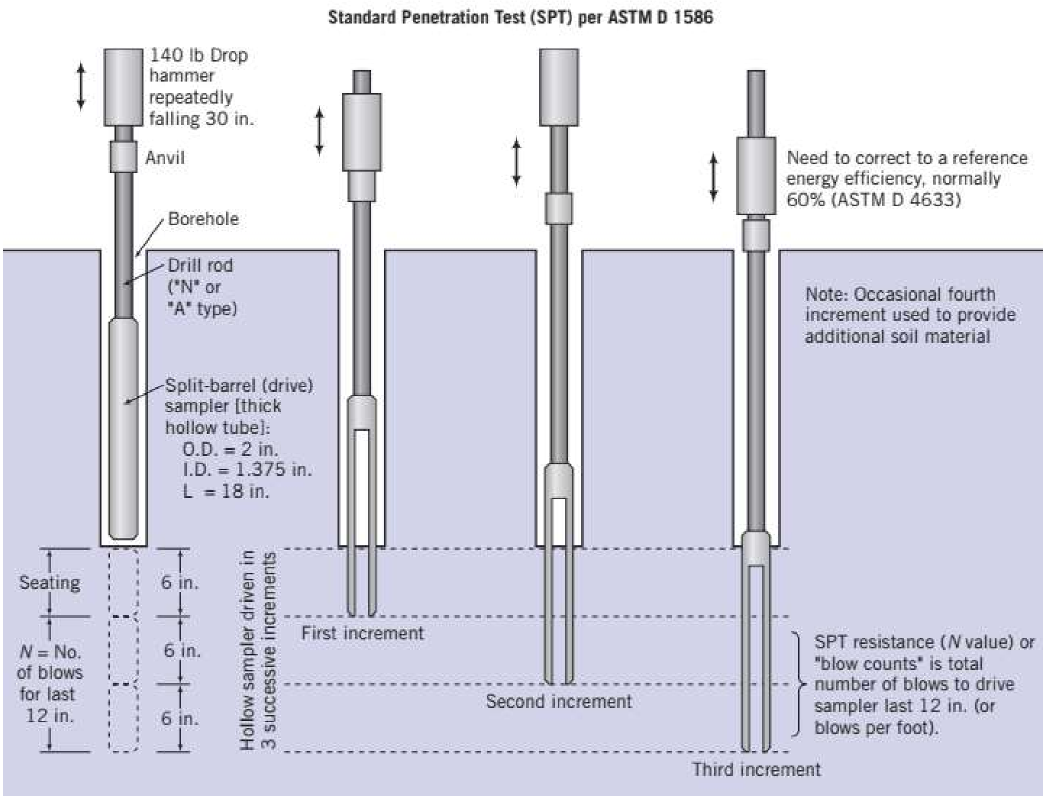


Figure 1.13. Driving sequence in a SPT test (Source: Professor Paul Mayne, GeorgiaTech)

iii. Cone penetration test

The Cone Penetration Test (CPT) is an invasive soil test that can identify soil strata type, soil properties, and strength parameters. It is highly repeatable, insensitive to operators, and can be performed in stiff to very stiff soils, and in some cases soft rock. Although this test retrieves no sample for laboratory testing or visual inspection, it produces enormous amounts of physical information based on correlations with other test methods such as SPT (Gunaratne, 2014). Although originally developed for the design of piles, the CPT is also used to estimate the bearing capacity and settlement of all foundation types.

The CPT is described in ASTM D-3441-79 and its enhanced versions such as the piezocone (CPTu) and seismic (SCPT), have extensive applications in a wide range of soils. The CPT is a conical tip with 60° angle and a base area of 10 cm^2 (Figure 1.14a) that is attached to a rod. An outer sleeve encloses the rod. The thrusts required to drive the cone and the sleeve into the ground at a constant rate of 2 cm/s are measured continuously and independently so that the cone or penetration resistance and friction or sleeve resistance may be estimated separately (Budhu, 2015). The total force acting on the cone, Q_c, divided by the projected area of the cone, A_c, produces the cone resistance, q_c. The total force acting on the friction sleeve, F_s, divided by the surface area of the friction sleeve, A_s, produces the sleeve resistance, f_s. In a piezocone, pore pressure is also measured, typically behind the cone in the u_2 location, as shown in Figure 1.14b (Robertson & Cabal, 2015).

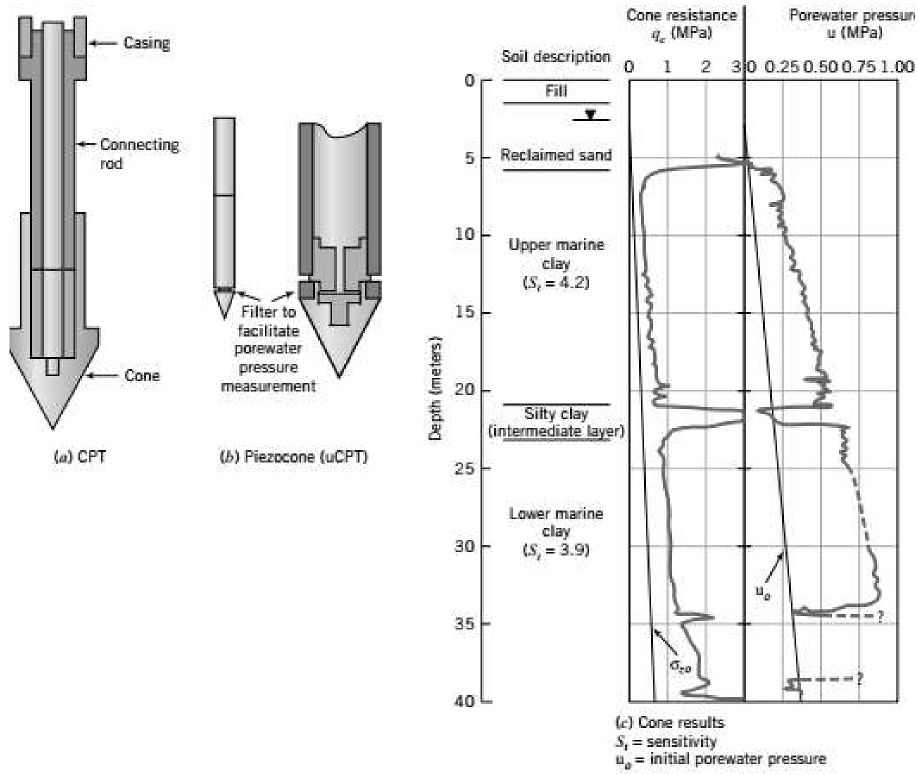


Figure 1.14. (a) CPT and (b) piezocone. (c) Piezocone results. (Chang, 1988)

Shamet et al. (2017) reported that low tip resistance zones suggest the possibility of partially ravelled or disturbed soils. The apparent drastic change in bedrock elevation along with corresponding varying thickness of loose soils is also a potential sign of sinkhole anomaly.

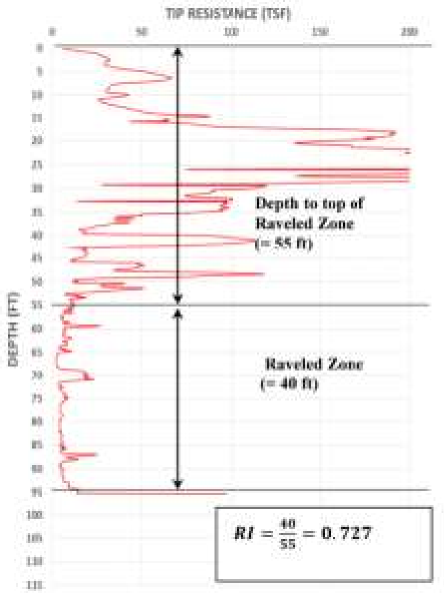


Figure 1.15. Example of calculated RI from CPT results (Shamet et al., 2017)

The Ravelling Index RI suggested by Foshee & Bixler (1994) can be useful to evaluate the sinkhole risk by using CPT and gives a relative indication of the degree of erosion taken place in the overburden sandy soil. RI is the ratio of the thickness of ravelled soil layer by the depth to the top of the ravelled zone. The smaller the ratio, the less deterioration of soil and the lower the risk of future sinkhole activity. An example of calculated RI ratio from CPT results is presented in Figure 1.15.

1.3.1.2. Geophysical tests

Geophysical tests are important in investigating cavity prone areas especially in urbanized sites.

i. Electrical resistivity tomography

In Electrical Resistivity Tomography (ERT) method, a potential difference is measured in response to the injection of a known amount of electrical current in the earth. Different earth materials have different resistance to the passage of current because of the variation in the degree of fractures, material types and degree of saturation. Both the injection of current and the detection of potential difference are carried out using four metal electrodes, current and potential, respectively (Hussain et al., 2020). The way in which these electrodes are configured has a direct influence on the results, and there are two adopted ways in which electrodes are configured as:

- Vertical Electrical Sounding (VES) is applied where the target is the determination of physical property of the subsurface with depth only (1D) and has a greater depth of penetration and spread length.
- Profiling is used for the estimation of both vertical and lateral changes in the subsurface, as is the case with karst studies. Under these conditions, 2D and 3D images of subsurface are obtained.

ERT has been applied successfully in karst studies such as their structures, soil cover and cavity geometry and more importantly the characterization of cavity sediments, the study of which is crucial for the speleogy, the groundwater vulnerability and the associated geological hazards. So, the method can be used as a control for the results accuracy assessment of the other applied geophysical methods – GPR and EMI (Hussain et al., 2020).

When compared to GPR or EMI, ERT is largely employed by geophysical companies and is less sensitive to electrical and electromagnetic noise. It also involves field operations that are less sensitive to the skill of the operating crew, if compared to seismic or gravimetric measurements, which is an advantage because uniform results can be obtained from different geophysical contractors. The probability of unambiguously detecting cavities of a given size by ERT is theoretically limited by the penetration and resolution capabilities of the resistivity survey and, practically, by specific conditions such as logistics and the electrical noise level at the experimental test site. In theory, the vertical resolution and the depth of the investigation are inversely related, and the depth of the investigation can be increased only at the expense of the vertical resolution, until the signal level decreases to unacceptable values.

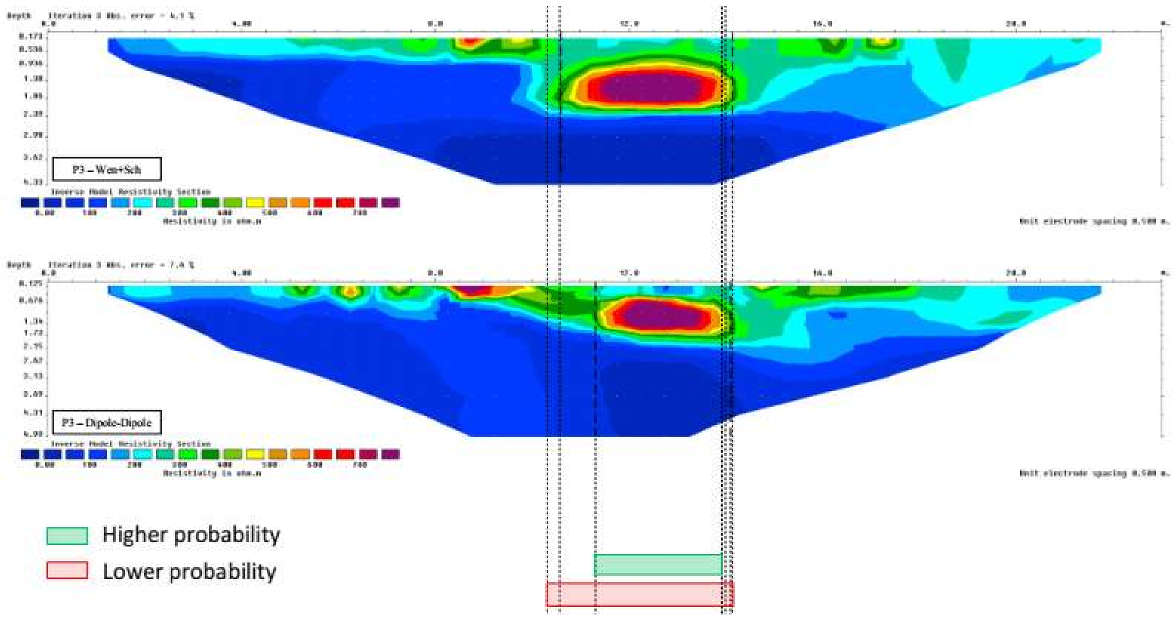


Figure 1.16. Void position estimation on ERT sections (Ungureanu et al. 2017)

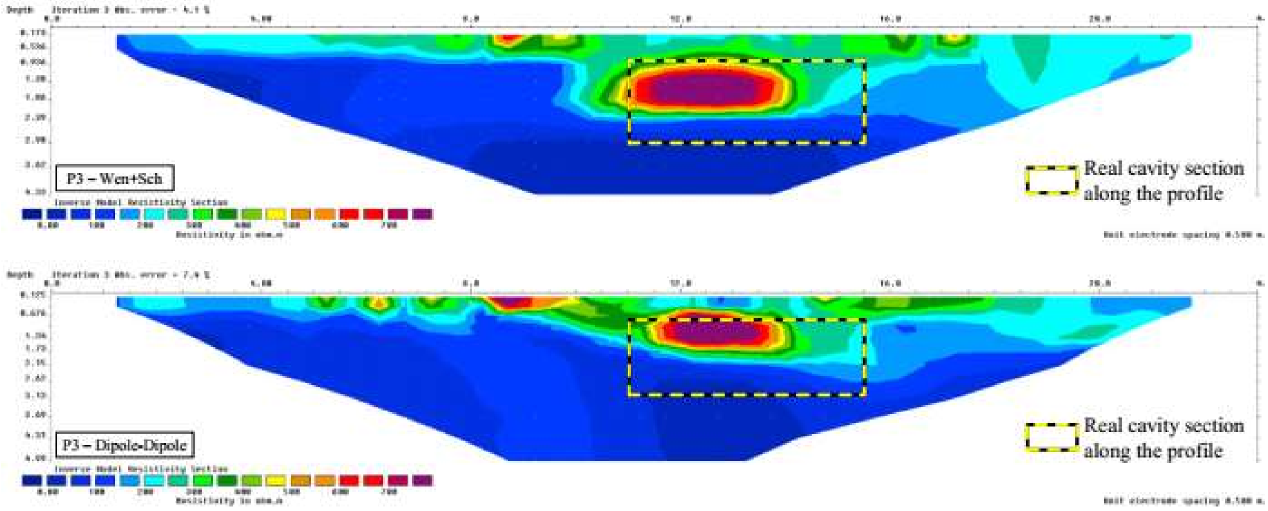


Figure 1.17. Real cavity distributions on ERT sections (Ungureanu et al., 2017)

Two popular arrays which are the dipole–dipole and the Wenner–Schlumberger array inverted results are superposed on the Figure 1.16 to obtain the area of higher probability of presence of cavities which gives the real cavity section along the profile on Figure 1.17. The depth of investigation can be increased significantly by using the dipole–dipole array with high values of the dipole separation factor, *n*, but only at the expense of the signal strength, which drops consistently when employing the dipole–dipole array. This signal strength can be unacceptable in the case of consistent electrical noise, causing too low of a signal-to-noise ratio at the potential electrodes (Fasani et al., 2013).

ii. Ground penetrating radar

To know the distribution of underground pipes and cavities, Ground Penetrating Radar (GPR) can provide real-time profile diagrams on the site which are clear and direct because of its high resolution, quick detecting and non-destructive feature.

Shi & Wu (2014) reported that GPR uses high-frequency electromagnetic waves in the form of wide-band pulses and transmits them into the ground via the transmitting antenna on the ground. When electromagnetic waves transmit underground, and they hit a medium interface or the targeted object such as a cavity, they can be reflected back to the ground and received by the receiving antenna, as shown in Figure 1.18. When the electromagnetic waves transmit in the underground media, their paths, waveforms and energy may vary with different electrical properties and geometric shapes of the media through which the waves transmit.

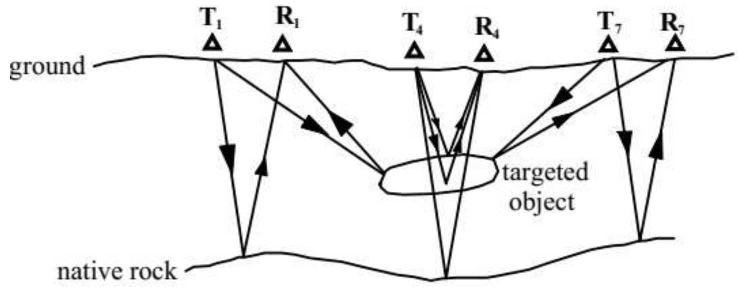


Figure 1.18. Reflection Detection Principle of GPR (Shi & Wu, 2014)

Therefore, through studies on time-frequency features and amplitude features of the reflected electromagnetic wave signals such as travel time (also called two-way time), frequency, breadth and waveform variation, the spatial locations and patterns of the underground interfaces or targeted objects can be identified precisely on Figure 1.19.

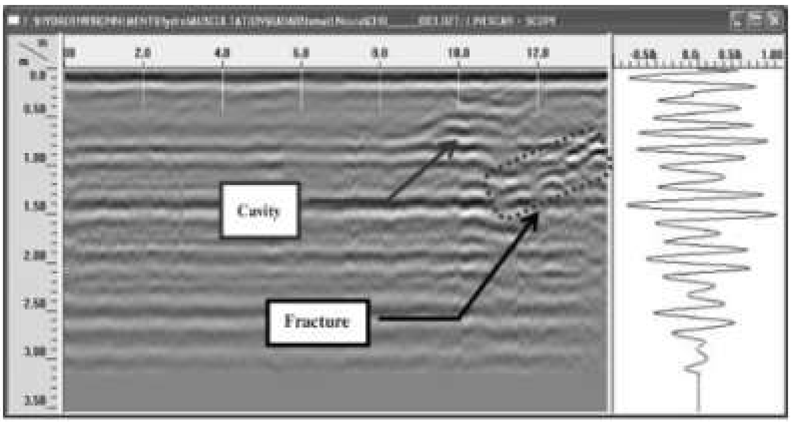


Figure 1.19. GPR profiles radargram showing cavity and fracture (Mouici et al, 2017)

iii. Electro-magnetic induction

The Electro-Magnetic (EM) surveys are based on the principle of electromagnetic induction. It differs from magnetometer. Instead, in EM induction method an alternating magnetic field in a coil or cable induces electric currents in a conductor. The conductivity of rock and soils is too poor to enable significant induction currents, but when a good conductor is present, a system of eddy-currents is set up. In turns, the eddy-currents produce secondary magnetic field that is superposed on the primary field and can be measured at the ground surface (Mori, 2009).

The typical EMI device is generally characterized by two dipoles at a constant distance. The transmitter dipole creates a primary electromagnetic field that provides an induction in the studied medium, producing a secondary electromagnetic field; their amount is recorded by the receiver and the relation between the primary and the secondary field can be converted to conductivity of the medium in which induction occurs. The instrument is not designed to be a metal detector but highly conductivity metals also generate a strong signal in response to the meter and their response tends to overload the circuitry (but it can be opportunely removed). Rather, it is designed to measure the much smaller signals generated by the conductivity properties of soils. The electronics of the device converts the signal into the measure of conductivity (because of its small size, p.p.m. and/or millisiemens per meter; mS/m). Electromagnetic techniques can be broadly divided into two groups.

- Frequency Domain Electromagnetic Method – FDEM: the transmitter current varies sinusoidally with time at a fixed frequency which is selected because of the desired depth of exploration of the measurement (high frequencies result in shallower depths).
- Time-Domain Electromagnetic Method – TDEM : the transmitter current, while still periodic, is a modified symmetrical square wave, where after every second n-period the transmitter current is abruptly reduced to zero for one n-period, whereupon it flows in the opposite direction.

FDEM method, more practical than TDEM concerning their utilization in field surveys, can operate in two different dipolar configurations: VDM and HDM. VDM (Vertical Dipole Method) has the vertical magnetic dipole referred to the transmitting coil; instead, in HDM (Horizontal Dipole Method) the magnetic dipole has horizontal direction. Generally the investigation depth of VDM is the twice of HDM (Mori, 2009).

An EM earth induction meter may record all of these events (Figure 1.20), but is quite important before doing a conductivity survey to have some idea of the local soil column, most directly but on a theoretical level, by consulting the local soil map and the description of the soil type.

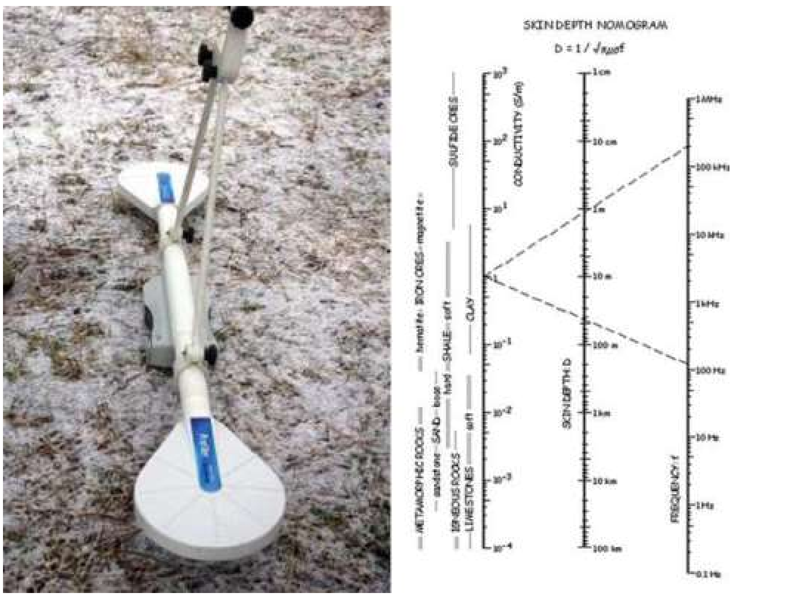


Figure 1.20. EM induction tool on the left (EMP-400 Profiler, by GSSI), and relationship among source frequency, ground conductivity, and skin depth on the right (from Won, 1996)

iv. Optical fibre sensor

Although the use of optic fibre sensors (OFSs) is not new technology, its use within the geotechnical community is a relatively new concept which competes well with current monitoring techniques since the OFSs are small, can survive chemically aggressive environments, is immune to electromagnetic interference and have the advantage that many different sensors can be multiplexed into one optical fibre (Chtcherbakov, 1997). OFSs are therefore ideal to be used in instability-prone areas since current measurement techniques, such as strain gauges and vibrating wire sensors, have the disadvantage of being vulnerable to electromagnetic interference, poor stability and poor durability.

Optic fibre consists of a glass core, enclosed by a glass cladding that has a higher refractive index than the glass core. Electromagnetic energy propagates in the core of the optical fibre by means of total internal reflection. A fibre Bragg grating (FBG) sensor in an optic fibre is a periodic modulation of the refractive index of the glass core of the optic fibre. The periodic modulation, i.e. the Bragg grating, is imprinted into the optical fibre by UV laser irradiation. The FBG reflects a Bragg wavelength that depends on the refractive index of the sensor area and the period of the refractive index modulation. The reflected Bragg wavelength of the FBG changes as the fibre is bent or if a temperature change is applied to it (Labuschagne et al, 2018).

1.3.2. Other investigation techniques

Other studies on gravimetric, seismic and geophysical methods have been carried out by researchers such as the use of waves and the topographical survey for a more precise work.

1.3.2.1. Cave topographical survey

Any survey which deals with caves requires a detailed mapping of the cavity. Our research has, therefore, carried out a great deal of survey work. However, the use of numerical modelling has made it possible to represent survey data in appropriate ways. Cave survey consists of recording distances, inclinations, and orientation of a succession of segments. Additional informations, such as width, height, profile and shape of the passages are sketched in a notepad (Mouici et al., 2017).

1.3.2.2. Radio waves

In considering the defects of exploratory drilling for searching for catacombs, it is possible to use radio waves based on a study of alternating electromagnetic fields induced in rocks. As radio waves propagate along the surface of the ground from an isolated transmitter located at a given point in the profile of investigation, they penetrate the ground to depths from 10 to 100 m and create called secondary fields, the intensities of which depend chiefly on the electrical properties of the rocks and on the wave frequency. By setting up radio receivers at some distance from the transmitter (at a spacing B) and measuring the components of the electromagnetic field, one obtains information concerning the geo-electrical conditions at depths corresponding to the penetration of the waves (Zaderigolova, 1973).

Conclusion

This introductory chapter was a review on soil weathering processes both physical alteration (with erosion agents such as wind or rain) and chemical decomposition (through reactions such as hydrolysis, dissolution and carbonation) of rocks in order to understand soil formation mechanisms on Earth's surface. Then a description of the various types of sinkholes with their formation and triggering mechanisms such as men activities, underground water disturbances has been made. To detect underground cavities, a list of in-exhaustive geotechnical and geophysical investigation techniques was presented to show up any details sometimes missed and handle the problem before any civil structures construction. Although, if those caves are found beneath a building foundation, geotechnical engineers must find the most suitable underpinning technique to prevent the collapse of the strucure.

CHAPTER 2: PRACTICAL APPLICATION

Introduction

Ground instabilities mostly caused by sinkhole hazards are generally the surface expression of the caves collapse and water level fluctuations. The ground surface expression could impact the civil structures above in various ways which need to be assessed. Sinkholes can have very noxious consequences such as road subsidence, building foundation collapse; and in the worst cases, injury or loss of life. When those damages already happened, the most rapid emergency solution to reinforce the soil beneath the foundations in order to avoid the building collapse must be designed by geotechnical engineers. In this chapter, a review has been done on structural deformations and their monitoring; then on the classical underpinning techniques and new grouting techniques as reinforcement solutions with an emphasis on compaction grouting design and field application.

2.1. Structural deformations

The development of sinkholes may cause deformations in the anthropogenic structures, where present and eventually leading to collapse (Delle et al., 2004). Those structural movements mostly consist of horizontal extension, differential settlement and angular distortion.

2.1.1. Horizontal extension

Horizontal strain or lateral distortion is the average strain due to the relative horizontal movement of two reference points. A combination of vertical and lateral displacements can also happen then give diagonal cracking in walls particularly (Figure 2.1).

