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STABILITY OF A HIGH EMBANKMENT ON SOFT SOIL ALONG THE BANGANGTE-FOUMBOT-BAMENDJING-GALIM ROAD

A thesis submitted in partial fulfilment of the requirement for a degree of Master of Engineering (MEng) in Civil Engineering. Curriculum: Geotechnical Engineering

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DEDICATION

This research project is dedicated to all my family members, who gave me every day the encouragement to afford this work.

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Words cannot adequately express my debt of gratitude to the almighty GOD who is the ultimate source of knowledge and strength

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LIST OF NOTATTIONS AND GLOSSARY OF ABBREVIATION

A	Area of the road section
A_s	Area of stone columns
BS	British standard
c′	Effective cohesion of the soil
C_c	Compression index
c_u or s_u	Undrained shear strength of the soil
C_s	Swelling index
C_α	Secondary consolidation index
CPT	Cone Penetrometer Test
D	Stone column diameter
Dr	Relative density of soil
D_w	Water table level
e_0	Initial void ratio
E_0	The soil's modulus of elasticity at depth z = 0 at the bottom of the foundation
E_t	the soil's modulus of elasticity at depth z ≥ 0
E_{oed}	Oedometric modulus
FOS	Factor of safety
FEM:	Finite Element Model
GTX:	Geotextiles
H	Height of the stone columns
K_{ac}	Active earth pressure generated by material of stone columns
K_{at}	Active earth pressure generated by soil
K_0	At rest coefficient
k_h	Horizontal permeability
k_v	Vertical permeability
L	Soil width between two columns

m_v	Volume compressibility
N	number of stone columns
OCR	Over Consolidation Ratio
SF	Safety Factor
S_v	Vertical settlement
S_h	Horizontal settlement
SPT	Standard Penetrometer Test
S	Spacing of columns
α	Area replacement ratio
γ	Unit weight of the homogeneous soil
γ_w	Saturated unit weight of the homogeneous soil
υ	Poisson's ratio
σ'_c	preconsolidation pressure, derived from e ~ log p curve using the Casagrande method
σ'_0	average effective stress
σ'_{vt}	Effective vertical stress in soil
σ'_{vc}	Effective vertical stress in stone columns
σ'_{ht}	Effective horizontal stress in soil
σ'_{hc}	Effective horizontal stress in stone columns
ϕ'	Internal frictional angle in drained condition
ϕ_u	Internal frictional angle in undrained condition
λ^*:	Modified compression index
κ^*:	Modified swelling index,
μ^*:	Modified secondary compression index,
ψ:	Dilatancy angle
γ_{dry}:	Dry unit weight

ABSTRACT

The objective of this work was to show that with an adequate ground reinforcement technique, the stability of a high embankment built on soft soil can be attained within acceptable time frame and costs. Soft clays are found in many road and highway projects in Cameroon and around the world. The strength of these clays is generally very low and as such, embankment failures can be encountered during and after construction such that settlement control, slope stability, and soil bearing capacity assessment become great challenges to construction of road embankments.. Ground improvement by using stone columns which is the method proposed in this study, is one of the techniques to improve soft soils for the foundation of high embankments. This study was conducted using a numerical analysis method with the aid of the software PLAXIS 2D for the simulation and analysis of a 6m high embankment built on one of the swampy areas of the Bangangté-Foumbot-Bamendjing-Galim road section. The results of our numerical analysis revealed that for an unreinforced soil, the settlement values are more important and occur over a long period of time. The modelling and simulation of the embankment on a stone columns reinforced foundation shows an improvement of the bearing capacity of the soil which leads to lower values of settlement and reduces the consolidation end time by approximately 35% compared to the unreinforced case, which guarantees the stability of the embankment by granting a minimum factor of safety of 1.12 in a shorter time. The study concludes with a cost analysis that confirmed the effectiveness of the solution we propose compared to the one carried out on site on an economic basis.