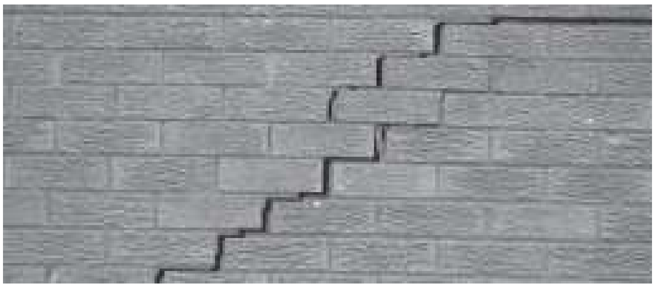


Figure 2.1. Diagonal cracking on structure (Holland, 2012)

2.1.2. Differential settlement

Foundation settlement and subsidence cracks often share some common features found on structures (Figure 2.2). Subsidence is movement of the foundations related to the ground. That means it is not caused by the weight of the building. For example, when subsoil shrinks, causing the foundations of a building subside, this would happen whether it was a bungalow with little load or a much taller building with a lot of load. The movement is caused by the ground moving beneath the foundations (Holland, 2012).

Figure 2.2. Townhouse damaged due to a sinkhole in Gauteng, (Council for Geoscience, 2011)

2.1.3. Angular distortion

Angular distortion or relative rotation is a measure of the shearing distortion of a structure. Angular distortion is often approximated as the rotation, due to settlement, of the straight line joining two reference points on the structure minus any rigid body tilt that the structure may have incurred. In general, the angular distortion calculated is an average value for the portion of the building bounded by the reference points (Boscardin & Cording, 1989).

2.2. Structural auscultation techniques

The development of sinkholes is generally related to the presence of underground caves, whose stopping towards the ground surface may cause settlement in the anthropogenic structures, where present, eventually leading to collapse (Delle et al., 2004).

2.2.1. Structural monitoring techniques

The selection of measurement method is based on an evaluation of accessibility, required accuracy, response time and cost because there is such a wide variety of instruments.

2.2.2. Crack monitors

Where the location of the movement is controlled by structural joints, cracks, or other zones of weakness, measurement can be made by simple means, such as placing a pencil tick on either side of the defect, of which the distance between can be periodically measured. Alternately, rulers can be placed across the joint in a similar manner, usually with one side of the defect at a major distance graduation (Figure 2.3). The beginning measurement of the free side is recorded, and any movement can be observed by the change in measurement.

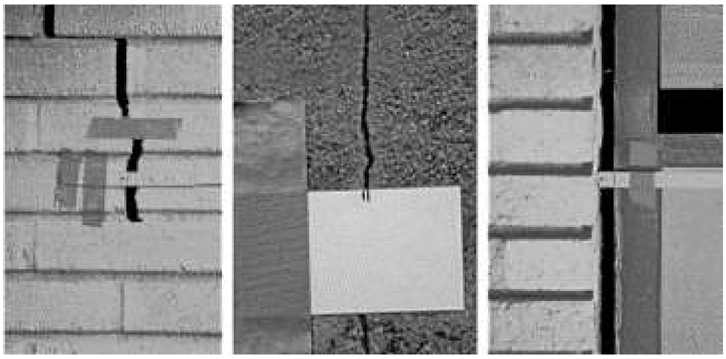


Figure 2.3. Simple crack width monitoring devices (Warner, 2008)

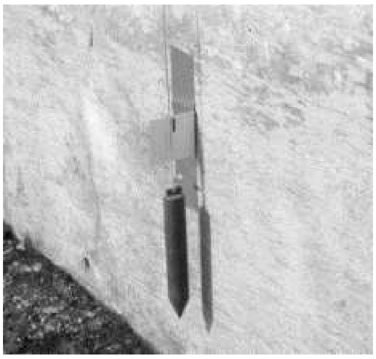
Although it does not give an exact value of the movement, it can give a signal that movement has occurred. Thus, use of pencil or paint marks is every bit as effective, they can be removed without marring the surface. Electronic extensometers or convergence gauges can be used for special applications where continuous monitoring is required via a recorder or telemetry for remote monitoring (ASCE, 2010).

2.2.3. Rudimental techniques

A simple string line drawn tightly across the area of interest is unquestionably simple yet is an effective method to detect differential movement. The line must be firmly anchored on both ends as showed in Figure 2.4.

Figure 2.4. Gauge block to check for surface movement under string line (ASCE, 2010)

Common plumb bobs and carpenter's levels are useful for monitoring structures. Vertical tilt can be observed from a changing position at the bottom of a plumb line suspended from the top of a structure. The line is best held out from the surface a few inches at the top and a bracket fixed to the surface at the base. A mark indicating the starting position should be placed on the bracket, as illustrated in Figure 2.5L.



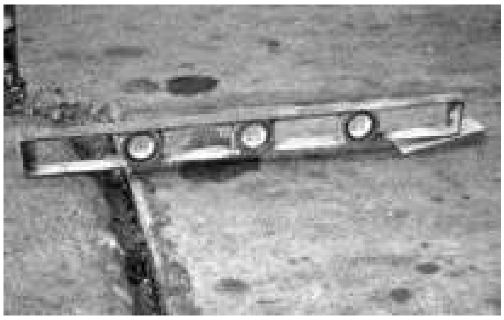


Figure 2.5. (L) Plumb line suspended from top of structure – (R) Carpenter's level wedged to a position across pavement joint (ASCE, 2010).

For active short-term monitoring of structural or ground surface level changes, a fluid level can be used as shown in Figure 2.5R. A wedge is conveniently placed under the low end so that the instrument is absolutely level, with the bubble perfectly centred. Any change in level can thus be easily observed (ASCE, 2010).

2.2.4. Tiltmeters

Tiltmeters measure angular movement or rotation of surfaces. A simple tilt measurement is to measure horizontal deviation over the length of a plumb line or spirit level. More sophisticated devices, such as the portable inclinometer and electronic tiltmeters using accelerometers or vibrating wire technology are also available (Figure 2.6). Both accelerometers and vibrating wire instruments can be set up to be permanently mounted and read remotely (ASCE, 2010).

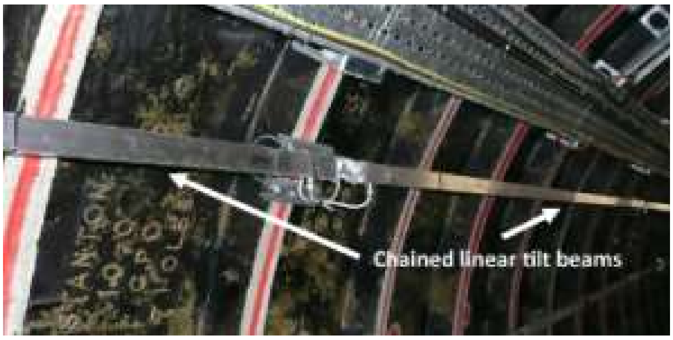


Figure 2.6. Chained linear tilt beams monitoring horizontal movement in a cast-iron bolted lining tunnel (GOESENSE, 2018)

2.2.5. Rotating laser levels

Rotating laser levels (Figure 2.7) consist of a rotating laser beam that impinges on one or more target prisms. The prisms are placed on stands or attached to structures and set to the level of the rotating beam. An audible sound emanates from any target that moves out of the laser plane. ASCE (2010) reported that this method is typically used during construction to establish planar surfaces of structures, slabs, or the ground surface. Although laser instruments indicate a change of position, they do not provide a direct measure of the magnitude of such movement, so where rotating laser levels are used they should be backed up with other devices, such as a surveyor's level. The degree of accuracy of these instruments varies greatly.

Figure 2.7. Rotating laser level for heave monitoring (ASCE, 2010)

2.3. Reinforcement techniques

The means and methods of supporting a structure foundation depends on many factors including: type of loads, state of existing foundations, magnitude of allowable deformations, subsoil and groundwater conditions, access to the foundations and potential for hazards.

2.3.1. Classical underpinning techniques

Underpinning is accomplished by digging underneath shallow footings and extending the foundation by pouring concrete in depth so that the foundation rests on a stronger soil stratum.

2.3.1.1. Mass concrete underpinning

The mass concrete (or traditional) method of underpinning is an established technique, suitable for relatively shallow depths of underpinning. The method is probably the most common form of underpinning and is often used for partial underpinning of sections of a building (Figure 2.8). The method can be used in cohesive or granular soils, but is widely used in shrinkable clays. It can be used to prevent movement due to subsidence and heave, and is often used to assist in retro-fit basement construction.

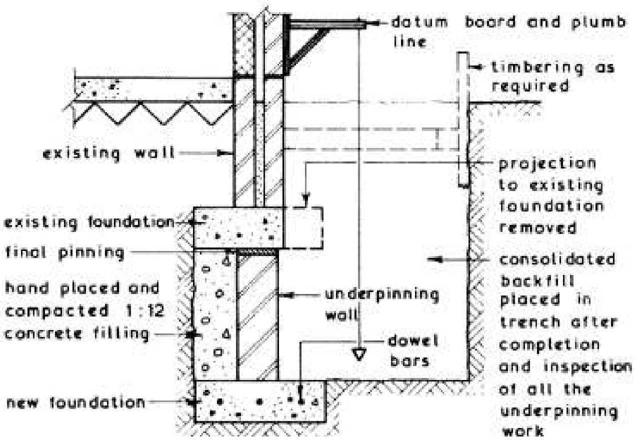


Figure 2.8. Traditional wall bearing underpinning (Chudley & Greeno, 2008)

The technique involves the construction of a new foundation beneath a failing section of building by extending the existing footings down to a greater depth where stable soil of a suitable bearing capacity exists. This is achieved by excavating individual bases in short lengths (usually not exceeding 1200 mm) in a predetermined sequence. Once each base is excavated to the appropriate depth, and before concreting, supervisory staff and building control inspect the excavation to check that the correct stratum has been reached and that the ground is free from soft spots, tree roots.[1]

Once the excavation has been approved, shutters are set in position and the base is backfilled with concrete. The concrete is usually cast to leave a narrow gap between the top of the base and the underside of the footing. When the concrete is cured dry pack is then used to fill this gap to help transfer loads from old to new. Base construction is repeated sequentially until the whole length of wall where underpinning is required has been supported. Bases will generally be linked together using ‘joggle’ joints to provide a key between adjacent bases. Reinforcement cages can be introduced using couplers to provide continuity between bases. Anti-heave precautions consisting of polythene sheeting or low-density polystyrene are usually installed when underpinning is constructed in shrinkable clay.

2.3.1.2. Micropilling techniques

Micropiles were developed in Italy in the early 1950's in response to the demand for innovative techniques for underpinning historic buildings and monuments that have endured damage with time.

[1] ASUCplus www.asuc.org.uk, 2020

This type of underpinning is used to stabilize or upgrade existing foundation by installing micropiles through pre-drilled holes determined by loading characteristics. Micropiles are described as small diameter (D<300mm) piles that can be installed in almost any type of soil and that can withstand a higher proportion of all the design loads (up to 500 tons) compared with conventional piles. These micropiles are steel reinforced (a pipe, a rebar or a group of rebars) placed into a small diameter hole and sealed to the ground by grout injections under relatively high pressure (Kordahi, 2004).

In terms of execution, micropiles can be: drilled, self-drilled (mainly used in the USA, with the implementation process involving only a few steps: drilling, injecting and sealing), or driven (less popular, mostly because of the vibration provoked in the ground, so its use is not recommended near other buildings). In the area of rehabilitation, micropiles have been used to underpin foundations (Figure 2.9) that exhibit excessive settlement, where the ground resistance is not enough to sustain the load of the superstructure or an additional load (Telmo, 2012).

In terms of classification, it is generally accepted that the sealing is important to the bearing capacity of the micropile, with the acceptance in various standards (FHWA, 2000, DTU 13.2, 1992) of 4 kinds of micropiles. Micropiles technique offers an ideal solution for enhancement of the building raft foundation which has been used widely for repairing foundations, underpinning and side stabilization.

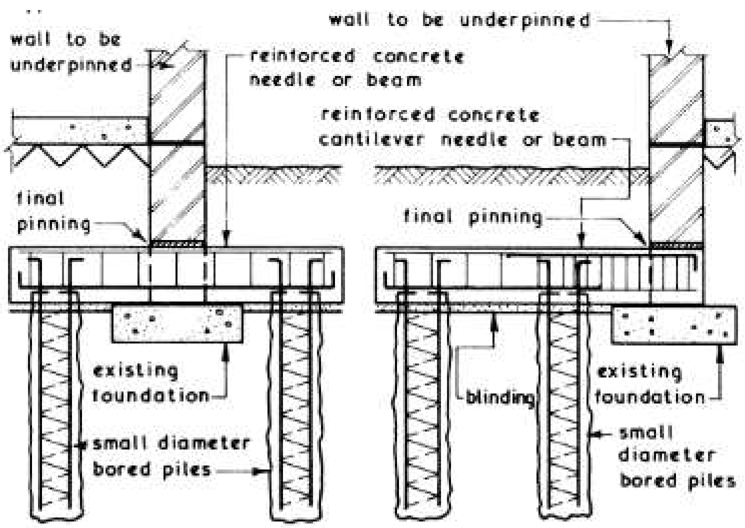


Figure 2.9. Pile underpinning (Chudley & Greeno, 2008)

Some of the advantages of micropiles are high carrying capacity, less site constraint problems (remote area), low noise and vibration, self-sustained operation and can be designed to have very low settlements about few millimeters or less under working loads. Under these conditions, its bearing capacity is not fully mobilized (Kordahi, 2004). Furthermore, micropiles are easily used in proximity to existing structures (Figure 2.10); adapted in difficult ground and drilling conditions (e.g. karstic areas, uncontrolled fills, boulders).

Figure 2.10. Retention by the system: underpinning beams and micropiles (Telmo, 2012)

2.3.2. Grouting techniques

Grouting is a soil improvement method which consists of injecting appropriate materials under pressure into rock or soil through drilled holes to make them less permeable and stronger.

2.3.2.1. Jet or Replacement grouting

Jet grouting technology have initially been developed in Japan (Yahiro & Yashida 1973), the UK and Italy. Its underpinning principle is similar to conventional methods, with the following objectives: minimize impact to existing structures/utilities, provide an engineered structure to support the loads, do it safely, do it quickly, monitor and control the operations.

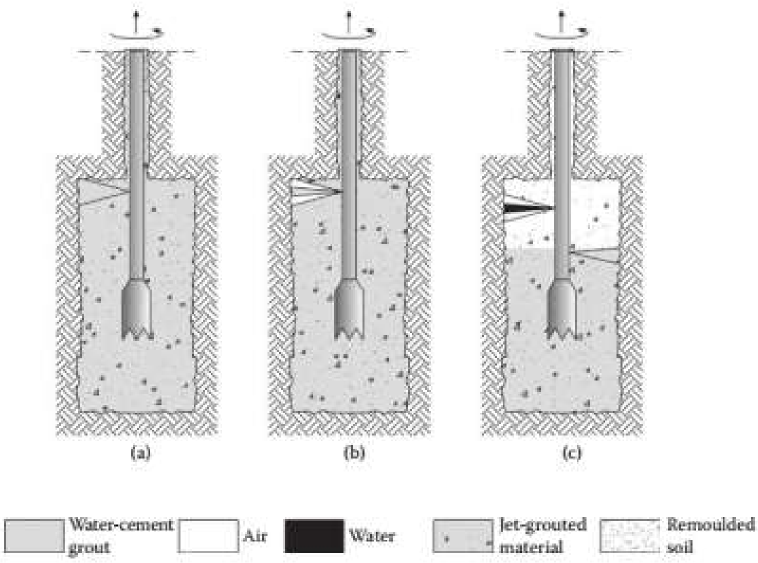


Figure 2.11. Jet Grouting: (a) single fluid, (b) double fluid and (c) triple fluid (Croce et al., 2014)

Recently, vertical jet grouting solutions have also become competitive and advisable in several and more usual scenarios, like foundations, earth retaining and underpinning works. According to the definitions of the European Standard on Jet Grouting (CEN/TC 288), jet grouted structures consist of interlocking jet grouted elements. An element is the volume of soil treated through a single borehole, which may be a cylindrical jet grouted column or a planar jet grouted panel.

Jet grouting has nothing to do with common grouting, as according to the jet grouting technique the soil is disintegrated by a jet of water or grout at very high pressure (bigger than 30MPa), obtained through the transformation of the high pressure flow (potential energy) into the high speed jet directed to the soil (kinetic energy) due to the very small diameter nozzles effect, and is subsequently mixed with the grouting material (Figure 2.11). A part of the mixed material returns to the surface along annular space around the drill rods or along neighbouring boreholes, serving for necessary three pressure relief (Croce et al., 2014).

2.3.2.2. Compaction or Displacement grouting

Compaction grouting consists of injecting low-slump (generally less than 5 cm) soil cement mortar or “low mobility grout” under high pressures (350-700kPa) to compact and displace surrounding soils.

The grout does not permeate into the soil pore space, but rather creates “grout bulbs” that expand at the injection point around the grout pipe tip (Figure 2.12). This application has been used most often for remediation of settlement problems, soil loss due to tunnelling activities, and slab or foundation jacking (re-levelling). It has also been successfully used for treatment of sinkholes, to mitigate liquefaction susceptibility beneath existing structures, and in sensitive urban sites where other surface access treatments such as vibro-methods are not feasible due to excess vibrations, access, or other concerns (Boulanger & Hayden, 1995).

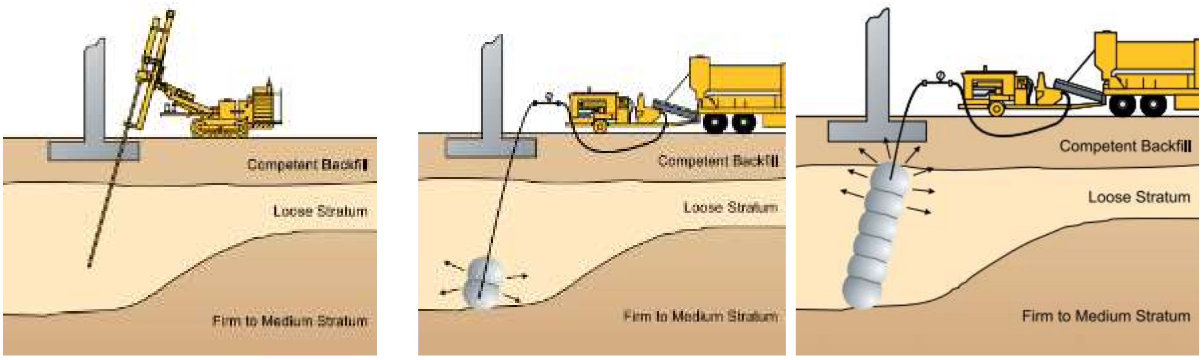


Figure 2.12. Schematic of compaction grouting process (Courtesy of Hayward Baker)

The improvement mechanism of compaction grouting is densification. While this will improve soil strength, it does not significantly improve the cohesion or render the soil into a monolithic mass. It is thus not nearly as effective as permeation grouting in reducing the propensity for soil loss into karst voids. Simple low mobility grout (LMG) is often represented as compaction grout when remediating karst, but this is technically erroneous due to its frequent fluid behaviour, irrespective of slump. While it can improve the density of very loose soil, it will not be able to achieve nearly as much compaction, and is thus less likely to prevent soil loss into karst features (Warner, 2008).

While grouting is typically an expensive proposition as compared to many other densification techniques, it may be an economical solution for certain difficult conditions—for example, where thin, loose, deep strata exist beneath dense layers, existing construction, utilities or other infrastructure. In fact, the cost of compaction grouting may be an order of magnitude greater than other deep densification methods, but may be the only alternative, and still be less expensive than using drilled shafts or driven piles.

2.4. Compaction grouting technique

Compaction grouting is a soil improvement and underpinning technique which objective is to displace and compact the surrounding soil without permeating or hydro-fracturing it. Compaction grouting has been developed and used almost based on field practical experience.

2.4.1. History and mechanism

The description of the mechanism combined with data derived from the excavation and removal of full scale test injections help understanding the progression of the method through the past.

2.4.1.1. Historic of the method

The earliest mention of the use of stiff mortar like grout, termed compaction grout, was by Graf (1969), who described the use of the Koehring Mudjack to pump a clayey loam mixture. He reported that some of the mudjack operators had found it useful to pump “zero slump” grout through pipes driven into the ground for raising structures and that larger than calculated quantities were sometimes used, concluding that the surrounding soil was being compacted. He referred to this as compaction grouting. One of the principal advantages of compaction grouting is that its maximum effect is in the weakest soil zone and it is effective in fine-grained soils that were formerly considered ungroutable.

A primary use of compaction grouting has been to compact loose fills or natural loose soils, underpin structures that have suffered differential settlement, and lift the structure, foundation, and subgrade to level (Graf 1969, 1992). It has also been used for improving in-situ soils to reduce the liquefaction potential during earthquakes (Boulanger & Hayden, 1995), and as a construction tool to limit ground movement during soft ground tunnelling (Baker et al. 1981).

2.4.1.2. Mechanism of compaction grouting

The mechanism of compaction grouting, unanimously accepted today in the USA, starts with the grout incorporation which causes the appearance of a complex propagation system of radial and tangential stresses. Figure 2.13 shows a zone of significant reworking in contact with the mortar mass, which results in shear and plastic deformation. Further on, the ground is in a state of elastic deformation. The extent of the zone of influence depends on (1) the characteristics of the soil (nature, water content, compactness); (2) the injected material (rheological characteristics); (3) the incorporation parameters (flow rate, pressure, quantities injected).

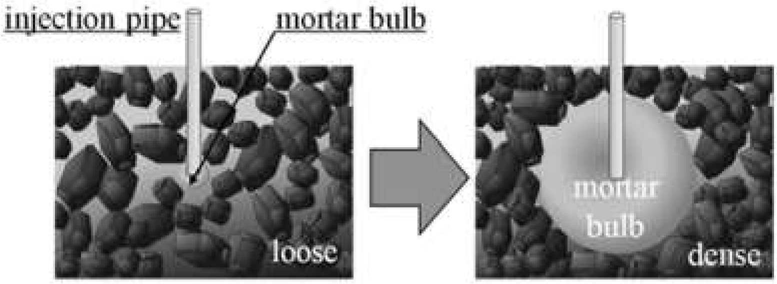


Figure 2.13. Compaction grouting principle (Takenouchi & Sassa, 2020)

It is the reworking zone hypothesis that makes it possible to explain the poorly understandable observations made by means of density measurements, where the remote soil appeared more densified than the soil directly in contact with the inclusion. In fact, these phenomena can simply be explained by the basic principles of soil mechanics (Figure 2.14): if we consider a small volume of soil at the periphery of a borehole and its deformation after incorporation of mortar, it seems clear that this extremely stretched volume of soil is in a plastic state. As the deformations decrease with the distance from the injection point, there is a distance at which the deformations are no longer plastic, but elastic. The ground at the periphery of the mortar ball breaks up up to a certain distance from the injection point, where it is the seat of important shear forces. An explanation for the formation of a zone of lower density at the periphery of the mortar ball can be found in the known phenomenon of dilatancy, at least in granular soils: they are all subjected to it, if the containment stress is sufficiently low, but compaction grouting is applied precisely to loose soils. However, it remains to be shown that this hypothesis is well verified in all cases. As for coherent soils, the increase in pore pressure leads to a deconsolidation of the soil near the mortar ball, which may be the cause of the drop in density. At a general treatment level, the mortar balls are made one above the other to form inclusions in the soil, either vertical or inclined (Iagolnitzer et al., 1996).

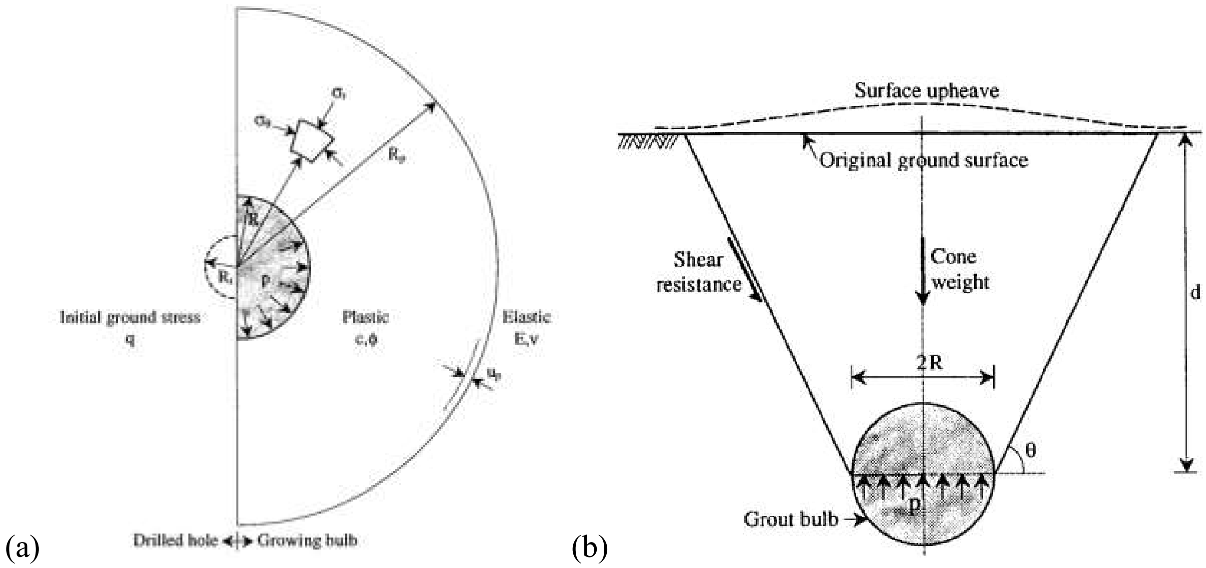


Figure 2.14. (a) Growth of bulb (b) Conical shearing failure above bulb (El-Kelesh et al. 2001)

The overall quantity of mortar to be incorporated is determined according to the desired soil improvement, more precisely according to the average soil density to be achieved. It is generally expressed in the form of an incorporation rate, varying between 2 and 12% of the volume of the soil treated, with an average of 5-6%. The quantity of mortar is then distributed evenly over the surface to be treated, according to a mesh. This mesh is defined according to the extent of the area of influence that can be reached in the terrain concerned, with the injection conditions provided, and taking into account the predetermined quantities. This phase is still arbitrary and empirical. The realization of mortar inclusions with the predetermined quantities and according to the defined mesh leads to an improvement of the intrinsic characteristics of the soil by compaction; and a reinforcement by the presence of a group of mortar inclusions.

Compaction grouting therefore combines the two types of treatment, soil improvement and reinforcement. Depending on the type of application, the nature of the soil and its mechanical characteristics, one of the effects may be predominant over the other.

2.4.2. Grouting materials composition

The grout properties depend on grout permeability (Borden & Ivanetich 1997), aggregate gradation (Warner, 1982), grout filter criteria (Bandimere, 1997), and injection rate (Byle, 1997). In practice, contractors involved in compaction grouting through experience know the proper grout consistency by feel. Many grouters describe the grout “body” and squeezing the grout in their hands when talking about this consistency. This criterion is far from a scientifically repeatable test, but to date is the only way of actually evaluating grout rheology in the field.

Generally, while dealing with grouting techniques three categories in Table 2.1 can be distinguished: (1) also called cementitious, particulate or suspensions grouts have a Binghamian performance; (2) then solutions which can be colloidal evolutive Newtonian fluids in which viscosity progressively increases with time or can be chemical non-evolutive Newtonian solutions in which viscosity is essentially constant until setting, within a controllable period; finally (3) miscellaneous materials which are gas emulsions.

Table 2.1. Principal types of grout (after Camberfort, 1987)

<table>
<tr><th></th><th colspan="3">Suspensions</th><th colspan="3">Solutions</th><th rowspan="2">Aerated emulsions</th></tr>
<tr><td></td><td>Unstable</td><td colspan="2">Stable</td><td colspan="3">Chemical products</td></tr>
<tr><td rowspan="2">Grout type</td><td rowspan="2">Cement</td><td rowspan="2">Bentonite + cement</td><td rowspan="2">Deflocculated bentonite</td><td colspan="2">Sodium silicate</td><td rowspan="2">Organic resins</td><td rowspan="2">Cement organic foams</td></tr>
<tr><td>hard gels</td><td>diluted gels</td></tr>
</table>

The grout usually consists of a mixture of silty sand, cement, and water to form a mortar-like material with a slump less than 2in (~5cm). Cement is usually included but is not a requirement and can be omitted. Depending on material availability, the silty sand may be obtained by adding appropriate fines to available sand. Addition of highly plastic clay, such as bentonite, or concrete pumping additives, has generally been found unacceptable and should only be considered under rare circumstances because they may cause the grout to behave as a fluid in the ground. Design of the grout mix must strive to fulfil three competing objectives:

(1) Pumpability must be sufficient to enable the grout to be injected.
(2) The grout must remain as a growing mass in the ground.
(3) Any bleed water must be able to dissipate into the ground (no water around the bulb).

These three considerations are primarily controlled by the amount and properties of the fine particles, which are defined as particles smaller than 0.074 mm (equivalent to a No. 200 sieve). The amount of water in the grout mix and the aggregate gradation are also extremely important. The particle distribution envelope for the aggregate presented on Figure 2.15 is well graded and contains particle sizes ranging from silt to gravel. Aggregate falling within the gradation envelope provides a satisfactory grout, with the larger, gravel size particles resulting in greater injection control and resistance to hydraulic fracturing of the formation.

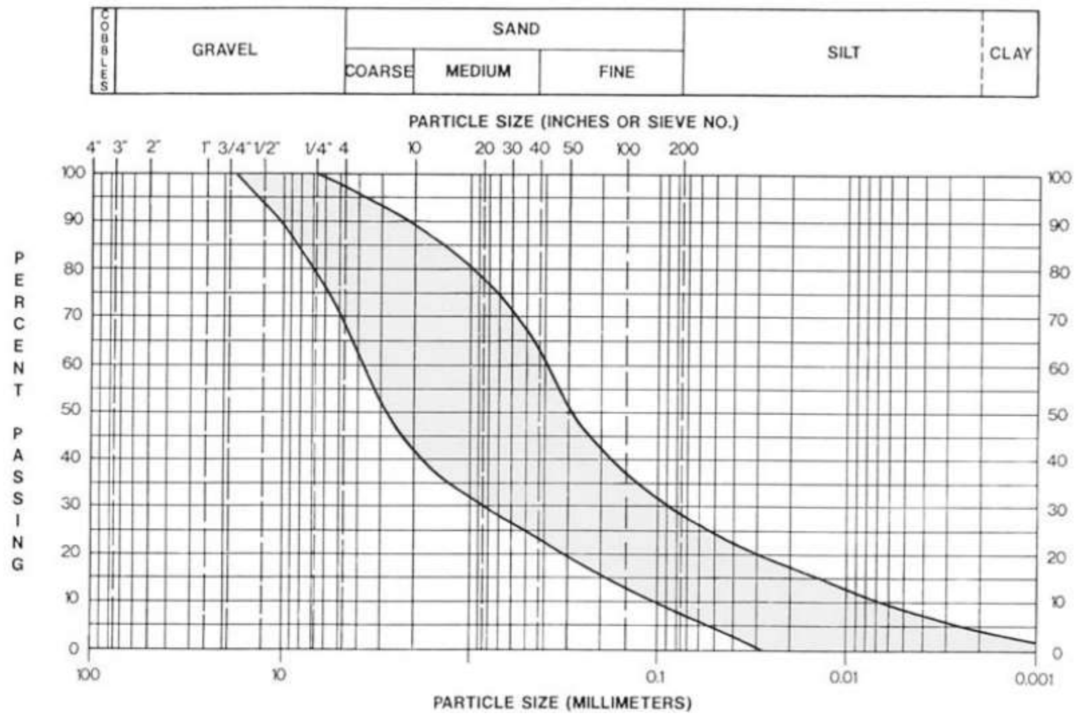


Figure 2.15. Updated envelope of preferred aggregate gradation (Warner et al. 1982)

Round-grained natural deposits should be used, especially for the finer fraction. As an example, substitution of crusher-run dust for the silt sizes generally does not produce a satisfactory grout. Where round- or semiround-grained materials are not available, trial batches should be made to confirm pumpability of the resulting grout mix.

Mineralogy of the aggregates used in the mix also affects the behaviour. It has been found (Bandimere, 1997) that limestone aggregates require a much smaller fines content to make them pumpable. They are easier to control and typically have greater strength, although for most compaction grouting applications the strength of the grout is only required to exceed or meet that of the in-situ soil.

2.4.3. Compaction grouting parameters

Based on subsoil physical factors, it is possible to estimate the grouting pressure, grout injection rate, grout quality and injected volumes required to achieve the desired densification.

2.4.3.1. Geometric description of the grout mass

During compaction grouting, the applied stress at the grout–soil interface exceeds the yield stress of the soil, and permanent displacements occur. As grout is injected, the soil substrate is displaced and plastically deformed in a limited zone around the injected grout. Beyond this zone of plastic deformation, the soil reacts elastically. The relative size of the elastic and plastic zones depends on the amount of grout injected and the characteristics of the soil.

It is uncontroversial to assume, for uniform soil properties, that the expansion of the grout bulb is radial from the injection location because it reduces a complex 3D situation to 1D variable: radius. It is usually assumed that for field grouting applications, the applicable radial expansion is cylindrical for two reasons. First, typical staged grout column construction results in restraint above and/or below the expanding grout bulb. The pre-existing grout bulb acts to prevent vertical grout expansion. Second, for sands and silty sands (commonly grouted soils), the lateral in-situ stress is typically lower than the vertical stress ($K_0 < 1$). Therefore, the direction of least resistance for grout flow is lateral as illustrated on Figure 2.16.

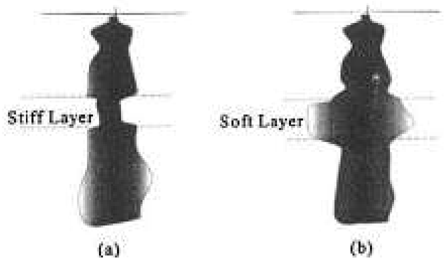


Figure 2.16. Influence of soil layering on column shape: (a) stiff layer causes reduced cross section and (b) soft layer produces enlargement (Byle, 2000a)

So far, the idealizations discussed are only applicable to single grout mass expansions. Most grouting projects involve multiple grout holes, and the soil between these grout holes is affected by multiple grout expansions. The existence of multiple grout holes makes the assumption of symmetric displacements around the grouted hole invalid. Therefore, grouting analyses ought to be conducted using a 3D representation of the grout holes, accounting for the sequence in which the grout holes are to be grouted. To date, this level of analysis is not routinely carried out in general practice, but it should be considered on sensitive jobs (ASCE, 2010).

2.4.3.2. Pressure – volume behavior

Suitable soil densifies itself while injected grout must displace, not permeate or fracture the soil.

i. Pressure – volume of in-situ soil

Change in soil density caused by shearing the soil and increasing the mean pressure is the mechanism of compaction grouting. Soils considered for grouting usually fall within the range of behaviours shown in Figure 2.17 with the conditions measured in drained triaxial compression.

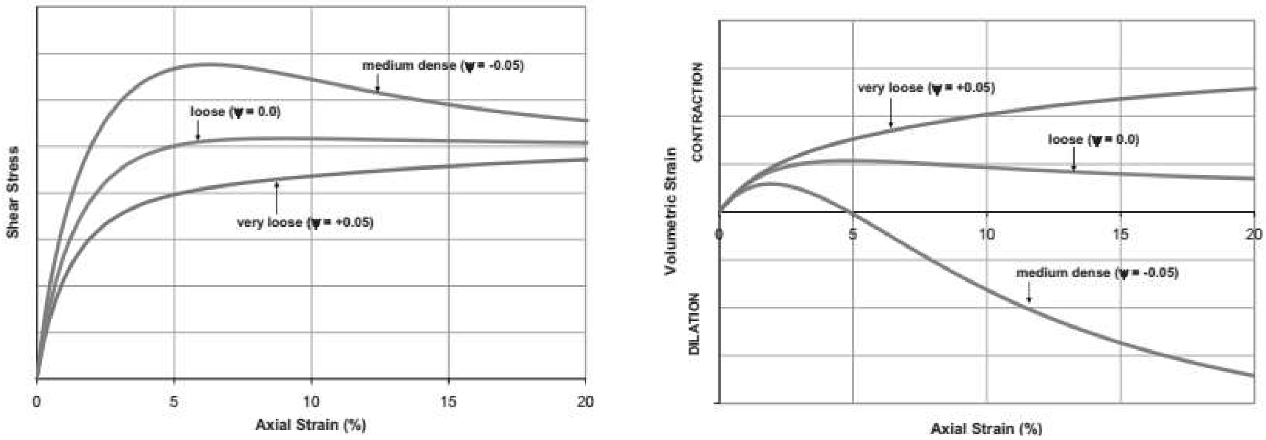


Figure 2.17. Typical soil responses in triaxial compression (ASCE, 2010))

Very loose soils (ψ > 0.05) are contractive and amenable to significant densification during grouting. Loose soils (ψ ≈ 0) are also somewhat contractive because of the change in the critical void ratio as the mean stress increases. Loose to medium dense (lightly dilatant (ψ ≈ −0.05) soils show initial contractive strains followed by the onset of dilation. Whether dilation overcomes this initial compression depends on the mean stress increase and the soil's compressibility. The soil's material properties, strength for example, affect the relationship between the stresses and strains that develop, but the basic pattern that remains is that shown in Figure 2.17, and this is essentially a function of the state parameter (or relative density).

ii. Pressure – volume of grout

Despite the practical importance of the grout mixture, no analysis to date has looked at the grout behaviour itself. Grouting is assumed to be an essentially inert way of applying pressure to the soil, at least for analysis of grouting effectiveness.

It is important not to lose sight of the limitations that this assumption places on any analysis. The lack of understanding of grout behaviour limits our understanding as to why hydro-fracturing occurs in some instances and not others, which in turn limits our ability to develop and improve the grouts used. We are dependent on accumulated experience alone, from which it is dangerous to deviate. On the other hand, it is clear that grouts are themselves soil mixtures and that water may move out of them (i.e., bleed) as the grout exits the injection pipe. This means that:

- The volume of the grout reduces after grout fluid permeates the in-situ soil; therefore, the grout bulb size is overestimated if the pumped grout volume is used.
- The boundary condition at the grout–soil interface is usually assumed in analysis to be impermeable. This assumption is inaccurate if bleeding occurs, and this permeable grout–soil interface leads to a pressure loss caused by seepage of pore fluid beyond the grout bulb. The

pressure loss is a function of the internal permeability of the grout, the size of the grout mass injected, and the permeability of the soil. The mechanics and nature of this grout–soil interaction has not been addressed in the literature, and no accepted means of estimating it exist. Therefore in practice, the grout pore fluid pressure losses are usually ignored.

2.4.3.3. Injection parameters

During grouting, it is important to measure injection rate and injected volumes required..

i. Injection sequence

Compaction grouting can be effective for sandy gravels to low-plasticity clayey soils when these soils are loose to medium dense The procedure has been successfully used in high-plasticity clays; however, intermittent pumping and very slow injection rates are both required, which becomes expensive and limits such work to special requirements.

Analyses on soil behaviour during grout injection have concentrated on the ground response to grout in a single hole. However, these same analyses also show a great dependence on boundary conditions and particularly the distance to stiff ground. This dependence means that grout hole spacing, and whether it is a primary or secondary hole, is of prime importance. The secondary holes already have densified ground around them (caused by primary grouting), and this denser ground provides a reaction that enhances the effectiveness of the grouting. Although by no means fully explored yet, analyses to date suggest that a disproportionate amount of grouting effectiveness arises from the secondary holes (ASCE, 2010).

Grout hole sequencing is therefore an important control. The optimum arrangement of grout hole sequencing is not known, but experience shows that isolation of the area to be improved and the typical primary and secondary injection sequence are effective.

To analyse multiple holes at the same time requires a 3D software package. Hence these analyses are rare. To account for the stiffness of the pre-existing grout holes in a typical 2D axisymmetric analysis, cylindrical cavity expansion of a single grout hole is analysed with a stiffer soil added at the radius of the primary grout holes to mimic the increased resistance.

ii. Injection spacing – meshing

The required spacing of injections is determined based on the zone of influence, degree of control required, and purpose of the grouting. The spacing of grout injections has been reported to range from 2.4 to 3.6 m (Warner 1982); 2.5 m (Bandimere 1997); and 1.8 m (Stilley 1982). However, too high a grout injection rate can detrimentally influence the amount of grout injected at an individual grout hole, thereby increasing the number of grout holes required (i.e. closer hole spacing). At higher injection rates, the in situ soil may behave in an undrained manner. An undrained in-situ soil response results in poorer densification, and the limit pressure is reached at lower injection volumes. Hence, as the soil becomes more undrained, less densification occurs at each grout hole.

In general, the selected spacing should be determined from site-specific conditions. The sensitivity of the site to movement and the soil in-situ dilation and contraction behaviour determine the largest injection that can be safely made. The diameter of the injected mass is dependent on the starting density, required improvement, and rate of injection. Grout hole spacing is typically on the order of three to six times the predicted mass diameter (ASCE, 2010).

iii. Pressure losses

Controlled grouting requires that injection pressures and flow rates be measured (and recorded and displayed) during grouting. Practically, this fact means that a pressure gauge and preferably transducers must be located at the surface. Analyses, on the other hand, start with the expansion of the grout bulb at the base of the injection pipe. There is then a disconnection between what we can practically measure and what we are analysing, the disconnect arising from what happens to the grout as it moves down the grout pipe.

Not all of the pressure applied by the grout pump is reflected as stress on the soil. Pressure is required to push the grout through the grout lines, as can be observed by the pressure difference measured between an in-line pressure transducer and a transducer at the injection header (Geraci, 2007). The losses depend on the length and configuration of the injection system (e.g., hose or pipe, size, number and radius of bends and elbows, and the diameter and wall type), the rate of injection, grout rheology, injection pressure, and other factors. Three main sources of pressure losses are injection losses, line losses, and pressure loss caused by seepage of pore fluid beyond the grout bulb. Injection losses and line losses are primarily related to equipment and grout properties. An additional pressure loss caused by seepage of pore fluid. Because no accepted means of estimating this loss exist, in practice it is usually ignored.

The usual approach for measuring the injection and line losses is to measure the pressure loss in the pipe by laying a length of pipe out on the surface and pumping grout through it. This measurement then gives an offset pressure used to relate the grout bulb pressure to that measured at the surface (after also allowing for a calculated pressure head from the weight of the grout). Where deep grout holes are required, the surface approach to calculating line losses contains some errors because the pressure in the grout near the surface differs from that at depth, resulting in different saturation and void ratio. Despite this lack of detailed information, for many soils the pressure loss appears to be small compared to the injection pressure, and hence, the conventional approach of a simple pressure offset between hole collar and injection point is reasonable (ASCE, 2010).

2.4.3.4. Measuring refusal

Many factors may affect the grout that can be safely and controllably injected in the soil.

i. Grout quantities

Estimating the quantity of grout required is a daunting prospect for most grouting projects. By far the largest complaint of owners is that the design consultant underestimated the grout quantities. The grout quantity can be reliably controlled and predicted only if sufficient in-situ soil information is available, if the grout consistency and injection methods are properly specified to limit mobility of the grout, and if the engineer takes an active role in quality assurance, control of the injection process, and verification of quantities during grouting (ASCE, 2010).

To achieve densification during compaction grouting, it is first necessary to ensure that during shear the volume change of the in-situ soil in the zone of influence is predominantly contractive. Although not appropriately treated by compaction grouting, dense soils dilate during grouting, resulting in a looser soil.

If compaction grouting is appropriate, a simple approach to roughly estimating the quantity of grout needed for densification is to compute the volume reduction required to achieve a target density. Consider the example on Figure 2.18, it is desired to obtain a minimum dry density of 1,600 kg/ m^3 for the full thickness of the fill in layer 2 under the building footprint and 3 m beyond the building limits. Increasing the soil dry density from 1,400 kg/m^3 to 1,600 kg/m^3 would require a 14.3% increase in density.

From assuming that the mass remains constant and knowing mass = volume × density, if the initial and final volumes are Vol_i and Vol_f, respectively. Then,

$$\mathrm{Vol}i \times 1{,}400 = \mathrm{Vol}f \times 1{,}600$$

Giving, $$\mathrm{Vol}f = 0.875\,\mathrm{Vol}i,$$

This increase in density corresponds to a 12.5% reduction in volume of the soil mass. This volume reduction is achieved by displacing the soil with grout. To compute the volume of grout required, multiply the volume of soil to be improved by 12.5%.

Volume of soil to improve is Vs = $16\text{ m} \times 16\text{ m} \times 3\text{ m deep} = 768\ m^3$

Total volume of grout needed is Vg = $768\ m^3 \times 12.5\% = 96\ m^3$

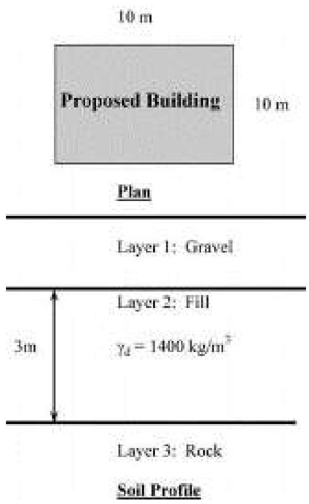
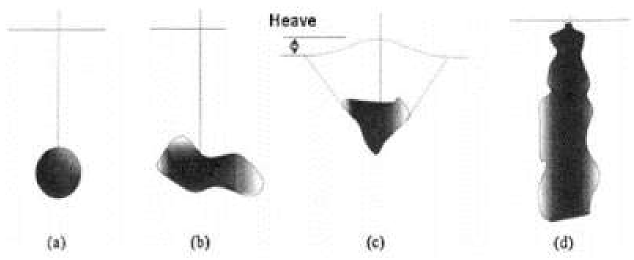


Figure 2.18. Example compaction grouting problem

Typically, this amount would be injected in vertical holes spaced 2 to 4 m apart. Assuming a 2.5-m hole spacing, this process would require approximately 41 grout holes injected at the average rate of 0.78 m^3 of grout per linear meter of injection hole.

ii. Soil confinement

Grout, even stiff grout, conforms to the stress and stiffness conditions in the soil. Homogeneity and isotropy conditions affect the shape of the injected grout mass and thus the control of injection. Nonhomogeneous and anisotropic subsurface conditions, by and large, yield nonhomogeneous and anisotropic grout masses. Where there is little confining stress, very slow pumping rates must be used. If the rate is excessive, the surface of the ground may heave and/or fracturing may occur.



(a) Ideal sphere at depth in homogeneous granular soil. (b) Irregular shape in non-homogenous stratum or filling a void. (c) Quasi-conical shape caused by lack of overburden confinement. (d) Columnar shape by controlled staged injection.

Figure 2.19. Injected grout shapes (Byle 2000)

Some typical grout column shapes due to soil layering are illustrated in Figure 2.16. Other unusual shapes may result from various combinations of grout properties, stress states, stratigraphy, and subsurface obstructions on Figure 2.19.

iii. Influence on structures

In other circumstances, the aim is to densify the soil without affecting overlying or adjacent structures. Movements induced in the soil by compaction grouting are controllable, and the magnitude of any such movements depends on the grout injection rate. The volume of soil displaced is slightly less than the volume of grout because of the escape of pore fluid from the grout and volume changes associated with compression of the grout during injection and curing. However, in application, the grout and displacement volume are generally assumed to be equal. The magnitude of the error associated with this assumption is believed to be small; however, that theory has not been verified (ASCE, 2010).

Movements in soil may occur in all in 3D. It is important to consider the 3D nature of movements because they can affect buried utilities, retaining walls, septic tanks, vaults, and other buried structures in addition to structures bearing on the grouted area. Uplift is usually localized and centered about the point of injection.

Surface heave is a frequent refusal criterion and typically limited to between 1.2 and 2.5 mm per stage. It is important to consider the cumulative surface response resulting from grouting at multiple stages, including the potential influence from neighbouring injection points. When carefully controlled, this effect can be used to recover settlement of supported structures and restore grades. However, it is important to limit surface uplift to an elastic or "recoverable" range when uplift is not desired or could otherwise damage overlying or adjacent structures.

Another technique historically used to limit the cumulative effect of surface uplift involves the use of top-down grouting, also referred to as stage-down grouting. This process is most effective in soft, near-surface soils because the grout-induced heave tends to settle back between stages. Top-down grouting is a more costly process because of the need to redrill the grout injection holes after each stage, but it may be the most effective way to restore a settled structure to grade where the faulty soil is at a shallow depth (ASCE, 2010).

2.4.4. Method of application on site

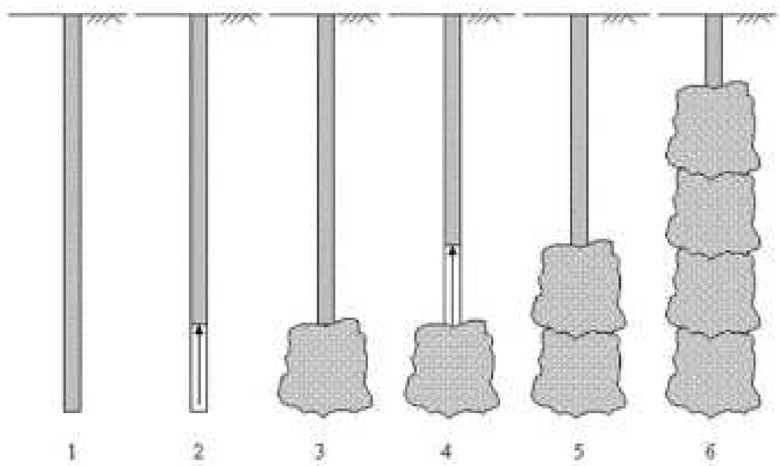
Field application of compaction grouting highly depends on the grouting methods and equipment.

2.4.4.1. Compaction grouting methods

The grout material is pumped into an open segment of borehole termed a stage. Each stage is typically on the order of 0.3 to 0.6 m, although up to about 1.8m stages have been used successfully at greater depths. A casing is installed tightly in the borehole, with little or no annulus, so that grout is forced to expand radially and is restrained from traveling upward along the pipe. Mobility of the grout is dependent on the grout mix and the rate of injection. The grout must behave essentially as a growing solid when it enters the soil. It must have sufficient internal friction so that it behaves as a continuous mass and does not induce hydraulic fracturing of the soil. The rate of injection is important to allow dissipation of pore pressures (ASCE, 2010).

Grouting may be accomplished by either the "stage-up" or "stage-down" method (also called "bottom-up" and "top-down").

- **Stage-up method**: as the name implies, stage-up grouting is accomplished by installing the casing through the full depth of the zone to be treated and grouting from the lowest stage upward toward the ground surface (Figure 2.20). For each stage, the casing is withdrawn a distance equal to the desired stage length and grout is injected until the established refusal criteria are reached. Stage-up grouting has substantial advantages in that it is the simplest, fastest, and least costly method. However, when grouting is used as a means of lifting foundation elements from relatively shallow (1–3 m) depths, the stage-up process can be difficult to control, especially when near-surface soils are particularly weak.



1) install casing, 2) retract casing to top of deepest stage, 3) grout first stage, 4) raise casing to top of next stage, 5) grout next stage, and 6) repeat steps 4 and 5 to top of improvement zone.

Figure 2.20. Stage-up grouting procedure (ASCE, 2010).

- **Stage-down method**: the process of stage-down grouting forming continuous columns. It has several advantages in that each stage is verified by drilling through it, and the upper stages provide some additional confinement for the deeper stages, allowing shallow injection and providing a means of limiting surface uplift to a smaller or more focused plan-view radius surrounding the point of injection. Because it requires more drilling, it is slower and hence more costly.

2.4.4.2. Equipment

The ASCE Compaction Grouting Consensus Guide (2010) reported that the equipment used in compaction grouting consists of a mixer, a pump, hoses, pipes, and gauges. Additional components are the drilling equipment and casing extraction apparatus:

i. Grout mix batch

Grout mixing may be accomplished in individual batches or by use of a continuous mixer. Grout is commonly mixed on site using batch- or auger-type continuous mixers. Obtaining grout from standard batch plants in ready mix trucks is sometimes contemplated for large projects. Concrete plants, however, do not stock the special aggregate used for compaction grouting, and even if they did, because of the fines content, it may not batch properly. Off-site mixing requires special measures because stiff compaction grout does not mix or readily flow from the drums of typical mix trucks. Also, unless multiple pumps are operating, the grout must be delivered in relatively small quantities and possibly a retarder added so that it does not set during the relatively slow injection process.

ii. Pumps

Positive displacement piston pumps are used for compaction grouting. They must be capable of pumping stiff grout at pressures up to about 10.35 MPa at variable rates from 0 to 57L/min. Most compaction grouting is accomplished with conventional small-line concrete pumps with material cylinders 4 in. in diameter or smaller. Use of concrete pumps with larger cylinders has been attempted but not found to be suitable because of non-uniform output when operating at the very slow pumping rates required for compaction grouting. The pump can be for simple injection or multiple injection types as on Figure 2.21.