Key words: stability, ground improvement, soft soil, consolidation, stone columns

RESUME

L'objectif de ce travail était de montrer qu'à l'aide d'une technique de renforcement des sols adéquate, la stabilité d'un remblai de grande hauteur construit sur un sol compressible peut être atteinte dans des délais et à des couts acceptables. Les argiles molles se retrouvent dans de nombreux projets de routes et d'autoroutes au Cameroun et dans le monde. La résistance de ces argiles est généralement très faible et par conséquent, des ruptures de talus peuvent être rencontrées pendant la construction, de sorte que le contrôle du tassement, la stabilité des pentes et l'obtention de la capacité portante du sol acceptable deviennent de grands défis pour la construction des remblais routiers. L'amélioration du sol à l'aide de colonnes ballastées, qui est la méthode proposée dans ce travail est l'une des techniques permettant d'améliorer les sols mous pour la fondation des grands remblais. Cette étude a été menée avec une méthode d'analyse numérique grâce au logiciel PLAXIS 2D permettant la simulation et l'analyse d'un remblai de 6m de hauteur construit sur l'une des zones marécageuses de la section de route Bangangté-Foumbot-Bamendjing-Galim. Les résultats de notre analyse numérique ont révélé que pour un sol non renforcé, les valeurs de tassement sont plus importantes et se produisent sur une longue période. La modélisation et simulation du remblai sur une assise renforcée par colonnes ballastées montre une amélioration de la capacité portante du sol qui conduit à des valeurs de tassement plus faibles et réduit le temps de fin de consolidation d'environ 35% par rapport au cas non renforcé, ce qui garantit la stabilité du remblai en octroyant facteurs de sécurité minimum égale à dans des délais plus courts. L'étude s'achève par une analyse des coûts qui a confirmé l'efficacité de la solution que nous proposons par rapport à celle réalisée sur le site sur une base économique.

Mots clés : stabilité, amélioration du sol, sol mou, consolidation, colonnes ballastées.

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GENERAL INTRODUCTION

In Cameroon like in other parts of the world, during the construction of roads, railways and other linear infrastructures, soft soils, known as soils with very low bearing capacities, high settlement susceptibilities, high water contents and high liquefaction potentials are always encountered. Generally, dealing with these soils to ensure the stability of the infrastructure requires a lot of time and money. That said, the Geotechnical engineers face problems of prolonged settlements and obviously, instability.

As remedy to these problems, Geotechnical engineers developed many solutions such as, soil substitution, preloading, prefabricated vertical drains, vertical sand drains, deep mixing, jet grouting, injection, geosynthetics, step construction, ordinary stone columns, encased stone columns… based on the realities on site, the method chosen for reinforcement in this research is reinforcement using stone columns for its efficiency.

The objective of this work therefore consists in the design of an adequate stone column supported embankment to guarantee the stability of the six (6) meters high embankment constructed between PK 12+650 and PK 12+950 along the Bangangté-Foumbot-Bamendjing-Galim road, section 2. Also, this work will extend to showing that the use of stone columns for reinforcement has a big financial and time frame incidence on the project.

To achieve the objectives announced in the previous paragraph, the work was divided into three main chapters. The first chapter deals with the literature review elaborating on the history and main features of soft soils and embankments in the first part and the second part concerns an overview on the notions of stability and ground improvement in geotechnics; it starts by elaborating on the different approaches to evaluate the safety factors according to different conditions and authors and finally elaborates on different improvement techniques but more precisely on improvement by the use of stone columns. Chapter two on its own shows the methodology achieved to obtain the results presented in chapter three. Chapter three then concludes with the presentation of the results obtained from the site visit and project documentation, followed by the geometrical and geotechnical data obtained from insitu and laboratory tests. The next part is the presentation of the results from the analytical and the numerical designs. The numerical design results inform on the behavior of the embankment under unreinforced and reinforced conditions. The main results necessary to evaluate the efficiency of the proposed solution stated in this work are the evolution of settlement in time for different conditions, the evolution of excess pore water pressure in time for different conditions, evolution of safety factors in time for different conditions, analysis of consolidation end times and finally the cost analysis of our solution.

Chapter 1 : LITERATURE REVIEW

Introduction

All soils are compressible, i.e. they settle when a vertical load is applied to them. The amplitude and speed of this compaction can vary up to large proportions depending on the type of soil considered. Soft soils are generally newly formed, not suitable for supporting a structure, but can, with certain precautions, be used as a foundation for an embankment.

Researches show that settlement, slope stability, and soil bearing capacity are all challenges to construction of road embankments on soft soils.