Figure 2.21. Pump for Multiple Simultaneous Compaction Grouting (Hyobum et al., 2019)

iii. Hoses and fittings

Either high-pressure hose or a combination of hose and rigid steel delivery lines are used. They most often have a 50mm inside diameter, although both 40 and 75 mm are sometimes used. Because the grout flows as an extrusion, all couplings and fittings should be the full inside diameter of the line. Wide-sweep bends should be used rather than standard elbow fittings. Clamp-type couplings, as used in concrete pumping, are generally used. To ensure fresh grout at the grout hole, a reasonable velocity of grout through the delivery system is required. Use of delivery lines larger than 50 mm inside diameter should generally be avoided, especially where long lines are used and/or the environmental temperature is high. A problem unique to compaction grouting is swelling of flexible hose lines when under sustained high pressure. The hose typically used is that manufactured and marketed for concrete pumping. In that use, the outlet end is always at zero pressure, whereas in compaction grouting a positive head pressure always exists. Furthermore, compaction grout flows through the delivery line as a stiff extrusion. A large pressure loss thus occurs when the expanded extrusion encounters the more rigid couplings and fittings in the delivery lines. Couplings and connections in the grout line should have constant inside diameter and should not protrude into the grout flow. They must be watertight under the full range of grout pressures to be used. Even minor leakage allows water to be squeezed out of the passing grout, which can result in plugging of the line.

iv. Casings

The grout casing should be of sufficient size to permit free flow of the grout with minimal head losses. Pipes smaller than 45-mm internal diameter are generally not used because they are prone to blockage. The casing must fit snugly into the hole to prevent grout from flowing up the annulus around it. Casing with an outside diameter of 65 mm or smaller typically has sufficient friction, or can be wedged, to retain it in the hole during grouting. Because of their larger end area, they generally require some means of external restraint to prevent being forced out of the hole by grout pressure acting on the end of the casing. Anchorage of larger casing is sometimes accomplished by using the weight of a drilling rig or frontend loader as a reaction. Individual joints of casing used in upstage injection should be no longer than about 1.5 m to facilitate removal during injection, except in the rare instances where withdrawal is accomplished with a drill rig or similar machine. It must be of sufficient strength to withstand the high withdrawal forces required. Additionally, it must also withstand the forces of driving when so installed. Conventional casing manufactured for core drilling is thus generally not suitable for compaction grouting. Experienced contractors often use proprietary casing that has been proven to withstand the forces involved, generally supplied in lengths of about 1 m. Where casing is installed by driving, or self-drilling, a disposable tip is used. The tip may be a point or cap for driving, but it may have teeth or a fishtail plate to aid in drilling. The tip may also have openings to permit fluid circulation where water is used as a lubricant for circulation in the drilling. The casing must be withdrawn slightly and the tip knocked out before grouting. For downstage progression, the holes are drilled. This process is usually accomplished with rotary-wash drilling, using either a small drill rig or a handheld boring motor.

v. Headers

The header is a pipe that connects the grout supply hose to the casing and provides a place to mount a pressure gauge and/or transducer. The header for grouting is a pipe of the same diameter as the casing with a 90° large-radius bend and a port for attaching a gauge saver and pressure gauge. The radius of the bend should be at least three times the pipe outer diameter to limit constrictions in the grout flow path. There should be no change in diameter between the pressure gauge and the casing because the constriction would interrupt the flow and could cause errors in the indicated gauge pressure.

vi. Pressure gauges

Pressure gauges should have a dial with a minimum diameter of 75 mm to be plainly visible. They must be provided with a gauge saver or other means to prevent contamination with the grout.

2.4.5. Grouting effectiveness verification

To be sure that the grouting operation was well done, an effectiveness verification has to be done at the post – grouting construction phase with adequate monitoring systems.

2.4.5.1. Verification importance

The success or failure of a grouting job is related to human as well as technical factors. It is most important that before grouting (1) the problem has been defined in sufficient detail to the satisfaction of everyone involved, (2) agreement has been reached on the criteria that define success or failure, and (3) data that may be necessary for before and after comparisons are gathered before grouting starts (ASCE Committee on Grouting, 1995). The accuracy of this grouting technique is related to how accurately the properties of the in-situ soil are known and the appropriateness of the analyses.

Grout formulations, injection rates, pumping pressures, and grout takes for each stage of each hole should be continuously monitored, recorded, evaluated, and appropriate changes should be based on the data retrieved during the work. The most important task to verify controlled injection of the intended quantity of grout in the required location is vigilant observation of the injection rate and pressure behaviour during grouting. A substantial advantage of compaction grouting is the ability to place grout in the intended location. When grout of proper rheology is injected in an appropriate manner, the grout forms an expanding, generally consistent mass.

2.4.5.2. Monitoring importance

The efficiency of a grouting operation can usually be determined as the work proceeds through careful monitoring combined with prompt corrective action when indicated.

The soil into which the grout is injected is opaque to human vision, so the flow of grout into or through the formation cannot be monitored by direct observation. The final location of grout is controlled to some degree by the practices and procedures used. The increase in soil strength or some indirectly related soil parameter is often measured to confirm the degree of improvement in the soil strength. It is often assumed that if the grout sets in the desired location, the job is successful. Under this assumption, success of the grouting operation can be measured by field procedures such as penetration tests to assess the in-situ density or through geophysical methods that verify the grout location (ASCE, 2010).

Unknown or unanticipated underground conditions may adversely affect grouting performance. Detailed geotechnical investigation is the best method to avoid unnecessary surprises.

2.4.5.3. Verification methods

In planning an effective program, the properties to be measured must be clearly defined. Verification of compaction grouting typically involves indirect methods that are correlated to density. The cost of any method of verification should be balanced against the benefit to be gained. For compaction grouting, these answers should be given: (1) the degree of densification is required; (2) the verification test results evaluated; (3) the acceptance criteria; (4) the consequences of failure and (5) the cost for the verification. A good verification program is cost effective, provides an appropriate level of assurance, provides results in a reasonable time frame, and uses the simplest, most reliable technology possible.

i. Verification by design

Verification methods should be selected consistent with project goals, and engineers should incorporate the verification into the design and constructability evaluation. Specifications should be established for the grouting, monitoring, and verification testing to provide the required level of quality assurance. The verification equipment and procedures should be made integral to the grouting program and other construction activities on the site. Consider the construction site environment (narrow or free areas) and available equipment which can be: geophysical direct methods (ERT, CSW or SASW) or geotechnical indirect methods (CPT, SPT, SCPT or DPH). If a test section is desired, include it in the specification and include verification testing for the test section.

ii. Test or Pre-construction grouting

On large or sensitive projects, preconstruction grouting is an efficient way of optimizing the grouting procedures and can be cost-effective. On most projects, a full preconstruction program is not warranted. However, every project benefits from reviewing the information gathered during the first few grout holes to see if grout hole spacing, grout pressures, and grouting rates could be adjusted to improve the end result. Where used, test grouting should include the same verification measures intended for the production grouting. Excavated test pits may be used to calibrate and verify the results of indirect tests. The project schedule should include sufficient time for the completion and evaluation of the test grouting before production grouting. It is of much less value to get the verification report indicating unacceptable areas after the grouting contractor has demobilized from the site than to get real-time data.

iii. Monitoring during construction

As a minimum, the grout consistency, injection rate, injection pressure, and injected volume in each stage should be monitored and recorded. These data should be reviewed to evaluate the ground and grouting performance. Injection pressures are lower, and injected volumes higher, in softer or less dense areas. A uniform pressure build-up at a given pumping rate indicates generally uniform soil conditions and controlled densification. Gradual pressure changes indicate non-uniform soil, whereas a sudden pressure drop signals hydraulic fracture of the formation and loss of control of the injection.

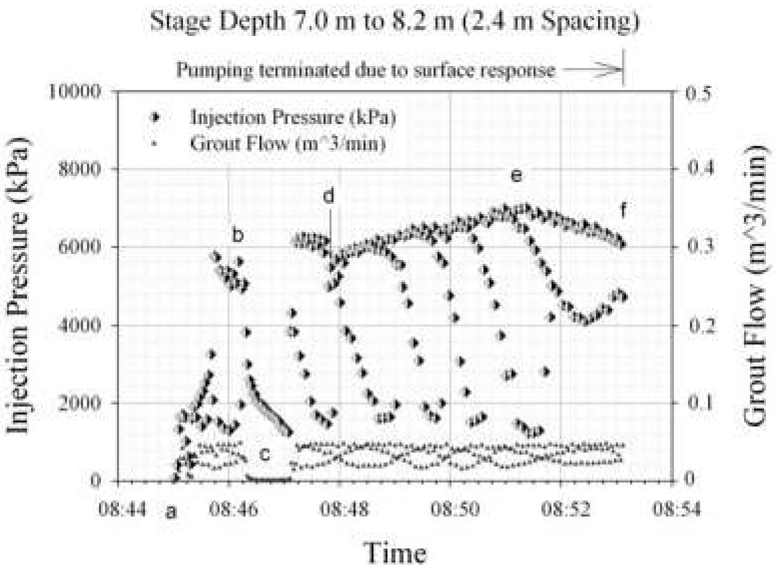


Figure 2.22. Time history of measured pressure and flow during grouting (Geraci, 2007)

The equipment is essentially the same as that used for cement grouting: a flow-meter, electronic pressure transducers, and a PC-based data acquisition system. Others authors described the use of near-real-time analysis in this way for the remediation of the Bennett Dam. Additionally, monitored injection pressures and grout flow rates could provide early warning of surface heave (see reduction in pressure with constant grout flow rate between *e* and *f* on Figure 2.22).

Conclusion

At the end of this chapter, the objectives were (1) to identify the possible damages a structure can undergo due to a sinkhole collapse beneath the building foundation with a review on monitoring techniques for vertical, horizontal displacements and angular distortion. Then, (2) to enumerate the various underpinning methods such as classical concrete mass, micro-pilling, jet grouting and compaction grouting techniques with the presentation of the principle, the design method, the field application and the density verification at end. To conclude, since the sinkhole is unpredictable, developing adequate monitoring techniques of the structures could help understand how and why fissures, cracks and settlements are going with time in order to avoid the collapse. This collapse could be mitigate with the compaction grouting as a soil reinforcement techniques.

CHAPTER 3: METHODOLOGY OF WORK

Introduction

For each analysis, it is necessary to define a methodology of work. The methodology of work is the part where the work main steps are described and explained. It gives the procedure to follow to attain the aim of work. This chapter will consist on the presentation of the studied area used for the analysis, and then it will be presented the structural damages survey, geotechnical and geophysical investigations done on this site in order to have the soil parameters useful to understand the type of soils beneath the foundations. Then, the design and application methods of the compaction grouting have been clearly explained in order to sustain the foundation structures affected and finally an examination method has been elaborated to analyse critically the compaction grouting as it were applied in the Douala – Bassa industrial zone.

3.1. General site recognition

In order to correctly answer the problem definition of this thesis, many information has been sought about where the project is located and all characteristics (physical and socio-economic) of the Douala – Bassa industrial zone. The site general recognition is done through a documentary research. The essential objective is to know the geographical parameters of the site of the case study, the site location, the climatic conditions (temperature, precipitations), the relief, the population, the hydrology and the economic activities of the site.

3.2. Site visit

During the site visit, some pictures of the project site were taken using a phone camera. The activity in the industrial zone was obtained by questioning the surrounding population and the staff members of Terratest Cameroun. Furthermore, the state of the existing structures was done by eye inspection on the site.

3.1.1. Observations

The infrastructure and the existing structures were identified followed by the relief and the surrounding environment. Observation pictures were taken using a phone camera. Furthermore, the structural damages and the sinkholes were observed and an inspection made on the structures.

3.1.2. Questionnaire

A questionnaire was addressed to the project manager of the company Terratest Cameroun and to the local population of Ndokoti concerning the climate in the region, the economic activities, and the old chemical industries where the ground collapsed.

3.3. Data collection

The project data from the Douala – Bassa industrial zone was given by the company Terratest Cameroun. The data collected are regrouped into 3 categories which are: geotechnical and geophysical data, and grout properties.

3.3.1. Geotechnical data

The geotechnical data were obtained at Terratest Cameroun Company. The geotechnical data are needed to understand the soil stratigraphy, know the soil parameters and detect underground cavities presence. The identification of soil types and their different parameters was done through in-situ tests and laboratory tests and the results necessary for our study are expressed in Table 3.1.

Table 3.1. Parameters to find for the study.

PARAMETERS	SYMBOLS	PARAMETERS	SYMBOLS
Saturated unit weight	γsat (kN/m^3)	Coefficient of consolidation	Cv (m^2/s)
Dry unit weight	γd (kN/m^3)	Admissible stress	σ (kN/m^2)
Frictional angle	φ	Void ratio	e
Cohesion	C, Cu (kN/m^2)	Compression index	Cc
Horizontal permeability	Kh, Kv (m/day)	Recompression index	Cr

3.3.2. Geophysical data

Geophysical data were collected at the same company in order to more understand the underground profiles depending of the soil strata physical properties. The study covers an area of the Douala – Bassa industrial zone including 19 electric tomography profiles of varying lengths depending on the logistics of the site.

3.3.3. Grout data

The grout composition has been also obtained at the same company. The water/cement ratio, the proportions of cement, sand, water and additives and the type of cement were collected. Those data will help to determine the grout behaviour.

3.4. Compaction grouting design method

Designing compaction grouting means take into account some parameters defined by the aims of the grouting campaign. The design method of compaction grouting is highly experience based despite its constant evolution. Indeed as many other soil improvement techniques, a lot of empiricism is needed. Nevertheless, some technical guides recommend to define

- In-situ soils: the density and behaviour type of the soils were used to formulate the suitable grout. The grout must displace, not permeate or fracture the soil.
- Grout rheology: Grout is formulated with various percentages of cement, sand, water and additives. Before being injected, its consistency and manoeuvrability are verified with the slump test (ASTM or EN standards). Furthermore, a good practice consists of also evaluate the grout viscosity by timing the efflux of a standard grout quantity from a normalised cone named Marsh cone.
- Grout quantities: The quantities were not computed based on the volume of voids. The densification was executed by injecting grout until the batch is empty
- Grouting meshing: Since the aim was to fill the underground cavities, therefore no grouting pattern was needed. The injections were then done following the cavities positions with a bottom – top stage.
- Refusal criteria: Since a good grouting have more than one criterion, at each grouting stage, the measured pressure limit with time combined the grout volume limit were used as refusal criteria to uniform and predictable results in compaction grouting.

3.5. Compaction grouting execution method

Concerning the grouting execution, depending on the aims defined in the design phase, parameters such as, the ground surface displacements and the cracks apparition on surrounding structures and the injection pressures were verified to stay in an allowable range.

The compaction grouting execution method was performed as a stage-up grouting process with no meshing. In fact, since the work consists of filling cavities, the meshing was not needed. The grout plasticity was not well verified but was at least its pumpablility was acceptable. The control of surface heave and hydrofracture were not monitored during the injection phase.

3.6. Compaction grouting evaluation

Analysing the compaction grouting efficiency is a huge step of the soil improvement technique which should be performed after the grouting phase. The aim is to compare the pre – grouting phase observations to the results of the post – grouting phase in order to assess the grouting effectiveness.

3.6.1. Monitoring while grouting

During the compaction grouting execution, parameters such as the injection pressure and the injected grout volume were measured. Archaic monitoring techniques were used to measure the possible surface heave. The basic solution used on site was made of common measurement tools such as plumb bobs and carpenter's levels.

3.6.2. Monitoring of structures

During the grouting phase, soils beneath the foundations of structures undergo high pressures due to the injection of grout. This densification soil generate ground surface heave which will cause damages on the surrounding structures. No recordings of these possible structural deformations on buildings have been done in order to evaluate the upheave effects.

3.6.3. Underground density verification

The density verification at the end of the grouting helps to understand how many holes have been correctly filled. This verification aims to check if the pre-defined objectives during the pre-grouting phase have been fulfilled. Underground density verification was not performed on field.

Conclusion

In this chapter, the applied methodology consisted of the design and the critical analysis of the compaction grouting as a soil improvement method. Firstly, the presentation of site recognition and the presentation of site has been done through documentary research and observations. Then, geotechnical and geophysical data collection step to obtain data useful for the design process presented with the detailed design procedures which included the verifications to be done. Finally, the focus was done on the presentation of the various elements necessary to the post – grouting efficiency analysis. At present, it is quite possible to achieve the objective pursued by the present work which is to propose adequate modern techniques to improve the compaction grouting performance and efficiency.

CHAPTER 4: RESULTS AND INTERPRETATIONS

Introduction

After a description of site and presentation of the collapsed sinkholes in the Douala – Bassa industrial zone in this chapter, the previously presented methodology will be applied to critically analyse the compaction grouting done. To do this, the preliminary geotechnical and geophysical investigation campaign results will be presented; they will serve as initial conditions to define the soil in-situ conditions before applying the compaction grouting on field. This will be followed by the presentation of the results of the injection phase and post-grouting phase. The second part of the chapter will consist of a presentation of the results of the reliability evaluation of the monitoring techniques used on site. Lastly, the presentation of the design method limitations and the proposals for improving performance and efficiency of compaction grouting will closed the chapter.

4.1. General presentation of the site

After combining the documentary research and the project data, the data obtained were informations such as the location, climate, relief, population, hydrology and economic activities in the Douala Bassa industrial zone.

4.1.1. Climate

Douala has a tropical monsoon climate, with relatively consistent temperatures throughout the year. Though, the city usually experiences cooler temperatures in July and August, Douala is characterized by warm and humid conditions with an average annual temperature of 27.0°C and 85% of average humidity. The city has plentiful rainfall during the year; the average rainfall is about 4078 mm precipitation per year. Its driest season is between the months of December and February where an average of 28 mm of precipitation falls, while its wettest month is August with an average close to 700 mm of rain falls during this month.

4.1.2. Relief

The city of Douala has a coastal area of around 400km, it is in form of arc constituted of successive sedimentary planes, rivers, and spring throwing in delta form to the Atlantic Ocean. This zone is marked by unstable mangrove swamps. The city is situated on a tray going to 1000m. The morphology change increasingly from coast to the inland.

4.1.3. Hydrology

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, and the city is divides in many watersheds: Epolo, Mbanya, Mboppi, Bologo, Ngoua, Lonmayagui, Kambo, Tongo-Bassa and Besseke.

4.1.4. Geological context

From a geological point of view, the city of Douala is situated as a whole in the sedimentary basin which is made up of deposits from the end of the Tertiary era (Miocene, Pliocene) and the beginning of the Quaternary. With a very humid and hot climate, the alteration is deep and the morphological differentiations are quite similar from one sector to another. The sedimentary section in the basin has a maximum thickness of 8-10 km, based on exploration drilling and gravity and magnetic modelling.

The Cretaceous outcrops have produced sandstones that are either friable, fine to coarse, with intercalations of kaolinitic sandstones, or calcareous or marly, which after alteration present formations of sand, sandy-clay or yellow or variegated sandy-clayey clay.

4.1.5. Geotechnical context

The ground in Douala is made of alluvial deposits of soft soils since the city is close to the estuary of the Wouri. Many site investigation tests show that in this region, the foundation soil is mainly constituted by peats and clayey soils characterized by worst geotechnical characteristics, not suitable for supporting higher loads (low bearing capacity) and high settlements.

An important feature of Douala’s soil is its salt content; close to the sea, the ground water table in this city is contaminated by the sea water. This is one of the reasons why geotechnical studies have to be well done before construction of infrastructures.

4.1.6. Project presentation

The project is located in the city of Douala, Littoral Region of Cameroon, the study area is in the vicinity of Ndokoti crossroad inside the Douala-Bassa industrial zone (Figure 4.1). The geographical coordinates of the centre of the Project are: Latitude 04°02'36.0"N and Longitude 009°44'44.5"E with an altitude of +42 m. The area is on paved ground with some industrial installations such as tanks, cisterns, pipes, machinery, circulation, storage and others. The land is almost flat.

Figure 4.1. Douala town map – Scale 1cm: 13.50km (Source: Google Maps)

The selected site is an industrial, commercial and living environment composed of chemical contents storage tanks, buildings and some commercial points especially the Ndokoti market (Figure 4.2). In order to secure the various investments made in this area, to preserve the lives who daily occupy these spaces which have become risky, and also to ensure the continuous maintenance of these 20 years old structures, a study of the risk zones in the Douala Bassa industrial zone was undertaken. Indeed, the expert study carried out focused particularly on the floors, structures, traffic areas and the sewer network.

Figure 4.2. Traffic jam in Ndokoti principal crossroad – Douala III district

4.2. Description of the site

At the time of our investigations, four areas affected by tank settlements, concrete pavement subsidence and cracks were identified. The affected areas are indicated as zones.

4.2.1. Structural damages survey

The four studied zones are occupied by storage tanks, residential buildings and roads. The following problems were observed in the Ndokoti cross-road vicinity. Observed cracks and fissures in the ground in concrete pavements and around the storage tanks in the vicinity of chemical facilities are illustrate on Figure 4.3.

Figure 4.3. Cracks and subsidence sinkholes on road pavement

The defective drainage system of wastewater on Figure 4.4 caused the formation below of an underground cavity. This obvious cavity was detected on site with almost 1.2 m of diameter.

Figure 4.4. Cracks on road pavement (L) – Aging housing water drainage network (R)

The foundation reconnaissance excavations were carried out using shovels and the opening of an excavation along the foundation of a building to determine the type and visible geometry of the existing foundation, as well as to appreciate the nature of the land that is under siege. The excavations revealed a type of general invert footing, resting on a sandy clay layer yellowish to blackish shown on Figure 4.5.

Figure 4.5. Excavated foundation below a storage tank

4.2.2. Sinkholes collapses survey

The survey work was done to estimate the damages degree on the various structures found in the four (04) were residential buildings, tanks and roads are located. Sinkholes collapses were found in the compound of a residential building (Figure 4.6L) and on a paved circulation area (Figure 4.6R).

Figure 4.6. Ground collapse inspection (L) close to a building (R) on a paved road

4.2.3. Definition of the problem

In the Douala Bassa industrial zone, the studied area in Figure 4.7, most of the existing structures are more than 20 years old age and their serviceability state is no longer verified. This is due to:

- The settlements around the areas where machines and tanks are placed
- The ground or pavement subsidence or sinkhole collapses around vats
- The degradation of the floors and slabs of buildings
- The apparition of fissures and cracks on the old existing structures
- The aging defective water network supply or drainage pipes
- The presence of underground cavities filled of water at a shallow depth (from 0.80m)

And after, the compaction grouting execution, no monitoring has been performed to evaluate the efficiency and effectiveness of the applied method. All those were observed problems in the four zones on Figure 4.7. They could conduct to collapse (ultimate limit state) of around structures and cause very important economic losses.

Figure 4.7. Douala Bassa industrial zone map – Scale 1cm: 13.50km (Source: Google Maps)

In order to avoid those losses, it is necessary and urgent to make a soil investigation, confirm the obtained results with a geophysical method, then choose the right technique to reinforce the soil beneath the foundation since it is weakening due possible underground cavities filled of water. If this water contained carbonic acid solution, it will erode and dissolute the kaolinitic or calcareous soils.

4.3. Data collected

Ground instabilities are mostly caused by sinkhole hazards which are generally the surface expression of the caves collapse and water level fluctuations. Their formation may be either natural or man.

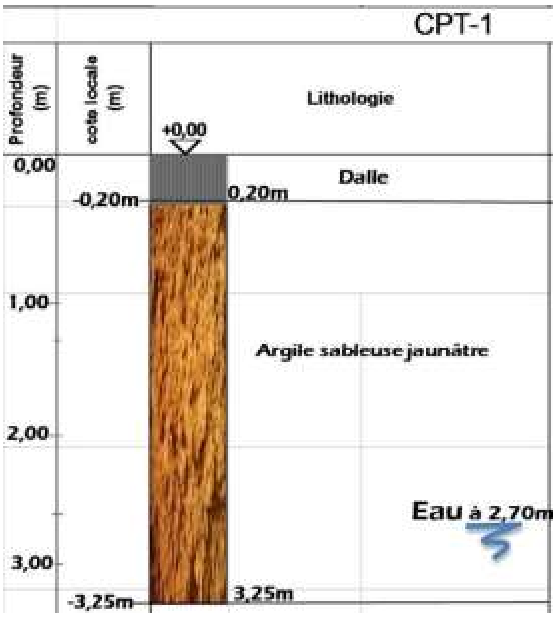
4.3.1. Geotechnical data

Through in-situ geotechnical investigation and laboratory tests, a soil characterization has been done in order to well understand the underground phenomenon causing the voids formation.

4.3.1.1. CPT collected results

The underlying soil is loose or soft under slab up to a depth of 9.20 m. On Table 4.1, the most critical section is found at depth between 5.60 m and 7.80 m from the top of the pavement.

While considering only the low Qc values (0.35 – 1.33 MPa), it can be assumed that the soil beneath the concrete slab is very loose (probably soft clay referring to Robertson chart) to resist to the cone penetration (Figure 4.8). On other side, coupling the analysis with the low Fs values (0.015 – 0.044 MPa), the soil inside the most critical section could be supposed to have underground voids because the soil showed a non-resistive friction to the cone penetration.



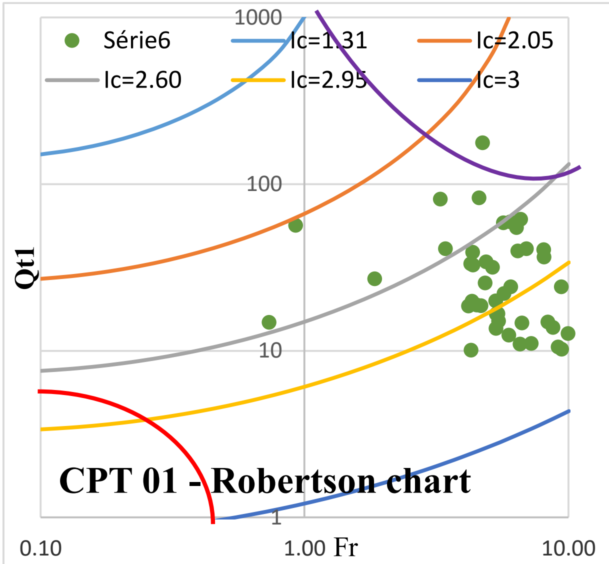
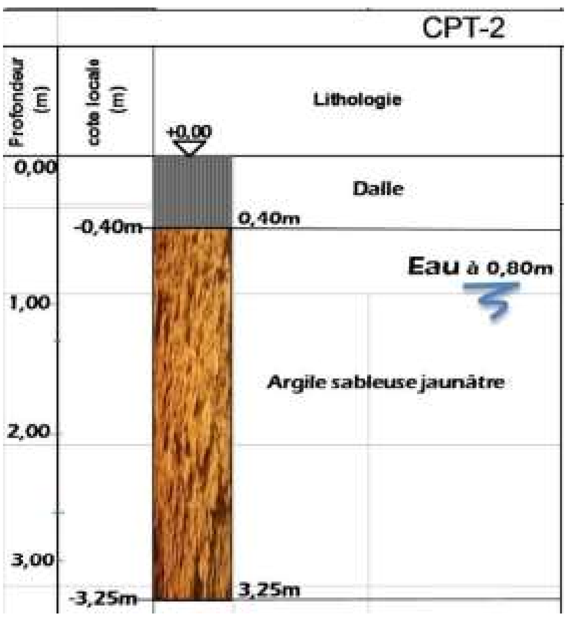


Figure 4.8. Lithology from CPT1 (from project data) – CPT01 Robertson chart

From the data obtained for each CPT have been reported to the following Robertson charts (Figure 4.9) with the objective to more understand the soil behaviour type.



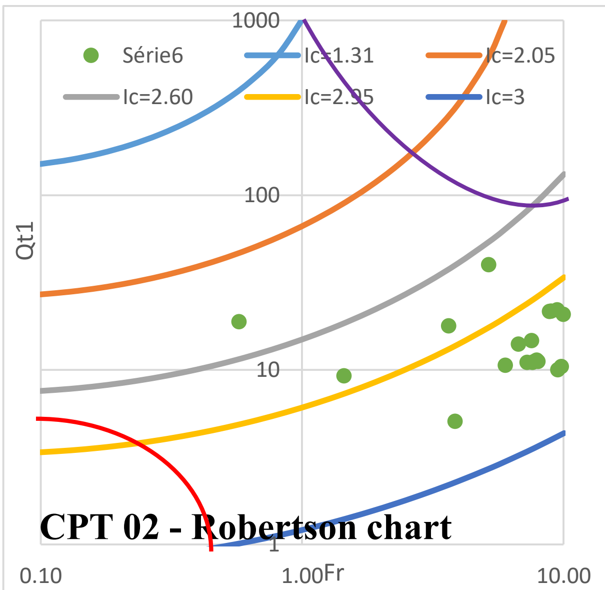


Figure 4.9. Lithology from CPT2 (from project data) – CPT02 Robertson chart

Table 4.1. Mean results of CPT 1 & CPT 2

Depth (m)	Fs (kPa)	Qc (MPa)	SBTn	N_{60}	Nature
0.2	209.08	5.90	5	12	Dense sand
0.4	113.37	0.35	4	1	Loose silt
1.0	100	1.10	4	2	Loose silt
2.60	50	1.20	4	2	Loose silt
3.40	70	1.50	3	3	Soft clay
4.20	70	0.20	4	1	Loose silt
5.60	15.34	0.29	2	1	Soft clay
6.60	36.01	0.74	3	2	Soft clay
7.20	33.89	1.03	3	2	Soft clay
7.80	44.25	0.97	3	2	Soft clay
8.10	88	1.00	3	2	Soft clay
9.20	44.72	1.33	3	3	Soft clay
10.50	160	1.52	3	3	Low consistent clay
12.40	110	2.30	3	5	Low consistent clay
13.80	146.38	3.00	3	8	Low consistent clay
14.40	174.55	5.27	5	13	Dense sand
14.80	238.56	6.30	5	16	Dense sand
15.20	275.55	5.37	5	15	Dense sand
16.60	287.65	7.03	5	19	Dense sand
16.80	585	10.00	5	20	Very Dense sand
17.10	550	9.99	5	20	Very Dense sand

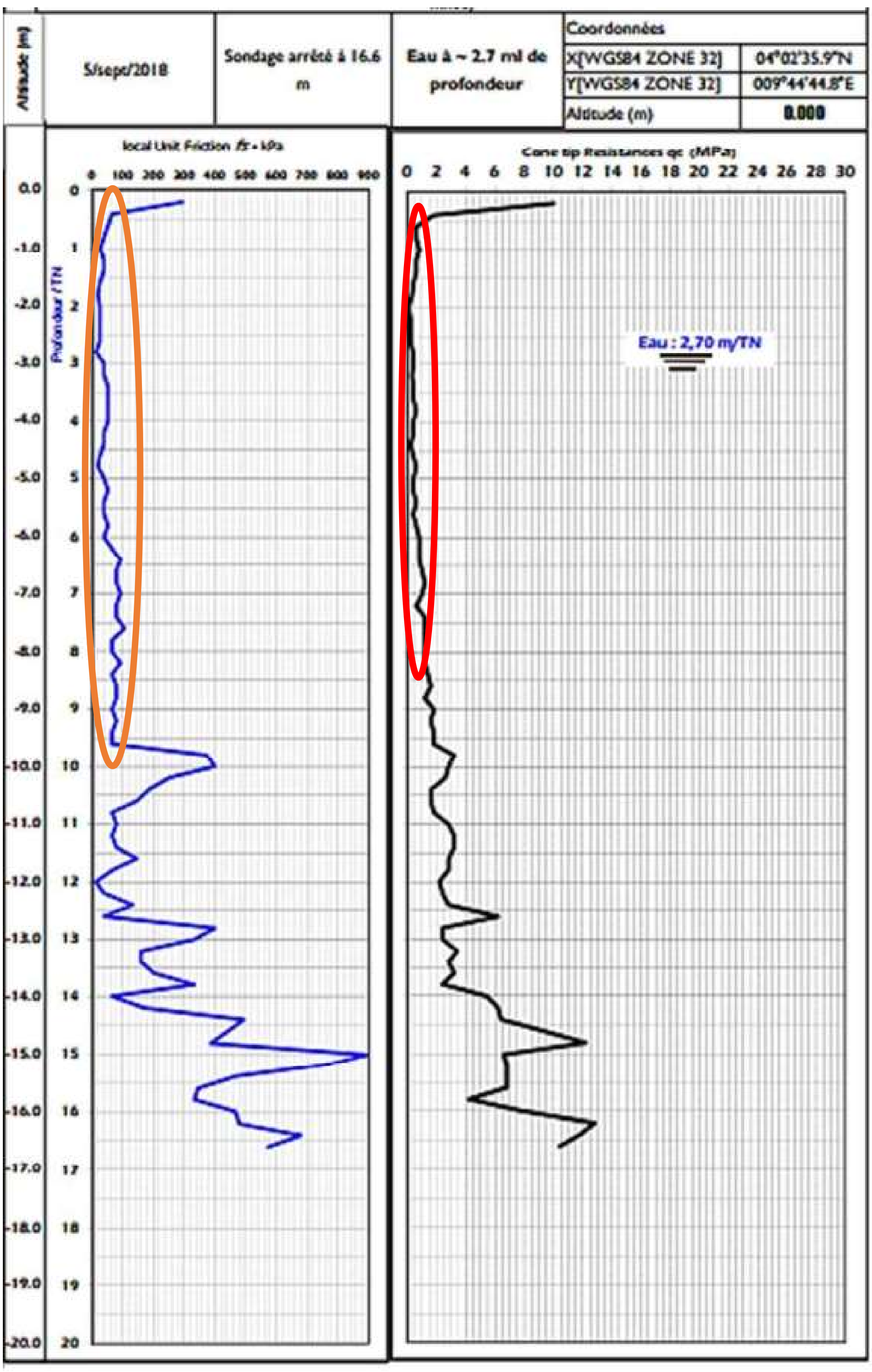


Figure 4.10. CPT1 local unit friction and tip resistance graphs (from data project)

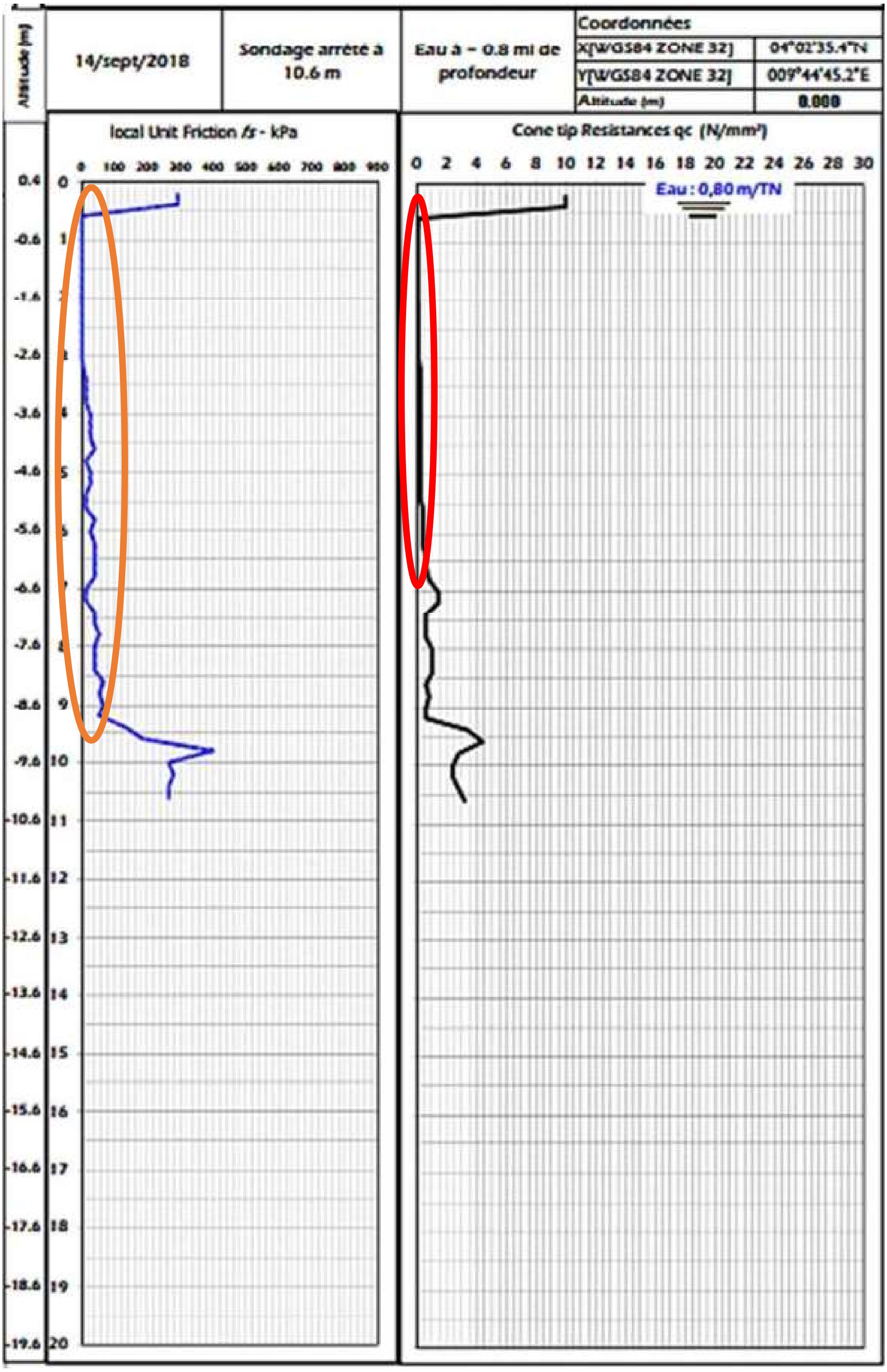


Figure 4.11. CPT2 local unit friction and tip resistance graphs (from data project)

4.3.1.2. DPH collected results

The heavy dynamic probe did not find any resistance while entering the soil with low N60 equivalent mean values between 0.90 – 3.7 m depth (Table 4.2) and the obtained soil sample helps conclude on soil nature which is a mixture of sand and clay.

Table 4.2. Mean results of DPH3 – DPH4 – DPH5 (from data project)

Depth (m)	N_{60} (SPT equivalent mean)	Nature	Consistence
0.30	20.0	SC-ML	Very dense
0.90	3.7	SC-ML	Loose
1.20	1.1	SC-ML	Very loose
7.20	0.90	SC-ML	Very loose
9.60	3.1	SC-ML	Loose
14.10	5.6	SC-ML	Meanly consistent
15.00	11.8	SC-ML	Dense

The most critical section is between the depths 1.20 and 9.60 m where the number of blows is very low. The soil there is then very loose or made of underground cavities (Figure 4.12).

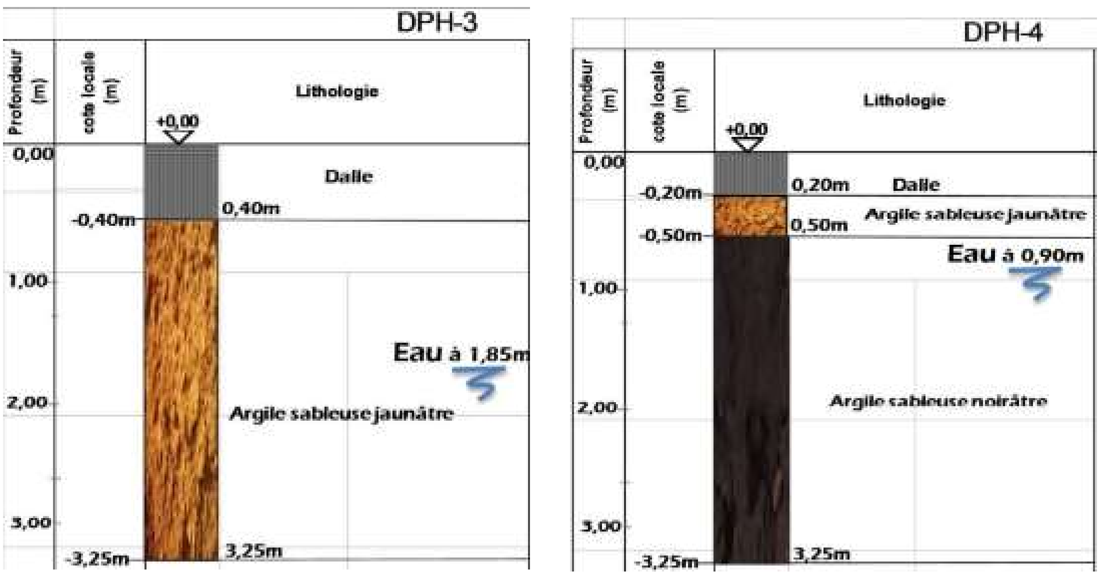


Figure 4.12. Lithology from DPH3 & DPH4 tests

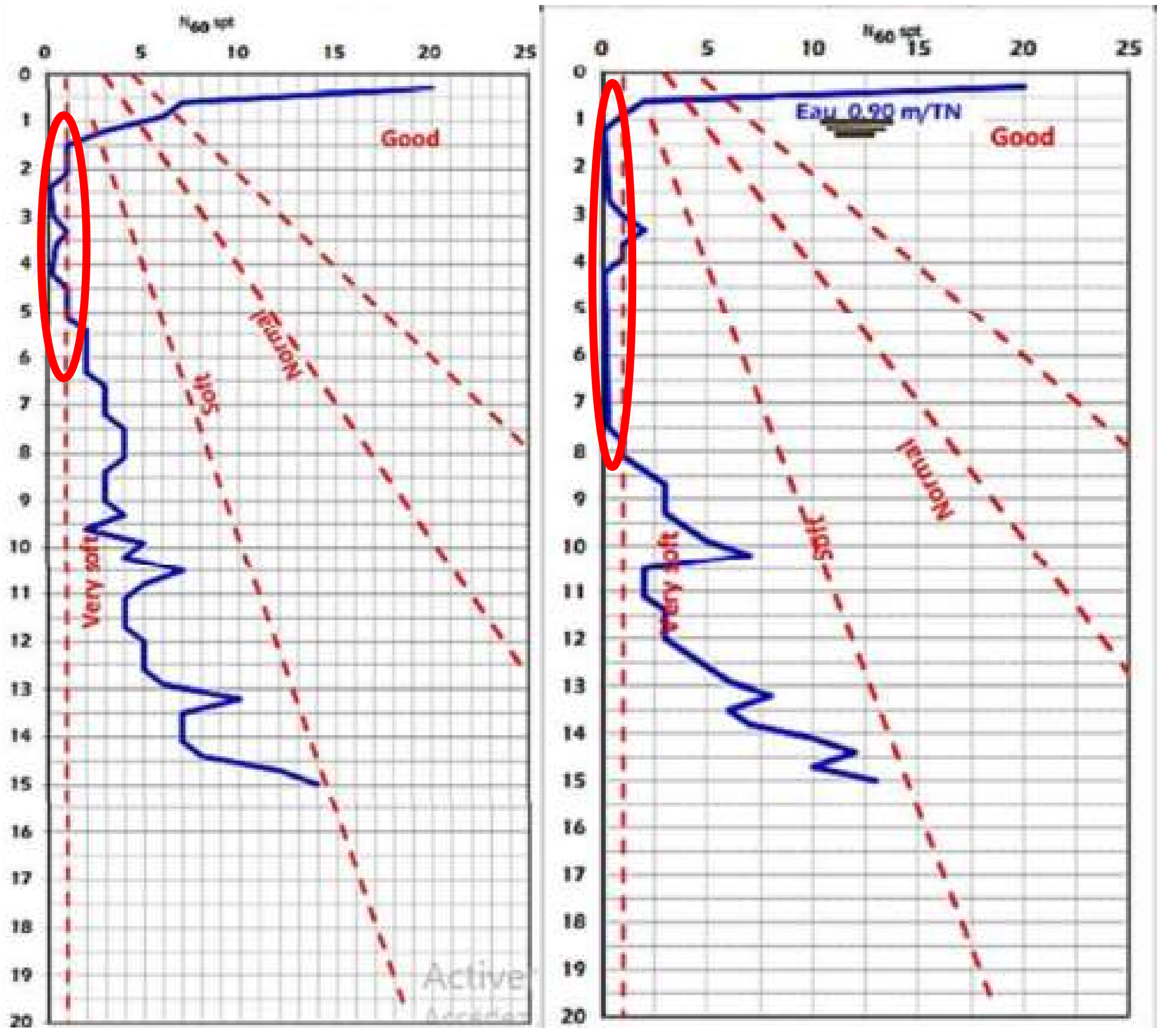


Figure 4.13. DPH3 & DPH4 equivalent number of blows N60 SPT graphs (from project)

4.3.1.3. Laboratory analyses

The collected undisturbed samples obtained (Figure 4.14) help identify normally consolidated soil mostly made of sandy clay. With a water level around 0.7 – 1 m depth, the two undisturbed samples geotechnical characteristics (Cu, Phi, Sr, e_0, LL, PI, Cv, Cc,) obtained through laboratory tests (Table 4.5) help conclude about the lower bearing resistance of the soil beneath the foundations of the studied areas.

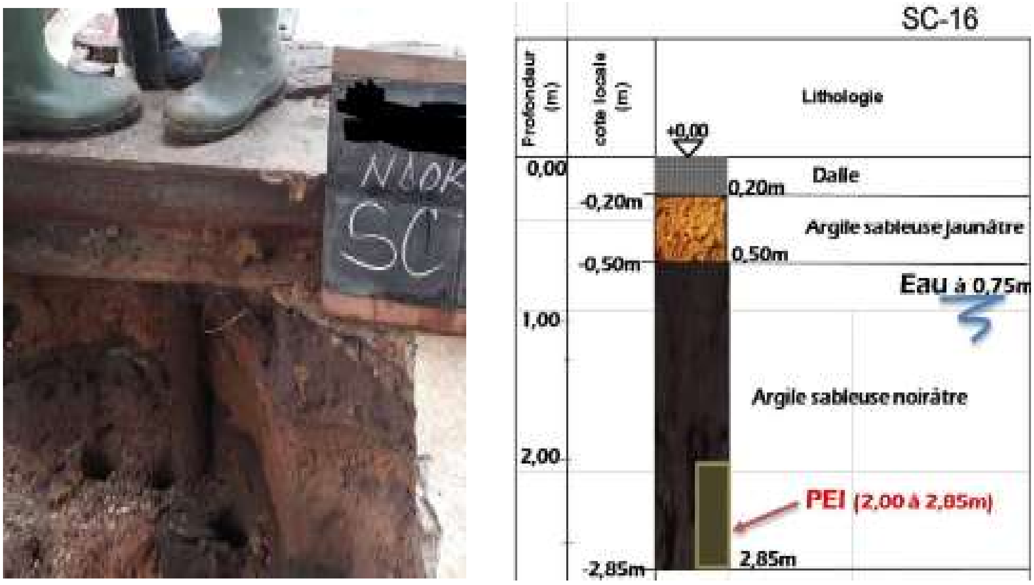


Figure 4.14. SC 16 excavated point & lithology

For the sample SC16, the results of the Atteberg limits are presented in the graph on Figure 4.15 and Figure 4.16. The main results values are Plasticity Index of 24.46%; Plastic Limit of 28.0% and Liquid Limit of 52.40%.

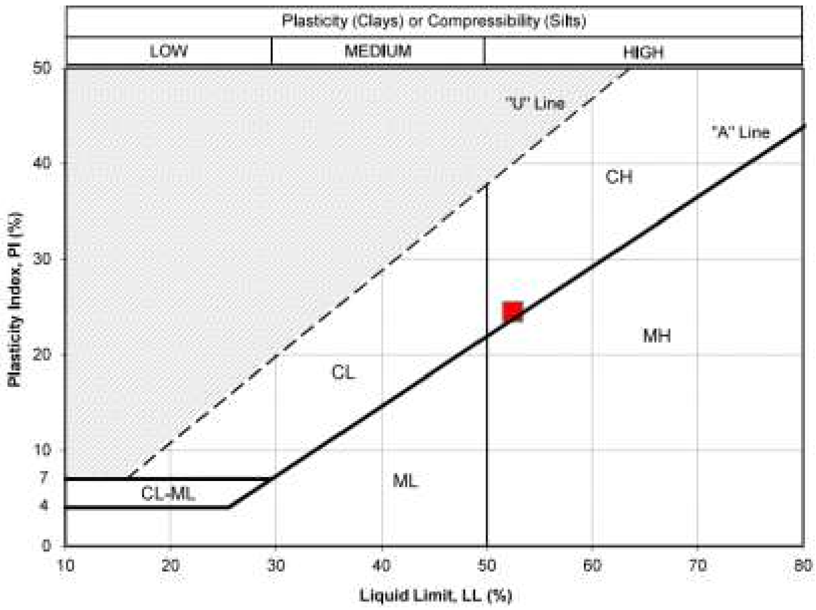


Figure 4.15. SC 16 liquid limit obtained from sample SC 16

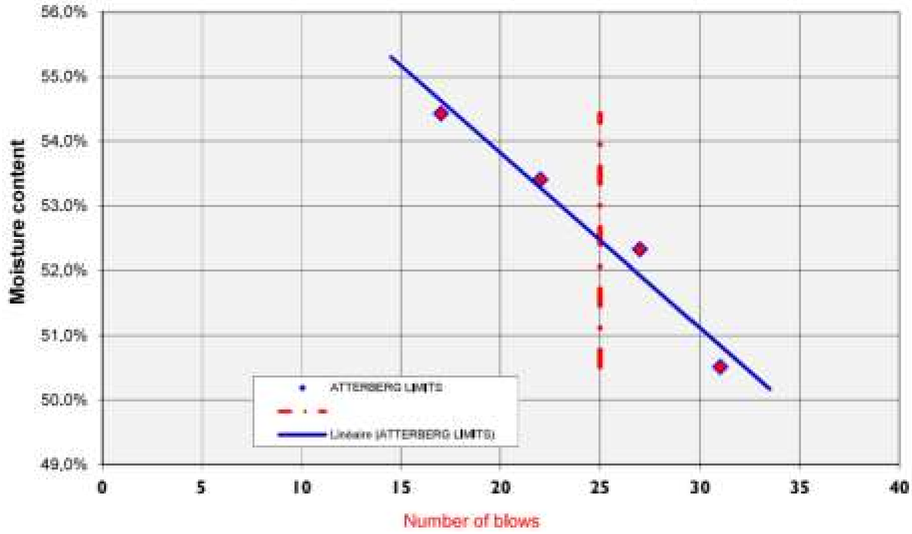


Figure 4.16. Liquid and plastic limits tests from SC 16

For the sample SC16, a compressibility test has been performed with the Equipment apparatus oedometer following the CV test - Taylor method with step from 1 to 2 kg/cm². The results are presented on Figure 4.17.

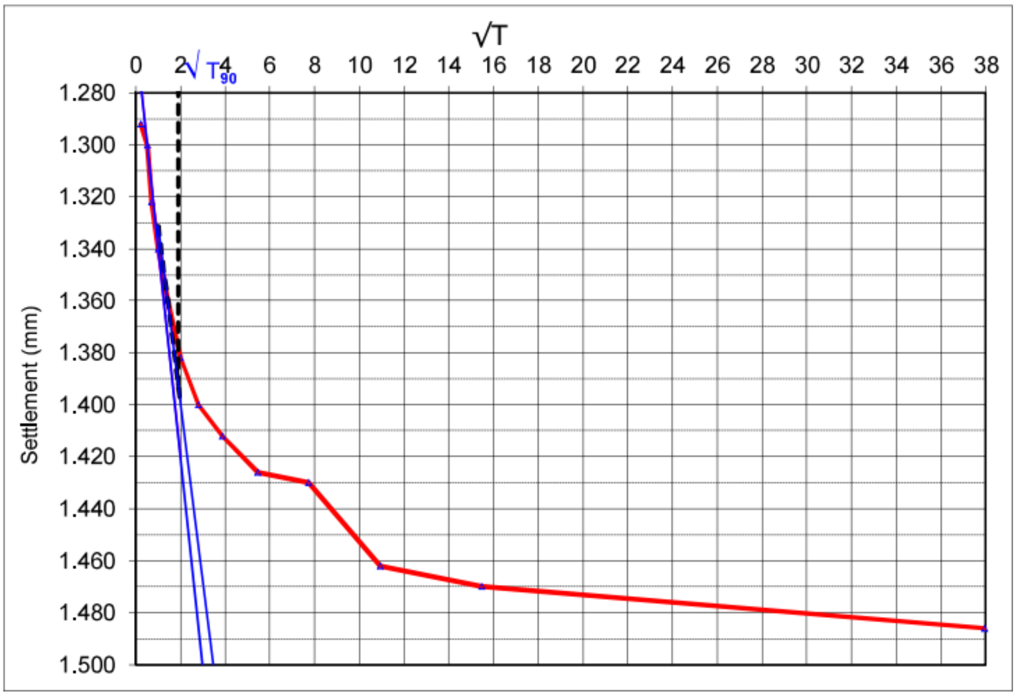


Figure 4.17. Compressibility test results from oedometer (Taylor method)

Following the French soil system of classification, the Figure 4.18 shows that the collected sample SC16 is obviously a silty clayey sandy soil.