Gravitational and seepage forces tend to cause instability in natural slopes, slopes formed by excavation and slopes of embankments. Natural forces (wind, water, snow, etc.) change the topography on Earth and other planets, often creating unstable slopes. Failures of natural slopes (landslides) and man-made slopes have resulted in much death and destruction, economic losses, and environmental damages. Slopes therefore have to be designed properly to assess stability before being constructed.

Most engineering constructions are carried out in soil and, obviously, poor soil conditions will be encountered on some construction sites. If such soil cannot be removed totally, then its engineering behaviour can often be enhanced by some method of ground treatment. Poor soil conditions usually are attributable to an excess of ground water or a lack of strength, and associated deformability. Treatment methods are therefore aimed at preventing ingress of groundwater to or removing it from the site in question on one hand or improving soil strength on the other.

With regard to the previous paragraphs, this chapter will deal firstly with the notion of soft soils, a review on their origin, typology and properties shall be made, the second part dedicated to embankments will elaborate on the definition and use of embankments in geotechnical engineering and finally the behaviour of problems of embankments on soft soils. The third part on its own deals with the notion of slope stability while the last deals with the notion of ground improvement.

1.1. Soils

To better understand the notion of soft soils, we shall talk about their origin (process of formation), typology and their properties. Many types of soil exist but in the case of the work, attention will be placed on soft soils.

1.1.1. Origin

The actions of frost, temperature, gravity, wind, rain and chemical weathering are continually forming rock particles that eventually become soils. To the civil engineer, soil is any un-cemented or weakly cemented accumulation of mineral particles formed by the weathering of rocks, the void space between the particles containing water and/or air (Craig, 2004). There are three types of soil when considering modes of formation.

1.1.1.1. Transported soil (gravels, sands, silts and clays)

Most soils are transported by water. As a stream or river loses its velocity y it tends to deposit some of the particles that it is carrying, dropping the larger, heavier particles first. Hence, on the higher reaches of a river, gravel and sand are found whilst on the lower or older parts, silts and clays predominate, especially where the river enters the sea or a lake and loses its velocity. Ice has been another important transportation agent, and large deposits of boulder clay and moraine are often encountered. In arid parts of the world, wind is continually forming sand deposits in the form of ridges. The sand particles in these ridges have been more or less rolled along and are invariably rounded and fairly uniform in size. Light brown, wind-blown deposits of silt-size particles, known as loess, are often encountered in thin layers, the particles having sometimes travelled considerable distances (Smith, 2014)

1.1.1.2. Residual soil (topsoil, laterites)

These soils are formed in situ by chemical weathering and may be found on level rock surfaces where the action of the elements has produced a soil with little tendency to move. Residual soils can also occur whenever the rate of breakup of the rock exceeds the rate of removal. If the parent rock is igneous or metamorphic the resulting soil sizes range from silt to gravel. Laterites are formed by chemical weathering under warm, humid tropical conditions when the rainwater leaches out of the soluble rock material leaving behind the insoluble hydroxides of iron and aluminium, giving them their characteristic red-brown colour (Smith, 2014).

1.1.1.3. Organic soil

These soils contain large amounts of decomposed animal and vegetable matter. They are usually dark in colour and give off a distinctive odour. Deposits of organic silts and clays have usually been created from river or lake sediments (Smith, 2014)

1.1.2. Types

Soft soil can be divided into four main categories namely; peat soils, marls, mud, silts and clays.

1.1.2.1. Peat

Prabir K. Koplay and Siti Taid (2018) defined Peat as an organic complex soil (contains more than 30% organic material), well known for its high compressibility and low stability and which cover about 5–8% of land area. Peat is formed naturally by the incomplete decomposition of plant and animal constituents under anaerobic conditions at low temperatures.

Researches such as conducted by some university faculties[1] have shown that peat is an accumulation of the debris of plant, trunk and roots in water logging condition which is usually in light brown or dark colour.

It has high organic matter and therefore it can be classified into different types of peat soil based on different level of humification of peat. Humification can be defined as the transformation of numerous groups substances (proteins, carbohydrates, lipids, etc.) and individual molecules present in living organic matter into groups of substances with similar properties (humic substances) (T. Teong and al., 2016). After that, the humic substances are changed into carbon deposits. In the carbon biogeochemical cycle, transformation from living organic to the carbon substances, in terms of humification is affected by the high variability of environmental conditions and affected by degradation and synthetic reaction.