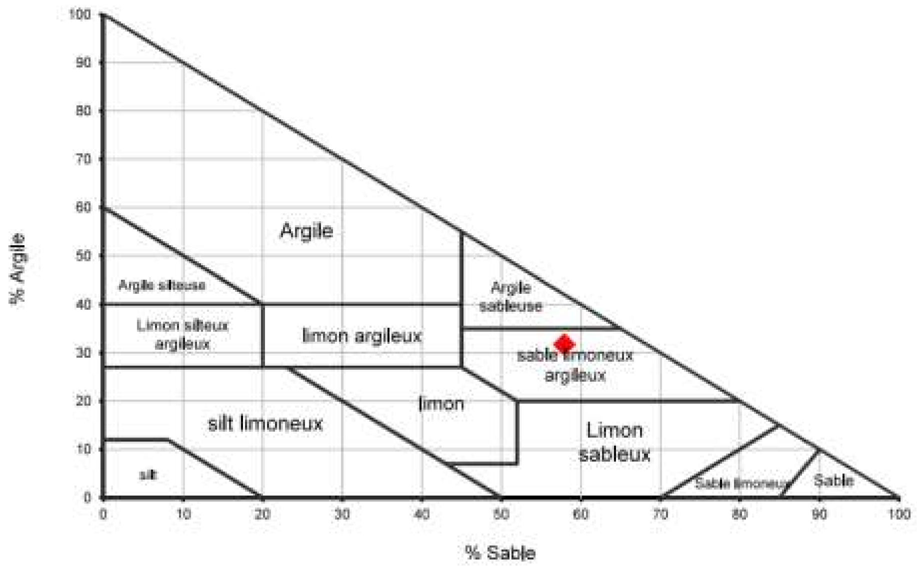


Figure 4.18. Soil classification from sample SC 16

Referring to the French standard NF P 94-071-1, the Casagrande shear unconsolidated undrained test has been performed. The obtained graph is shown on Figure 4.19.

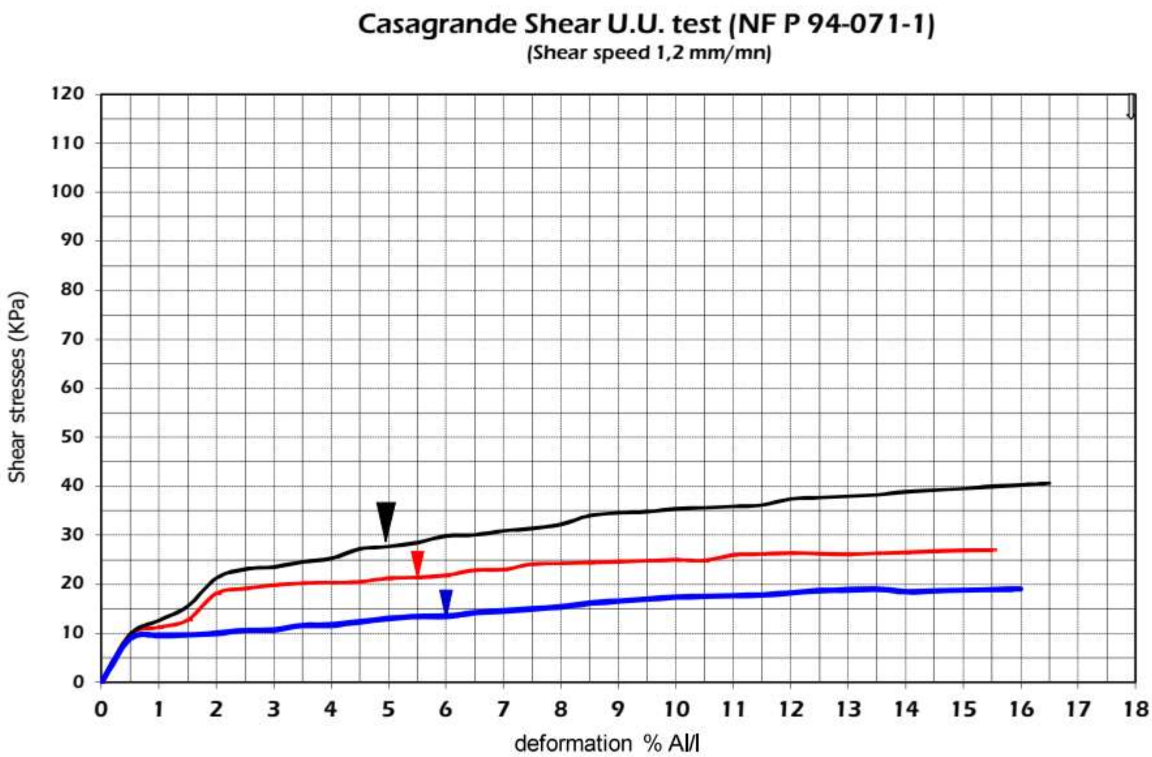


Figure 4.19. Casagrande shear UU test (NF P 94-071) test results

The direct shear test (DST) performed on sample SC16 gave an undrained shear strength of 10.1 kPa and an undrained friction angle of 9.56° (Figure 4.20).

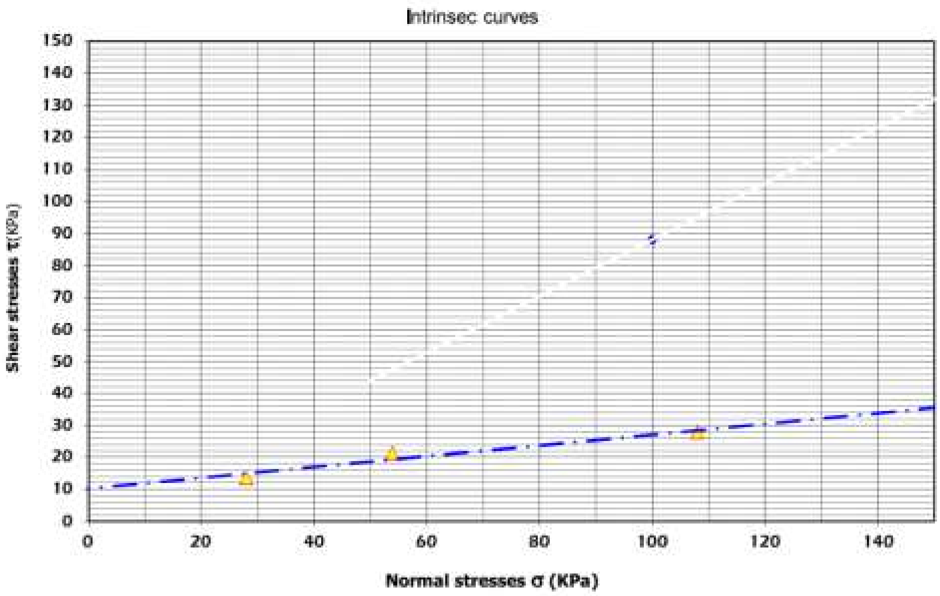


Figure 4.20. Direct shear test graph from samples SC 16 analysis

With an applied pressure of 100.0 kPa, a soil void ratio of 0.759 and 31.72% of passing under 80-µm (No. 200) Sieve, a permeability test on fine soils ASTM D 5856. (Falling Head) / AASHTO T-89 has been performed on the sample SC16. The results are presented on Table 4.3.

Table 4.3. Permeability test results (on fine soils ASTM D 5856)

Depth recovery (m)	Applied pressure (kPa)	Manometers tubes		Log(H0/H1)	s/A	t (s)	T (°C)	Permeability k (cm/s)
		Ho	H1					
2 – 2.85	100.0	1000	500	0.301029996	0.00862245	52	28.5	1.85E-04

The French standard NF P 94-050 test help finding the moisture content of the sample. The results are presented on Table 4.4.

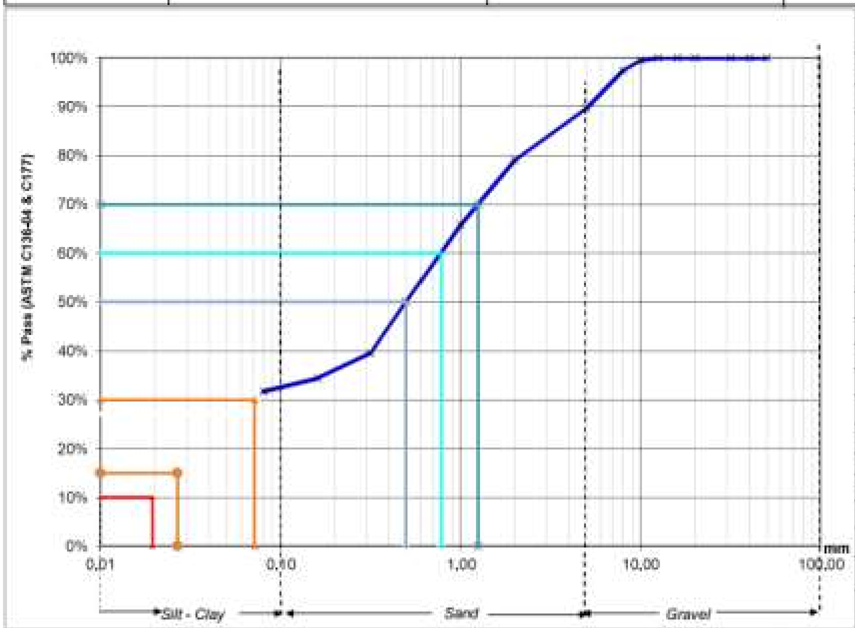
Table 4.4. Moisture content test (NF P 94 – 050) results

Tare no	Bulk weight	Dry weight	Tare weight	Real dry weight	Water weight	Dry density	Moisture content
A	2.25 T/m^3	1.91 T/m^3	59 g	1.32 T/m^3	34.5 g	1.517 T/m^3	26.1%

Table 4.5. Sample SC 16 laboratory tests results

SAMPLE IDENTIFICATION			
Depth	2.00 – 2.85 m	Sample ID	Undisturbed SC_16
γsat	19.14 kN/m^3	Sample texture	Sandy limono-blackish clay
γd	15.17 kN/m^3	AASHTO class	A-2-6(2) (Silty sand)
SIEVE & HYDROMETER ANALYSES			
% Passing < 20mm	100.0 %	CC	1.798
<10mm	99.4 %	Cu	0.443
<5 mm	89.6%	Cz.	1.167
<2 mm	79.0 %	Gravel	10.36 %
< 1 mm	65.8 %	Sand	57.92 %
< 0.315 mm	39.5 %	Clay	31.72 %
< 80 μm	31.7 %	Median	0.499 mm
ATTERBERG LIMITS			
ωc	26.1 %	LL	52.40
PI	24.46	γs	2.605 T/m^3
DIRECT SHEAR UU			
Su	10.1 kPa	φ	9.56 °
COMPRESSIBILITY TEST (ASTM D2435)			
Cc	0.193	σ'_p	31.436 kPa
e_0	0.861	Sr	49.62 %
Cs	0.06172	k_x	0.0001854118 cm/s

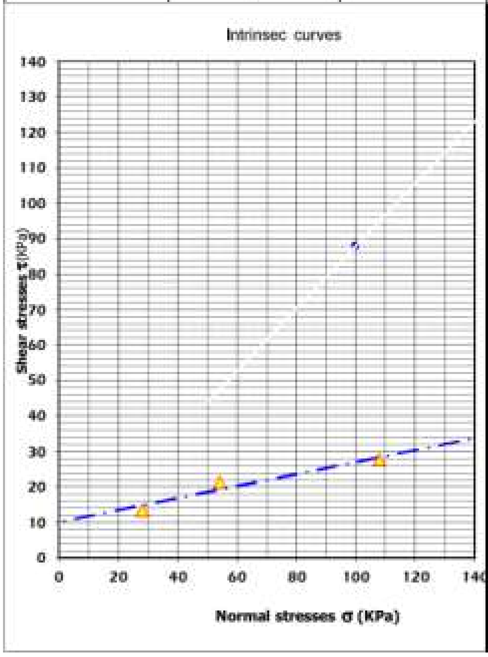
	Expertise de plateforme dans l'enceinte de la ZI Ndokoti		Argile sableuse limono-noirâtre (SC_16) [Altitude :-2m] Sédiment mal classé		depth recovery / TGS		2.00 - 2.85 m	
	Gravel (%)=	**10.36%**	Sand (%)=	**57.92%**	Clay (%)=		**31.72%**	
	Bulk density γ_h	1.914	Cu =	40.24	Cc		0.34	
	Dry density γ_d	1.517	Liquid Limit	52.4	Specific gravity		Classification	
29-sept.-18	Moisture content W_{nat}	26.1%	Plasticity Index	24.46	2.605 T/m3		A-2-6(2) [SM_Silty Sand]	



Sieves (mm)	% passing
0.08	31.7%
0.094	31.8%
0.1	32.6%
0.16	34.4%
0.315	39.5%
0.425	46.3%
1	65.8%
2	79.0%
5	89.6%
8	97.4%
10	99.4%
16	100.0%
20	100.0%
31.5	100.0%
40	100.0%
50	100%

Médiane : 0.499 mm
Moyenne : 1.201 mm
Coef. d'Asymétrie : 0.442
Coefficient d'Acuité : 1.798
S0 Indice de tri Trask : 5.612

Direct shear test	Undrained shear strength, Cu	10.1 kPa	Compression Index Cc	0.19311	Swelling Index Cs	0.0617
Final Moisture content =24.04%	Internal friction angle φu	9.56 °	Preconsolidation pressure σ'p, Kg/cm²	0.31436	Initial void ratio e0	0.861



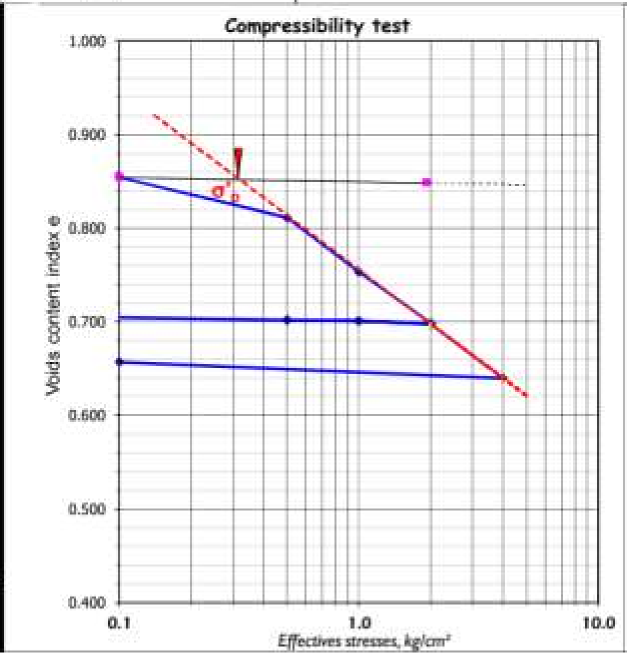


Figure 4.21. Sample SC 16 laboratory tests graphs

4.3.2. Geophysical data

The results of the geoelectric resistivity tomography are presented and interpreted based on the nineteen ERT pseudo 2D-profiles and the generated 3D models obtained from the company.

4.3.2.1. Zone A ERT results

At the level of the Zone A, five profiles were carried out, 3 longitudinal and 2 cross profiles.

i. ERT parameters

In this Zone A, five profiles: RES 1 – 2 – 3 – 4 – 5 were done as in order to well understand the soil stratigraphy. Electrodes are regularly spaced with an inter-electrode distance between 0.5m to 0.6m with a total of 96 electrodes with a constant pseudo-minimal depth of 1m as shown in Table 4.6.

Table 4.6. Recapitulative of ERT parameters – Zone A

	RES 1	**RES 2**	**RES 3**	**RES 4**	**RES 5**
Acquisition parameters					
Inter-electrodes distance	0.5m	0.6m	0.5m	0.5 m	0.5 m
Profile length	48m	57.6m	48m	48 m	48 m
Analysis parameters					
N° of block model	711				
Model levels	15				
Pseudo-maximal depth	8.72m	10.5m	8.72m	8.72 m	8.72 m

Concerning all the inversion parameters, all the blocks have equal widths and the refinement model (cells with widths equal to half the inter-electrode distance) are the same for all the profiles.

ii. 2D Profiles

On the 5 profiles, an alternation of conductive layers and resistant layers was noticed:

- Existence of resistant anomalies that can be interpreted as dry voids especially in the RES1 profile on Figure 4.22.
- Circular conductive anomalies that can be interpreted as cavities filled with water in the RES2, RES3 and RES5 profiles (Figures 4.23 & 4.24).
- At the level of profile RES4 we notice the existence of an abnormal structure that can be interpreted as a fracture.

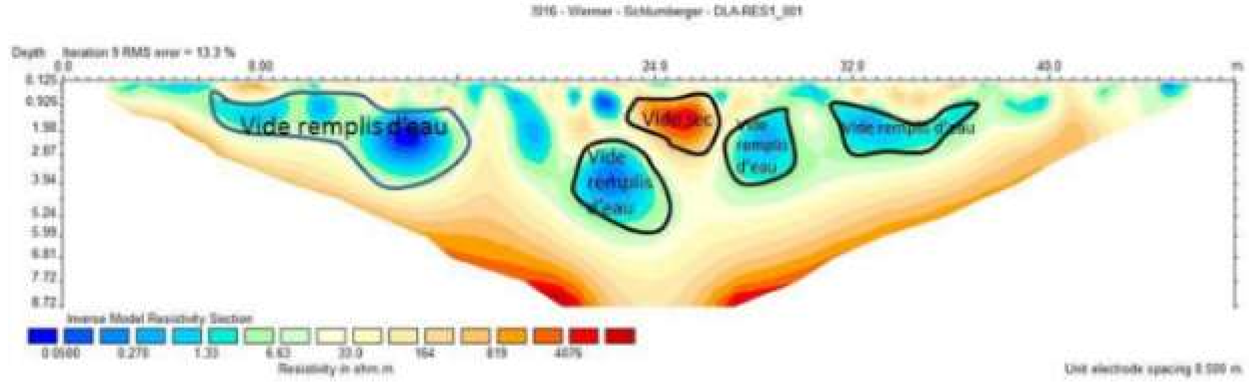


Figure 4.22. Zone A RES1 2D tomography

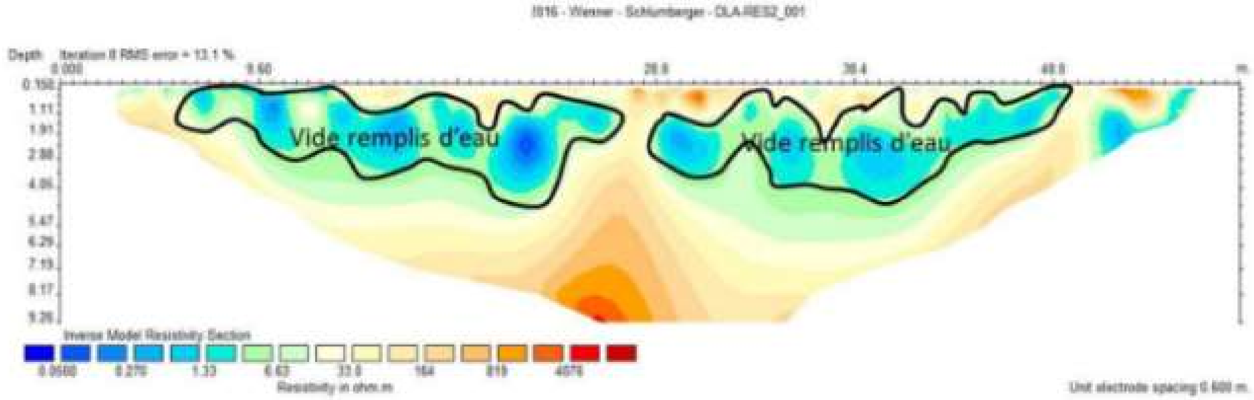


Figure 4.23. Zone A RES2 2D tomography

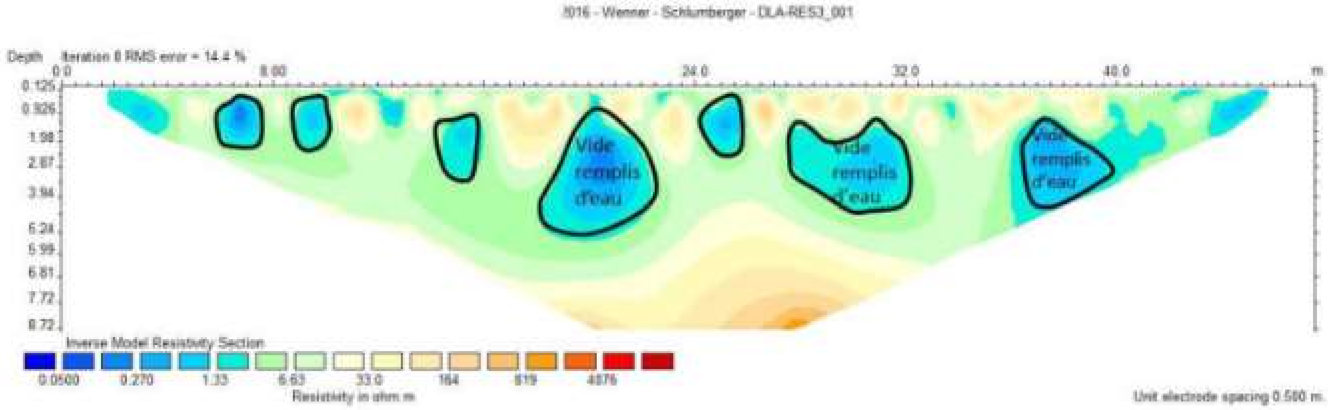


Figure 4.24. Zone A RES3 2D tomography

iii. 3D models

At the level of Zone A, it was noted from the 3D modelling that the voids are generally located at the level of the superficial part not exceeding a depth of 2m but in some cases can exceed 3 m in depth. As shown on Figure 4.25, those underground voids occupy a volume of about 1970 m^3.

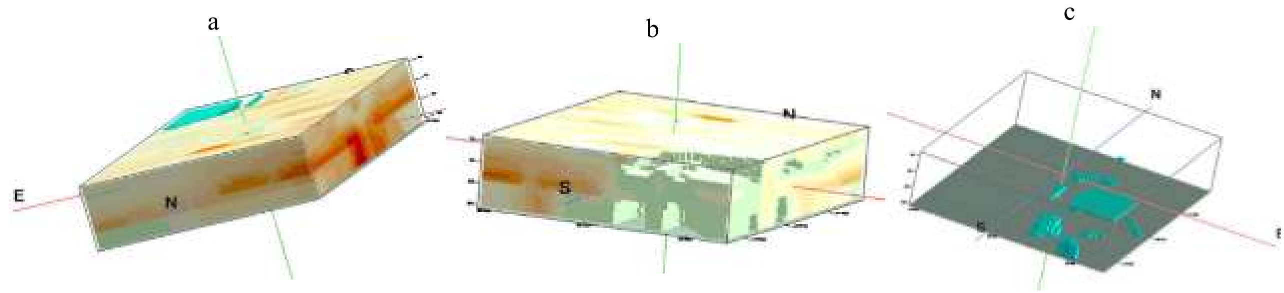


Figure 4.25. Zone A 3D model a)complete resistivity b)showing voids c)total voids volume

4.3.2.2. Zone B ERT results

In the Zone B, seven profiles were made, of which 3 were longitudinal, 3 were transversal and one was oblique.

i. ERT parameters

In this Zone B, seven profiles: RES 11 – 12 – 13 – 14 – 15 – 23 – 24 were done in order to well understand the soil stratigraphy. Electrodes are regularly spaced with an inter-electrode distance between 0.4 m to 1 m with a total of 96 electrodes and a pseudo-maximal depth of 17.4 m as shown in Table 4.7.

Table 4.7. Recapitulative of ERT parameters – Zone B

	RES 11	**RES 12**	**RES 13**	**RES 14**	**RES 15**	**RES 23**	**RES 24**
Acquisition parameters							
Inter-electrodes distance	0.4m	0.40m	0.6m	0.5 m	1 m	1 m	1 m
Profile length	19.2m	19.2m	57.6m	48 m	96 m	96 m	96 m
Number of electrodes	48	48	96	96	96	96	96
Analysis parameters							
N° of block model	711						
Model levels	15						
Pseudo-minimal depth	1m	1m	1m	1 m	1 m	1 m	1 m
Pseudo-maximal depth	3.65m	3.65m	5.47m	8.72 m	17.4 m		
Inversion parameters	All blocks have equal widths		Refinement model (cells with widths equal to half the inter-electrode distance)				

ii. 2D profiles

From these profiles we noticed the existence of certain structures:

- In all the profiles we notice an alternation of conductive layers and resistant layers in RES10 profile.
- It is noted that for the longitudinal profiles, the important voids are mostly located on the straight parts.
- Existence of resistant anomalies that can be interpreted as dry voids especially in the RES 7 profile (Figure 4.26).
- The existence of conductive anomalies, sometimes circular and sometimes lenticular, which can be interpreted as voids filled with water in the profiles RES11, 12, 13, 14, 23 and RES24 (Figures 4.26, 4.27 & 4.28).

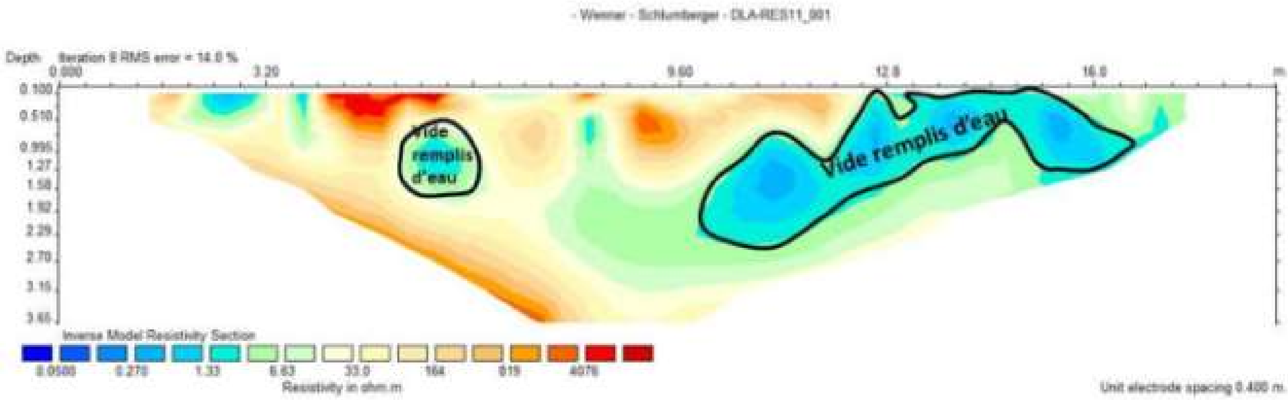


Figure 4.26. Zone B RES6 tomography results

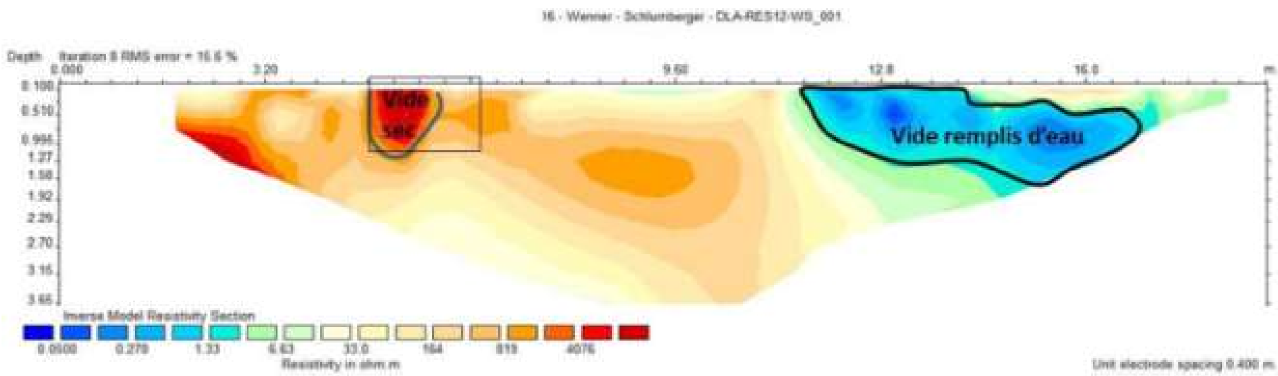


Figure 4.27. Zone B RES7 tomography results

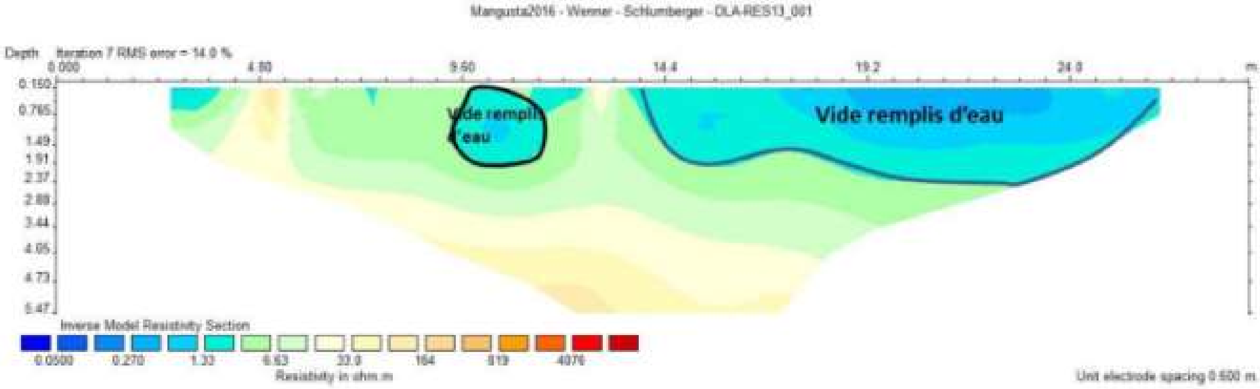


Figure 4.28. Zone B RES8 tomography results

iii. 3D models

In the Zone B and the outdoor area, it was noted from the 3D modelling that the voids are generally located at the level of the superficial part not exceeding a depth of depth of 2m. As shown on Figure 4.29, those underground voids are occupying a volume of about 3100 m^3.

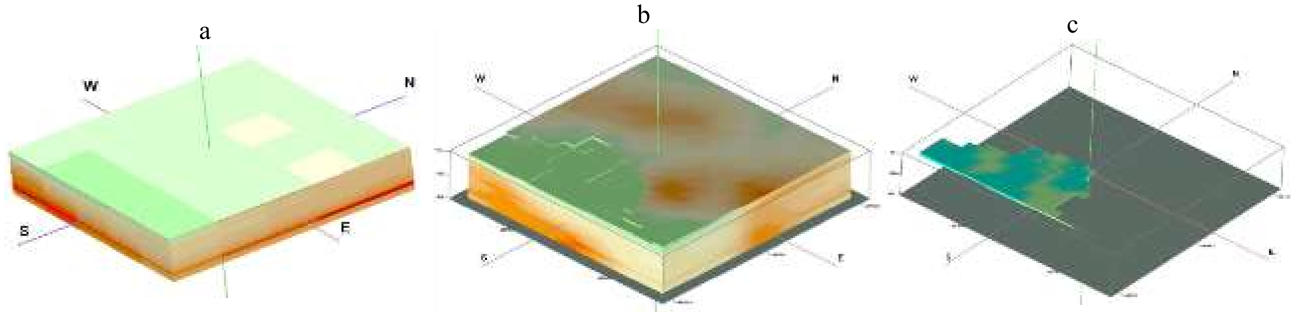


Figure 4.29. Zone B 3D model a)complete resistivity b)showing voids c)total voids volume

4.3.2.3. Zone C & D ERT results

At the level of the Zone C, 4 profiles were made and in the Zone D, 3 profiles were made.

i. ERT parameters – zone C

In this Zone C, four profiles: RES 16 – 17 – 18 – 19 were done in order to well understand the soil stratigraphy. Electrodes are regularly spaced with an inter-electrode distance between 0.5 m to 0.7 m with a total of 96 electrodes and a pseudo-maximal depth of 12.2 m as shown in Table 4.8.

Table 4.8. Recapitulative of ERT parameters - Zone C

	RES 16	**RES 17**	**RES 18**	**RES 19**
Acquisition parameters				
Inter-electrodes distance	0.5 m	0.5 m	0.5m	0.7m
Profile length	48m	48m	48m	67.2m
Number of electrodes	96	96	96	96
Analysis parameters				
N° of block model	711			
Model levels	15			
Pseudo-minimal depth	1 m	1 m	1 m	1 m
Pseudo-maximal depth	8.72 m	7.72m	8.72m	12.2m
Inversion parameters	All blocks have equal widths	Refinement model (cells with widths equal to half the inter-electrode distance)		

ii. 2D profiles – zone C

The profiles made were 2 longitudinal and 2 transversal. In the 4 profiles, was observed:

- An alternation of conductive and resistant layers on Figure 4.28.
- The existence of conductive anomalies more developed and close to the surface, which can be interpreted as voids filled with water in profiles RES16, 17, 18 and RES19 (Figures 4.30, 4.31, 4.32 & 4.33).

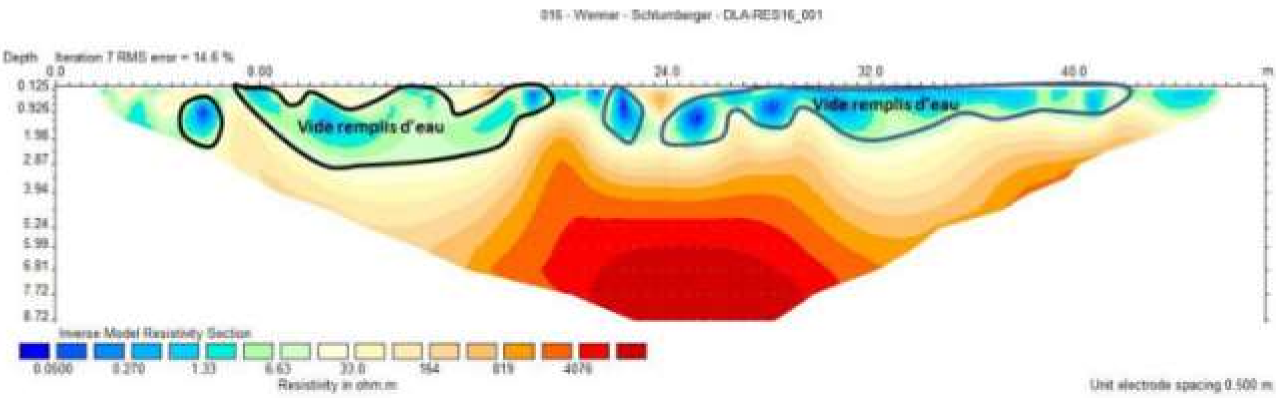


Figure 4.30. Zone C RES16 tomography results

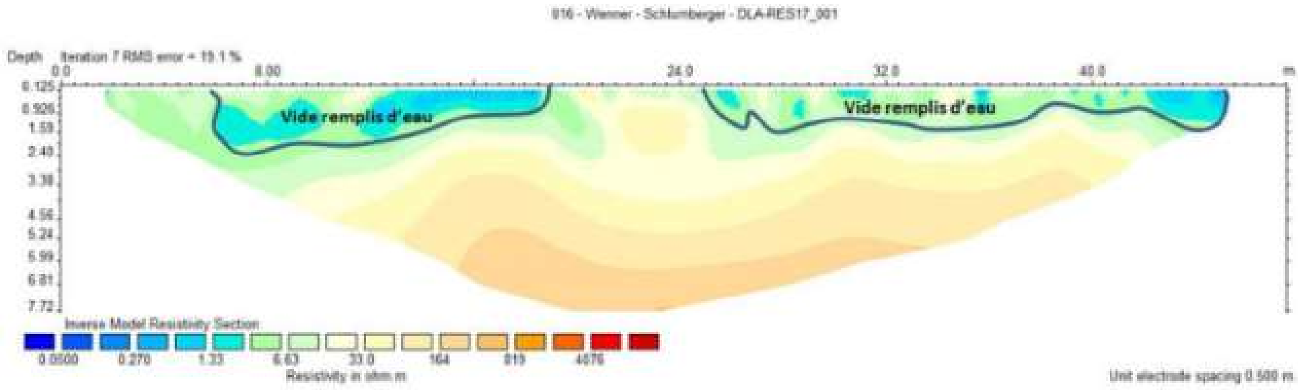


Figure 4.31. Zone C RES17 tomography results

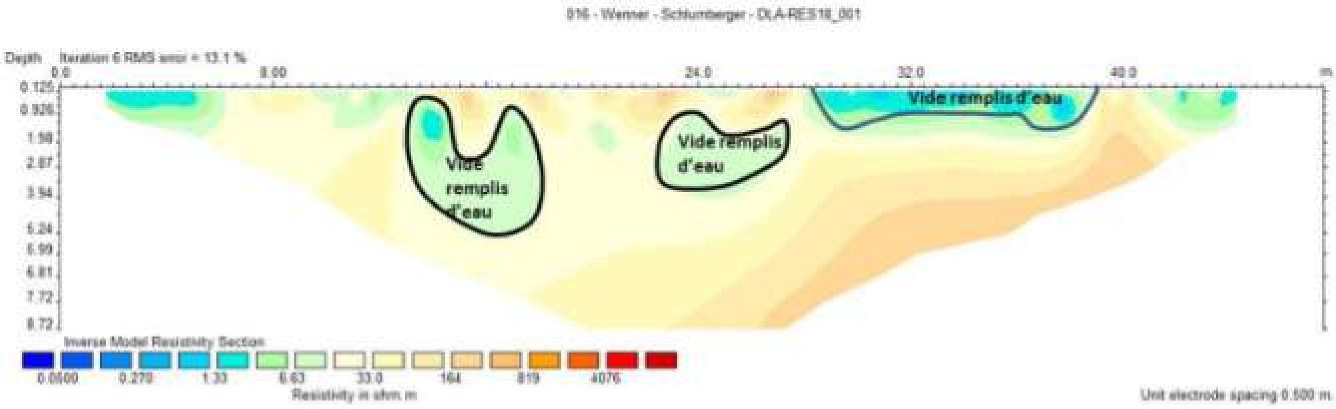


Figure 4.32. Zone C RES18 tomography results

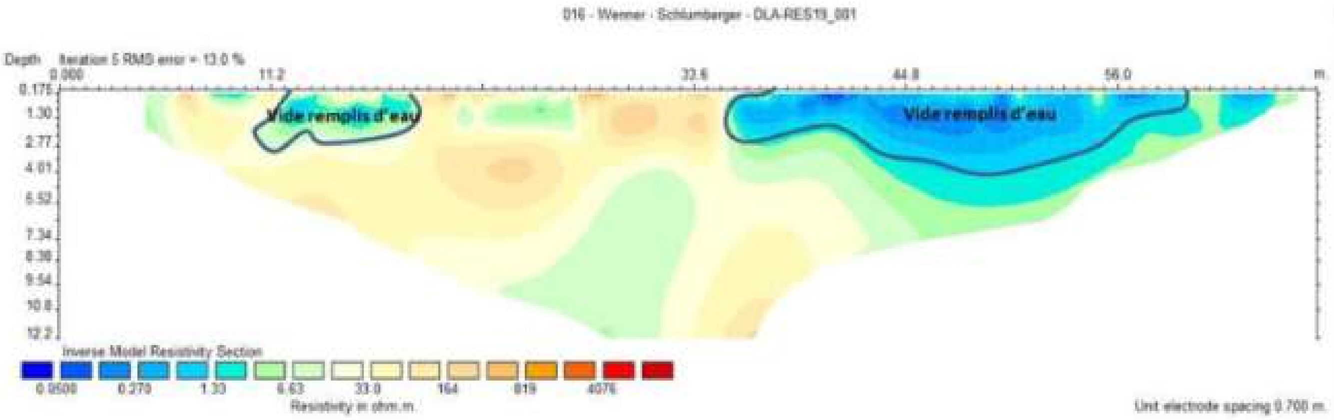


Figure 4.33. Zone C RES19 tomography results

iii. ERT parameters – zone D

In this zone 5 called Zone D, three profiles: RES 20 – 21 – 22 were done as shown on the Figure 3.14 in order to well understand the soil stratigraphy.

Electrodes are regularly spaced with an inter-electrode distance of 0.5 m with a total of 96 electrodes. Concerning all the inversion parameters, all the blocks have equal widths and the refinement model (cells with widths equal to half the inter-electrode distance) are the same for all the profiles. The constant pseudo-maximal depth is 8.72 m with a constant pseudo-minimal depth of 1 m. The acquisition and analysis parameters of the profiles are presented in Table 4.9.

Table 4.9. Recapitulative of ERT parameters – Zone D

	RES 20	**RES 21**	**RES 22**
Acquisition parameters			
Inter-electrodes distance	0.5 m	0.5 m	0.5m
Profile length	48m	48m	48m
Number of electrodes	96	96	96
Analysis parameters			
N° of block model	711		
Model levels	15		
Pseudo-minimal depth	1 m	1 m	1 m
Pseudo-maximal depth	8.72 m	8.72m	8.72m

iv. 2D profiles – zone D

Two longitudinal and one transversal profiles were made. From these profiles, the existence of certain structures was noted:

- In the 3 profiles, a super-imposition of a conductive surface layer and a resistant layer underneath it
- The existence of lamellar conductive anomalies that are more extensive throughout the room, which can be interpreted as voids filled with water in profiles RES20, RES21 and RES22 (Figures 4.34, 4.35 & 4.36).

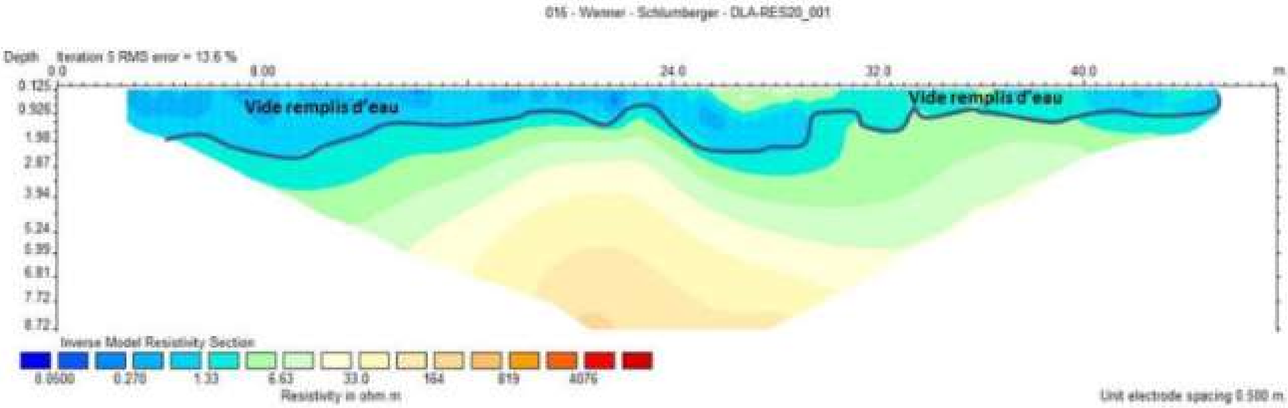


Figure 4.34. Zone D RES20 tomography results

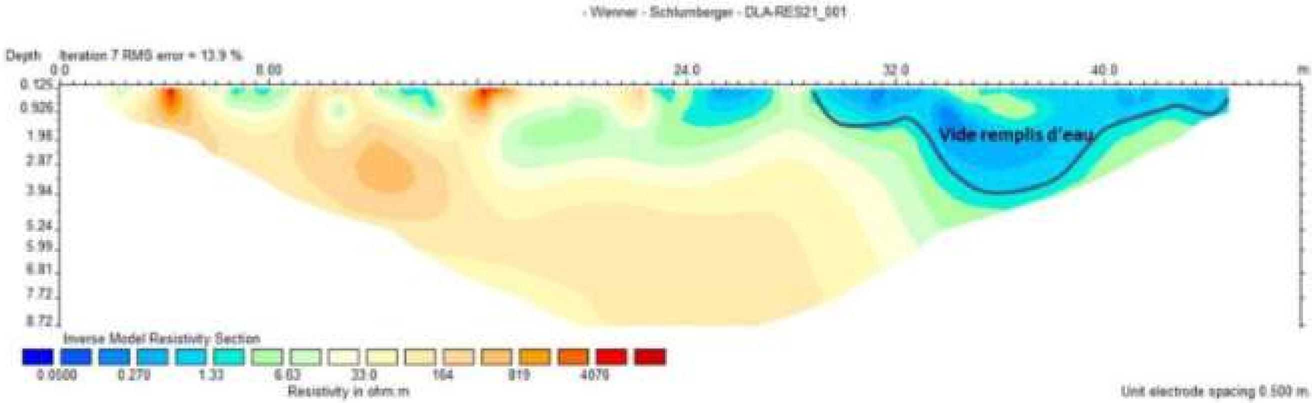


Figure 4.35. Zone D RES21 tomography results

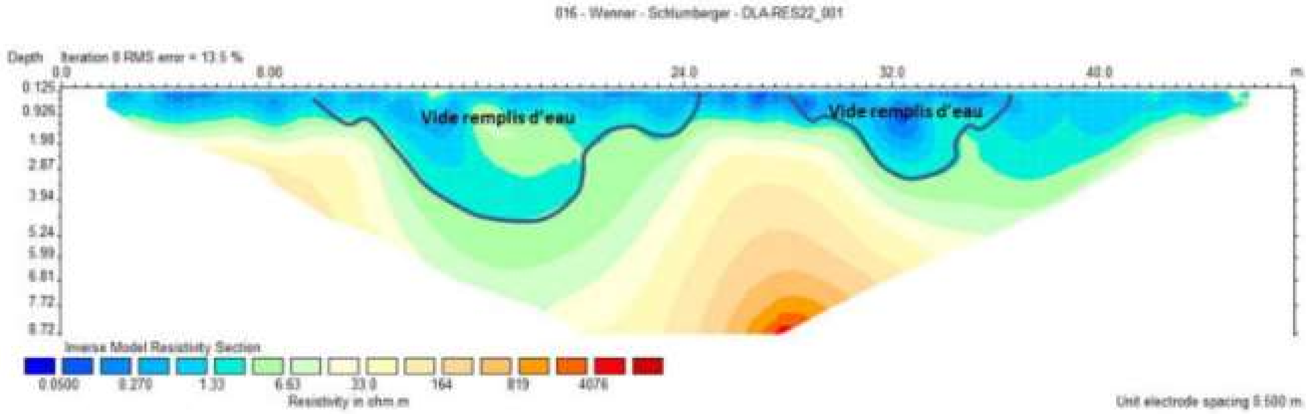


Figure 4.36. Zone D RES22 tomography results

v. 3D models – zones C-D

In the engine room and the outdoor area, it was noted from the 3D modelling that the voids are generally located at the level of the superficial part not exceeding a depth of 2m. As shown on Figure 4.37, those underground voids are occupying a volume of about 4560 m^3.

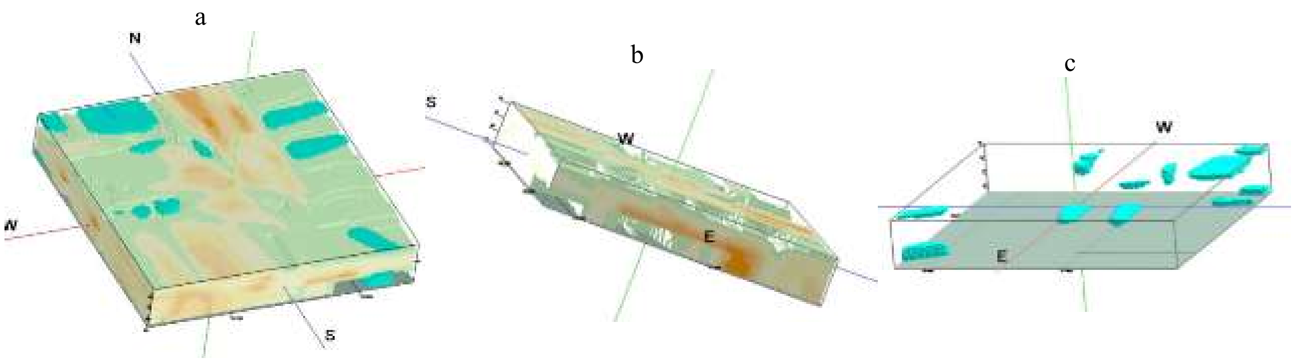


Figure 4.37. Zone C-D 3D model a)complete resistivity b)showing voids c)total voids volume

4.3.3. Grout mix composition

Since the pre-defined objectives were known before starting the compaction grouting, the grout composition was then carefully formulated in order to fill the underground voids and compact the surrounding areas which improve the bearing characteristics of the soil.

Mobility of the grout is dependent on the grout mix and the rate of injection. The grout must behave essentially as a growing solid when it enters the soil. It must have sufficient internal friction by adding to the mixture sandy soils so that it behaves as a continuous mass.

With the most appropriate aggregates range found, the right grout plasticity was obtained with a water/cement ratio of 0.5. The mixt grout used on site for 1 m^3 was composed of: water, sand, cement CEM I 42.5 N (CPA 45) and additives with proportions indicated in Table 4.10.

Table 4.10. Grout components proportions for $1m^3$ (from project)

Materials	**Cement**	**Sand**	**Water**	**Additive**
Ratios	800 kg/m^3	800 kg/m^3	400 m^3	5 kg/m^3

4.4. Compaction grouting execution

After finishing the design phase of compaction grouting, the next step is to apply the method on field. The whole process of grouting execution which is composed of two principal phases: the injection phase and the monitoring phase will be presented.

4.4.1. Injection phase

Injection phase is the first before anything else step. It concerns the preparation of injection equipment, the grouting process and the observations after the grouting.

4.4.1.1. Injection equipment

The equipment used for this project was easily movable from one point to another one. From the grout mixer on site, to the grout holes passing through the pump, many hoses, pipes and gauges were used. The whole equipment was connected together through hoses. In order to be efficient, a distant lesser than 100 m has been imposed between the pump (Figure 4.38) and the injection holes to reduce pressure losses.

Figure 4.38. Grout pumping machine (from data project)

Some additional components such as the drilling equipment and casing extraction apparatus were also used on site. Indeed, before installing the grout casing on Figure 4.39, a core drilling has been done till the desired depth. A casing with a sufficient size permitted a free flow of the grout with minimal head losses and was fitted snugly into the hole to prevent grout from flowing up the annulus around it.

Figure 4.39. PVC – grout casing for injection hole (from data project)

Besides, it is important for all the piping and fittings to be tight and all changes in diameter to be gradual because although no slump and viscosity tests have been done to evaluate the grout consistency, leakage in the fittings results in water loss from the grout that can lead to blockages.

4.4.1.2. During the injection

As illustrated on Figure 4.40, the whole injection process with the equipment consists of three principal sequences with the core drilling of grout holes, the casing installation and then the grouting. The injection has been done following the bottom – top stage in areas where roads or buildings were located.

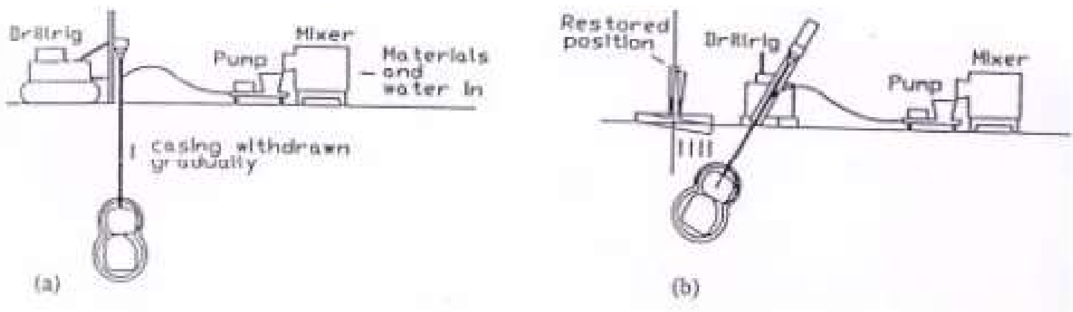


Figure 4.40. Remedial works by compaction grouting of a) road and b) below a foundation

The grout material is pumped into an open segment of borehole termed a stage. A casing is installed tightly in the borehole, with little or no annulus, so that grout is forced to expand radially and is restrained from traveling upward along the pipe. Figure 4.41 shows the hole bored before grouting.

Figure 4.41. Slab coring before injecting (from project data)

The injection is done by means of special pipes which could be single or double headers (Figure 4.42) with valves at every 0.5 cm depending on the admissions encountered in the first phase of the drilling treatment to stop the injection pressure losses through the holes.

Figure 4.42. Single headers while grouting a hole (from project data)

These headers have to be adapted to the passage of grout and must support pressures between 5-7 bars. The grout is injected with pressures ranging from 3 to 6 bars. The rate of injection is important to allow dissipation of pore pressures during and after the grouting phase.

During the execution of the injections, there were two refusal or stop criteria:

- Stop by volume when the planned volume to be injected is reached. Those volume were computed using the 3D ERT models of each underground cavity to fill.
- Stop by pressure: when the planned limit pressure is reached. The limit pressure has been defined based on the soil in-situ conditions, the operator experience and the type of superstructures.

4.4.1.3. After the injection

Two principal refusal criteria have been used: the volume and the injection pressure but in some cases, fissures and cracks are warning signals to stop grouting. The grout injection ended immediately when the apparition of fissures on the concrete slab started. Indeed, the slab shown on Figure 4.43 was not enough resistant during the whole grouting process to all the injection pressures.

Figure 4.43. Head of grout column in the slab (from project data)

The density and compressive strength verification of the grouted columns after 3 days, 7 days, 14 days and 28 days has not been done, it is therefore impossible to know the compressive resistance of the grouted columns. In addition, the getting out of grout through appeared fissures on the slab is perceptible on Figure 4.44.

Figure 4.44. Grout getting out the concrete slab (from project data)

4.4.2. Field monitoring phase

The monitoring phase is following the injection phase. It concerns the surrounding of the zone where the injection holes are located and the post – grouting monitoring of foundations.

4.4.2.1. Injection parameters

The parameters of the injections will be defined according to the specifications of the execution project by controlling the grout composition as well as the injection pressure and the volume injected in order to respect the specifications.

During the grouting, the injection parameters monitored and recorded were the injected volume in each stage and the injection pressure by means of pressure gauges (Figure 4.45).

Figure 4.45. Pressure gauges fixed on fittings (from project data)

4.4.2.2. Post-grouting measurements

The injection ended, the ground surface heaved in some points with various amplitudes. These amplitudes have not been assessed correctly. Indeed, although carpenter's levels, rulers and physical appreciation were used for heave as shown on the Figure 4.46; it was not possible to know how much the ground was heaving. Those available measurement tools were archaic, no precise and automatic.

Figure 4.46. Concrete slab fissures at a structure joint (from project data)

4.5. Applied compaction grouting evaluation

Evaluating the applied compaction grouting in the case study of this project will help well understand the improvement method. Therefore, the intrinsic limitations found in the mechanical principles and the defaults in the grouting execution will be presented.

4.5.1. Compaction grouting limits

Compaction grouting limitations can only be determined by carefully understand its mechanical principles coupled with its execution procedures.

4.5.1.1. Mechanical principles

Referring to the mechanical equations of grout propagation during compaction grouting, the grout must be enough consistent to be injected at pressures around 10MPa. Those pressures are most of times too higher to be supported by a ground surface with overburden pressure. Therefore, compaction grouting has a relative ineffectiveness in stabilizing near-surface soils.

As the deformations decrease with the distance from the injection point, there is a distance at which the deformations are no longer plastic, but elastic. The ground at the periphery of the mortar ball breaks up up to a certain distance from the injection point, where it is the seat of important shear forces. If the process is not well controlled and monitored to be stopped at the right moment, it could induce hydro-fracture inside the soils. For the fracturing risk, research today is not sufficiently advanced to directly predict the behaviour of a grout in interaction with soil.

4.5.1.2. Guidelines and standards

In practice, to find the suitable grout with the appropriate range of particle size distribution, the grout mix formulation is still based on experience of contractors involved in compaction grouting. Many grouters describe the grout by squeezing it in their hands when talking about his consistency. This criterion is far from a scientifically repeatable test, but to date is the only way of actually evaluating grout rheology in the field.

With the several publications related to compaction grouting published describing the procedure and basic concepts of compaction grouting, the practice of compaction grouting has expanded greatly through the efforts of those working in this area. Despite, the 30-plus years since these original papers, design standards are not still available for engineers who can only have guide of good practice.

4.5.2. Field observed limits

The absence of some verification tests and monitoring system has been shown as limits especially for compaction grouting on field since it is causing ground heave through soil compaction.

4.5.2.1. Grout consistency tests

The continuous mass behaviour of the grout is useful to avoid hydraulic fracturing of the soil. The consistency and plasticity of grout which have not been verified by the slump test (with Abrams cone) and the viscosity test (with Marsh cone) could be at the origin of this grout rising through the concrete slab fissures observed on Figure 4.47.

Figure 4.47. Grout rising to the surface through cracks (from project data)

4.5.2.2. Displacement monitoring

During the injection of the grout, the underground pressure increase until it reach a level where cracks appear. When considerable, this ground surface heave also called uplift can cause a lot of structural damages on the surrounding buildings. In fact, other types of deformations could be generated on the structures such as: subsidence, horizontal extension, diagonal cracking and angular distortion. Since, the deformations were not measured adequately, it is really difficult to how much the ground heaves as observed at the structural existing joint shown on Figure 4.48.

Figure 4.48. Slab breaking post-grouting due to ground heave (from project data)

4.5.2.3. Density verification

After ending the grout injection, most of technical guidelines always recommend to make a ground density verification. In fact, this post-grouting density verification are very helpful to understand how and where the grout has been distributed inside the ground.

Since, the aim of compaction grouting was to densify the ground after grouting, it is therefore impossible to verify the degree of densification and the grouting effectiveness because this density verification has not been made.

4.6. Post-grouting quality program

This post – grouting quality program aims to assess the quality assurance by verifying the grouting effectiveness, and propose a quality control to improve the grouting performance degree. This will therefore determine the compaction grouting efficiency as a soil improvement method.

4.6.1. Quality control

The performance degree of the compaction grouting is estimated on a long period of time through the monitoring of parameters such as the displacements and the ground water conditions.

4.6.1.1. Grout rheology

Grout formulated with various percentages of cement, sand, water and additives, before being injected, depends of the pre-defined objectives and the type of ground (sand, clay, silt or gravel).

The grout consistency and its manoeuvrability should be verified with the slump test referring to ASTM or EN standard. Furthermore, a good practice consists of also evaluate the grout viscosity by timing the efflux of a standard grout quantity from a normalised cone named Marsh cone.

4.6.1.2. Long term monitoring

Since surface heave is a frequent refusal criterion, it should be measured and limited to between 1.2 and 2.5 mm per stage. The compaction grouting is a high pressure injection technique which could create the ground surface heave when not well executed. The vertical displacements of the ground surface and the cracks or fissures on the surrounding structures can induce many structural damages which could be avoid by monitoring techniques such as:

i. Tiltmeters

They measure the angular movement or rotation of surfaces and planes. Using accelerometers or vibrating wire technology to be more precise and automatic, electronic tiltmeters placed on walls will measure their angular inclinations during the grout construction.

ii. Fissuremeters

They measure the amplitude of fissures in a structure or a crew in the ground. Installing them on a wall, a beam, a slab will be helpful to have the amplitude of cracks. Depending on their sizes, fissuremeters (Figure 4.49) can record very small dimensions less than 1mm.

Figure 4.49. Different sizes of fissuremeters (sisgeo.com, 2021)

iii. Clinometers

They measure the slope of a plane or a pipe with the lecture which may be manual or automatic. They can be installed on the ground to evaluate its settlement or uplift as shown on Figure 4.50.

Figure 4.50. Sketch of a clinometer measuring railway settlements

iv. Laser levels

They can be useful to indicate a change of position with the prisms attached to structures and set to the level of rotating beam.

4.6.1.3. New compaction grouting method

Takenouchi & Sassa (2020) reported that the improved CG is a new method of injecting the grout while moving the injection tube up and down (Figure 4.51). Differently from the conventional CG stage up-down process, the new method CG is a cyclic up-down process. While increasing the up-down length, the number of up-down cycles and the pulling up speed, the upheaval control become more efficient.

The experimental results of the model showed that the proposed up and down method can reduce the ground upheaval quantity by more than 80%. The results of the full-scale experiment demonstrate that the new CG can reduce the ground upheaval quantity by more than 90%. Tested in laboratory and verified on a full-scale field, the cyclic injection of grout improve the CG quality control.

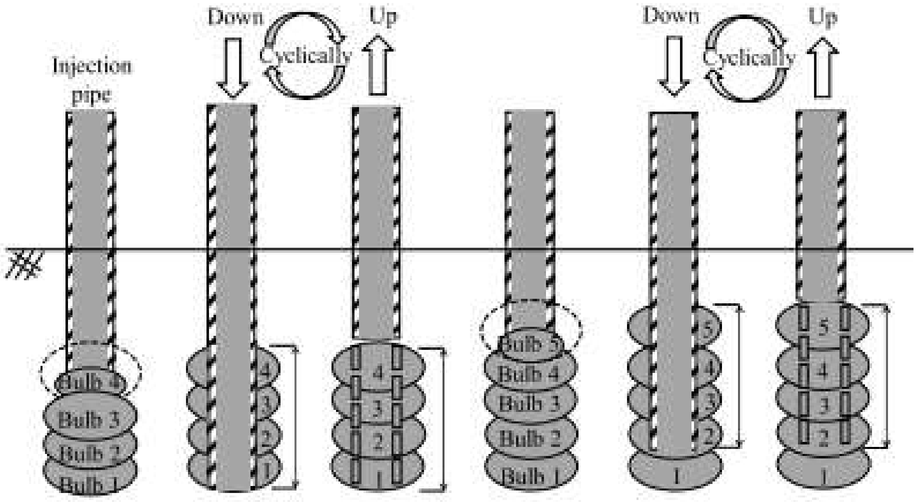


Figure 4.51. The new compaction grouting method (Takenouchi & Sassa, 2020)

Hence, the new CG can be applied to facilities where the allowable displacement amount is severe and conventional CG may not apply, thereby facilitating a sustainable infrastructure development. The upheaval control became more efficient with increasing the up and down length, the number of up and down cycles and the pulling up speed.

4.6.1.4. Ground water conditions

The water pore pressure is monitored to know the pressure in some points in order to define the maximum piezometric head, the position of the piezometric line or, in alternate, and the position of the water table. The measures obtained from electrical piezometer (Figure 4.52) must be in relation with rain and with the presence of permanent or temporary spring on the ground in order to understand how the consolidation process is going on after the injection.

Figure 4.52. Electrical piezometer probe head (soilinstruments.com, 2021)

4.6.2. Quality assurance

The compaction grouting effectiveness is evaluated with some verifications to assess the degree of densification, the strength and the bearing resistance of grouted columns relatively to loads.

4.6.2.1. Density verification

The degree of densification produced by compaction grouting could be verified by various methods which are grouped in two categories: direct methods and indirect methods.

i. Direct methods

The direct methods help understand the density variations in the underground with their results interpretation only without requiring a borehole or driving the soil. They consist principally of geophysical survey techniques such as the common employed:

- Electrical Resistivity Test

Since it has been done in the pre – grouting construction phase, a post – grouting tomography will be useful to verify if all the detected cavities have been correctly filled with grouts. The superposition of the pre –grouting ERT results to the post-grouting ERT results will help to understand the grout propagation, the most or less densified zones and probably the underground water flow.

- Ground Penetrating Radar

When the electromagnetic waves transmitted to the underground will hit a medium interface or cavity, they will reflect back to the ground. Those waves will be received by an antenna and the spatial locations can be identified precisely.

- Electro-Magnetic Induction

The instrument is designed to measure the much smaller signals generated by the conductivity properties of soils. This helps then to understand the underground density variation through the conductivity variations.

ii. Indirect methods

The second methods are considered as indirect because they could need to drill a borehole or to push the soils when the probe is entering. These methods are geotechnical investigation tests are:

- DPH

Linked to the SPT, this test helps to understand through the low number of blows counts for a layer if the soils are too loose or a cavity is present.

- CPT – CPTU – SCPT

It gives the excess pore water pressure, the friction and the tip resistances. In fact, if the probe cone detects low skin with high tip resistances, the soil is probably a loose to medium dense sand

(Figure 4.54). Besides, if the probe cone detects high skin with low tip resistances, the soils could be stiff to very stiff clay. Therefore, if both the skin and tip resistances are low, the probe is probably crossing an underground void.

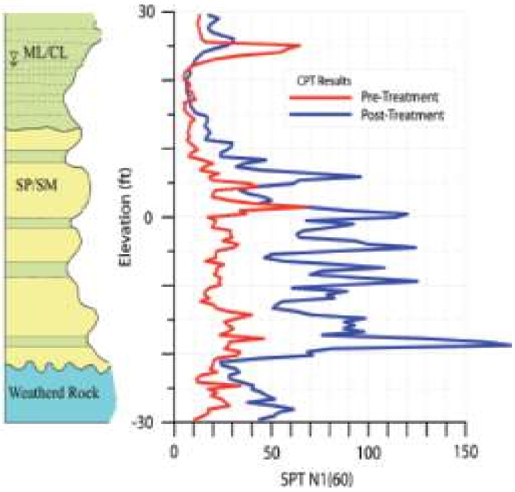


Figure 4.53. Comparison of pre- and post-treatment N1 (60) values from a liquefaction mitigation project in Laguna Beach, California (Geraci, 2007)

Although all those methods both direct and indirect have different results with various deduced analyses, the most important in a density verification is to superpose the results obtained from the same test in the pre-grouting and post-grouting construction phases.

4.6.2.2. Core drilling

In geotechnical engineering, verification on sites most of times included the laboratory tests where a lot of conditions can be controlled. So the best manner to be sure of the bearing resistance designed is obtained on field is to make a core drilling of the grouted columns.

This core drilling should be done at least three times after the grout hardening which means 7 days, 14 days and 28 days after grouting in order to verify if the compressive strength is obtained.

4.6.3. Compaction Grouting historic cases

Although rarely applied in Cameroon, compaction grouting has been used widely in many parts of the world and especially in the USA for reducing the void ratio of loose soils, relevelling of settled buildings, underpinning of footings and slabs, controlling settlements, and for mitigation of liquefaction potential of the soils in the last 10 to 15 years.

The use of compaction grouting which is going increasingly can be comforted as proofs by some well-known historic cases such as Adkins water treatment plant in South Carolina, the historical military site of Pearl Harbour in Hawaii, the Laguna Beach in California and the Randstadrail between the cities of Rotterdam.

Conclusion

The aim of this chapter was to conduct a critical reliability analysis of the compaction grouting in the Douala – Bassa industrial zone. To attain this objective, the first step was to present the geotechnical and geophysical data collected and the induced design method of the compaction grouting. In fact, starting from the grout mix rheology, the grouting execution and the field monitoring, those phases have been presented in order to analyse the compaction grouting limits observed on site. The obtained results help understand how to make a field post – grouting assessment which goals are: reduce the ground surface upheaval, evaluate the vertical displacement amplitudes, verify the compacted soil density, evaluate the grout compressive strength and control the excess pore-water pressures conditions during the consolidation process.

CONCLUSION

The purpose of this research work was to study and critically analyse the compaction grouting by evaluating its effectiveness and performance as an efficient soil improvement method in the Douala – Bassa industrial zone vicinity. Indeed this studied area had been undergoing significant subsidence due to underground cavities formed beneath the existing structures.

This dissertation has been performed by initially making an exhaustive literature review in two chapters. Firstly on the soil formation processes, all the possible sinkholes collapses schemes and the underground cavities detection. Then, a brief presentation of structural deformations, monitoring techniques, compaction grouting theoretical knowledge and associated installation procedures have been discussed. The methodology consisted of the structural damage survey, the geotechnical and geophysical investigation on site. Followed by the design and execution of the compaction grouting, it was achieved with the structural deformations monitoring techniques. This helps evaluating the reliability and efficiency of all the monitored injection parameters: injection pressure, injected volume, grout quantities, ground surface upheaval and influence on structures.

This paper which described the design and installation processes of the compaction grouting, as well as the monitoring results collected to date; also discussed the difficulties encountered during interpretation of the in-situ testing. Some recommendations for design and execution techniques have been proposed to improve the compaction grouting effectiveness and performance degree. The present critical study revealed the following important results:

i. The grout consistency and behaviour must be carefully analysed. Indeed, its formulation must be performed in order to avoid grout rising and ground surface heave
ii. The post-construction long-term monitoring of the structures with tiltmeters, fissuremeters and clinometers will be used to evaluate if the amplitude of structural cracks or ground vertical displacements are allowable
iii. The application of the new compaction grouting method could reduce the surface upheave by more than 90% during the injection in urbanized areas where the allowable displacement amount is severe
iv. The ground water conditions monitoring would help understand how the consolidation process of the surrounding soils compacted is going on
v. The density verification through the same techniques used (SPT, CPT and ERT) in the pre-grouting phase would help to make a comparison to appreciate the densification degree

vi. The cored drilling of some grouted columns would help verify their compressive strength and assure their resistance to structural loads.

Despite, its evolution, compaction grouting which treats not only the symptom of the problem (subsidence) but also the cause (the underlying soft soil); thereby cost-effectively, and with limited intrusion extend the life of the structure; does not still have its design standards. Indeed, principally based on previous experience and relied on pre- and post-construction evaluation of soil properties, this design practice is not always suitable.

Nevertheless, the methodology used, this study presents some limitations which are the lack of the soil mineralogical and chemical analyses, the grout mix batch and the pump specifications.

For future research, it could be interesting to find the suitable grout composition able to avoid ground rising; study the soil consolidation process after grouting, study the jet-grouting as an alternative to the compaction grouting or study the field execution procedure of the new CG method.

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ANNEX

Annex I – Geotechnical investigation (in-situ &laboratory tests)

Table A.1. DPSH-A apparatus characteristics (DIN 4094 standards)

Materials	Hammer weight	Drop height	Cone section (90° / D=3.6cm)	Rod weight (D=2.2 cm)	Mass	velocity
DPSH-A FORDYA	63.5 kg	0.75 cm	20 cm^2	6.20 kg	18.00 kg	15-30 drop/min

Figure A.1. Exploratory drilling operation (from data project)

Annex II – Geophysical investigation (tomography test)

Figure A.2. Mangusta geo-resistivity meter from AMBROGEO (from data project)

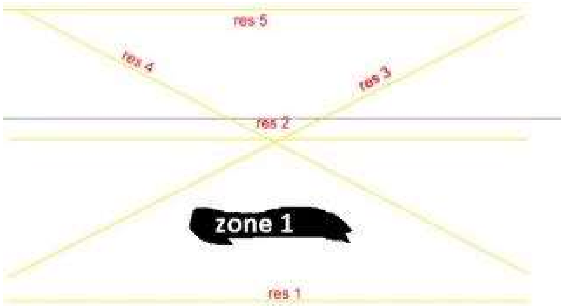

Figure A.3. Zone A profiles distribution – RES1 linear electrodes disposition

Figure A.4. Zone B profiles distribution – RES11 linear electrodes disposition.

Figure A.5. Zone C profiles distribution – RES16 linear electrodes disposition

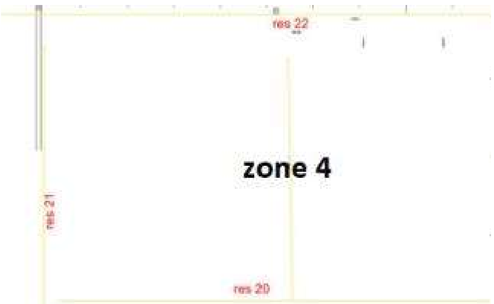

Figure A.6. Zone D profiles distribution – RES21 linear electrodes disposition

Annex III – 2D Tomography of RES profiles in Zone A

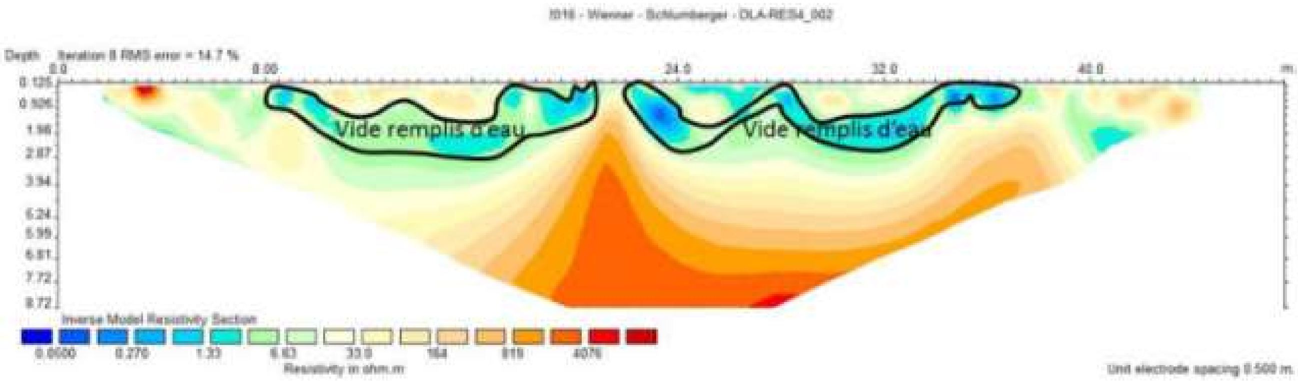

Figure A.7. Zone A RES4 2D tomography

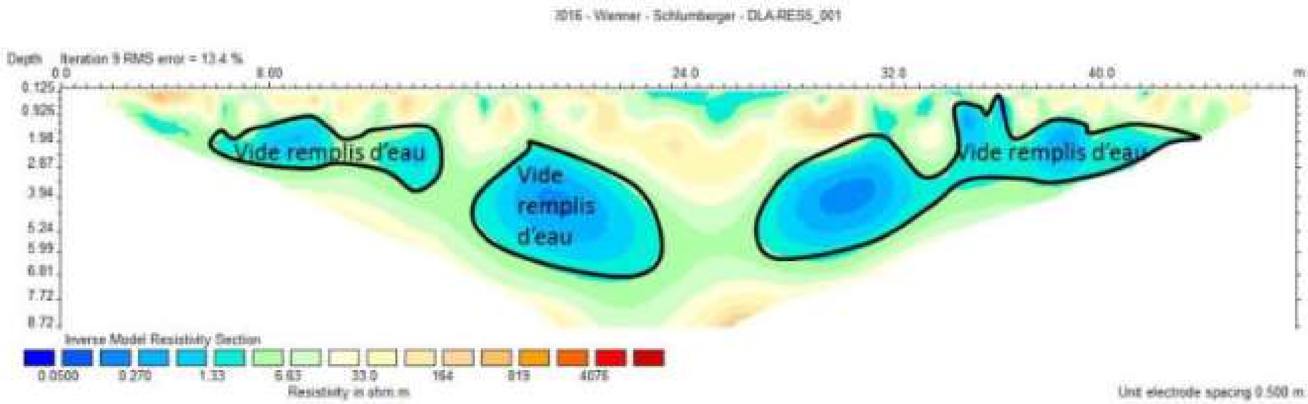

Figure A.8. Zone A RES5 2D tomography

Annex IV – 2D Tomography of RES profiles in Zone B

Figure A.9. Zone B RES14 tomography results

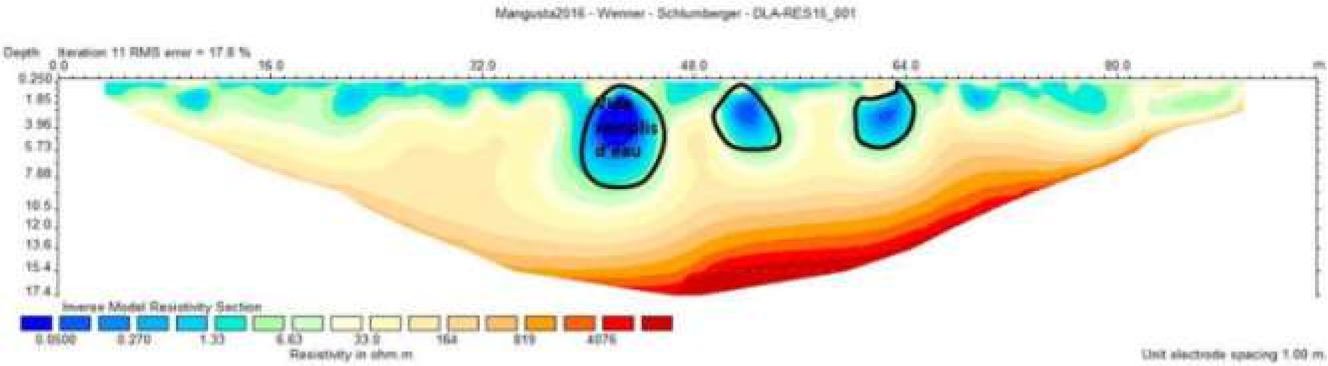

Figure A.10. Zone B RES15 tomography results

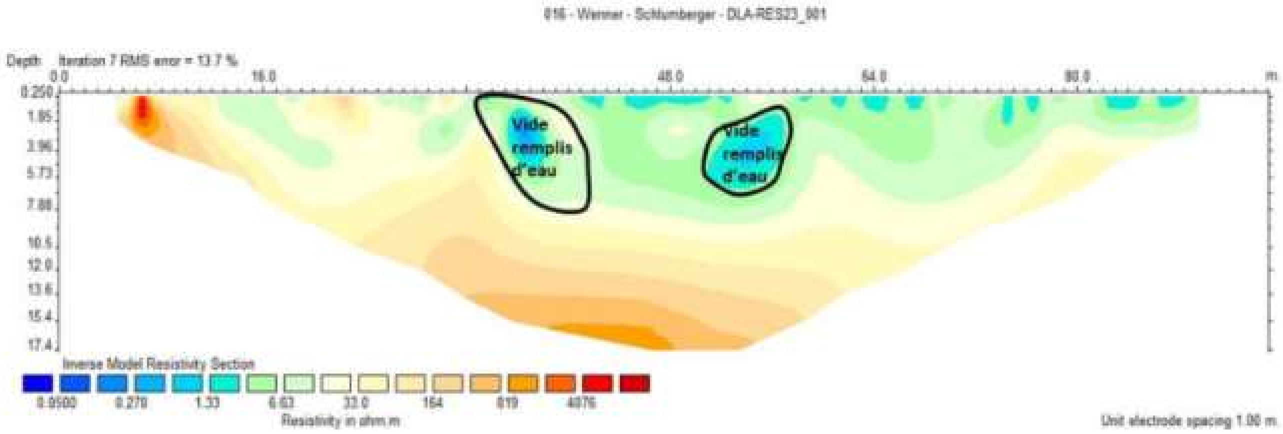

Figure A.11. Zone B RES23 tomography results

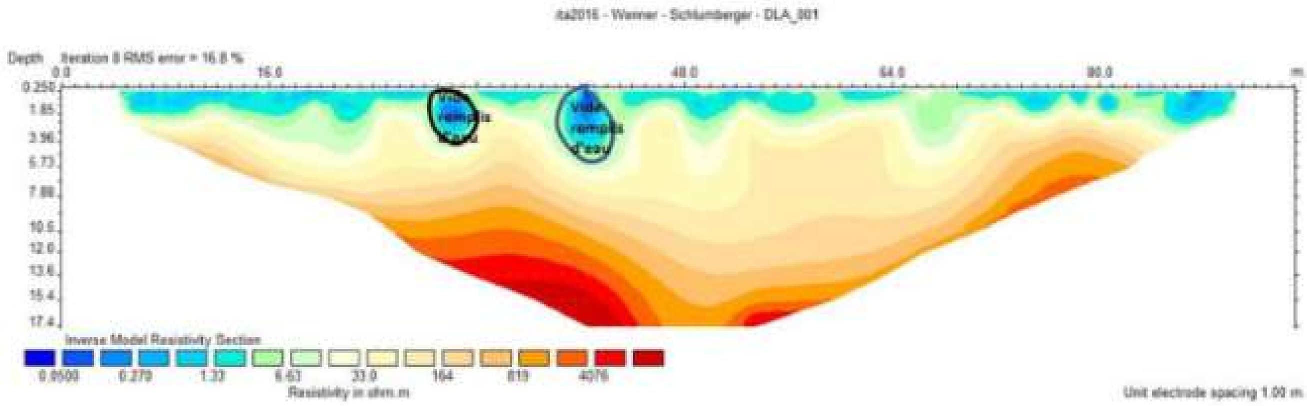

Figure A.12. Zone B RES24 tomography results