In geotechnical field, peat soil is defined as a problematic soil due to its physical properties. Peat soil which has very high natural moisture content, high fibber content, high compressibility and water-holding capacity, low specific gravity, low bearing capacity, medium to low permeability and low shear strength cause the soil cannot support high loading and not suitable for construction (T. Teong and al., 2016). Hence, characterization and improvement of peat is necessary to construct any type of infrastructure on it. Figure 1.1 shows the natural texture of a black peat soil.

Figure1.1. Peats (Antonio Jordan, 2015)

[1] Faculty of Civil and Env. Eng., Research center for Soft soil of Universiti Tun Hussein Onn Malaysia

1.1.2.2. Mud

The soft muddy soil is considered as an unconsolidated soft fine grained soils deposited and formed in a standing or slow flowing water environment by physical, chemistry and biological effects, which belongs to modern recent sediments (Qianwen Wang and al., 2018). When encountering soft muddy soil in engineering construction, traditional methods, such as throw fill disposal, dehydration, heat treatment and blow-filled land reclamation, are limited in practical engineering application, such as, large occupation of engineering site, great increase of economic cost, secondary pollution, project delay and unsatisfactory construction effect, etc. Muddy soil has higher natural moisture content and natural pore ratio (Qianwen Wang and al., 2018).

The mud generally contains a small proportion of organic matter. They can be peaty if the presence of certain micro-organisms promotes the formation of peat. In coastal areas, the presence of sodium chloride prevents the proliferation of these micro-organisms and therefore the deposited mud is not peaty. Figure 1.2 shows the natural texture of a dry mud soil

Figure1.2. Mud (Boughton)

1.1.2.3. Silts

For Ray Bradley (2018), Silt is a light and moisture retentive soil type with a high fertility rating up of rock and mineral particles that are larger than clay but smaller than sand. Individual silt particles are so small that they are difficult to see. To be classified as silt, a particle must be less than 5* 10-2 millimetres and more than 2* 10-3 millimetres (Morgan Stanley, 2011).

As silt soils compromise of medium sized particles they are well drained and hold moisture well. As the particles are fine, they can be easily compacted and are prone to washing away with rain.

Silty soil is slippery when wet, not grainy or rocky. The soil itself can be called silt if its silt content is greater than 80 percent. By adding organic matter, the silt particles can be bound into more stable clumps (Morgan Stanley, 2011). Figure 1.3 shows the natural texture of a dry silt

Figure1.3. Silts (Boughton)

1.1.2.4. Clays

It is generally believed that rock fragments can be reduced by mechanical means to a limiting size of about 0.002mm, so that a soil containing particles above this size has a mineral content similar to the parent rock from which it was created. For the production of particles smaller than 0.002mm some form of chemical action is generally necessary before breakdown can be achieved. Such particles, although having a chemical content similar to the parent rock, have a different crystalline structure and are known as clay particles. An exception is rock fluor, rock grains smaller than 0.002mm, produced by the glacial action of rocks grinding against each other (Smith, 2014). The three main groups of clay minerals are as follows;

a) Kaolinite group

This mineral is the most dominant part of residual clay deposits and is made up from large stacks of alternating single tetrahedral sheets of silicate and octahedral sheets of aluminium. Kaolinites are very stable with a strong structure and absorb little water. They have low swelling and shrinkage responses to water content variation.

b) Illite group

Consists of a series of single octahedral sheets of aluminium sandwiched between two tetrahedral sheets of silicon. In the octahedral sheets some of the aluminium is replaced by iron and magnesium and in the tetrahedral sheets there is a partial replacement of silicon by aluminium. Illites tend to absorb more water than kaolinites and have higher swelling and shrinkage characteristics.

c) Montmorillonite group

This mineral has a similar structure to the illite group but, in the tetrahedral sheets, some of the silicon is replaced by iron, magnesium and aluminium. Montmorillonites exhibit extremely high water absorption, swelling and shrinkage characteristics. Bentonite is a member of this mineral group and is usually formed from weathered volcanic ash. Because of its large expansive properties when it is mixed with water it is much in demand as a general grout in the plugging of leaks in reservoirs and tunnels. It is also used as a drilling mud for soil borings. Readers interested in this subject of clay mineralogy are referred to the publication by Murray (2006).

The two structures of the clay deposits are as follows;

- Macrostructure; The visible features of a clay deposit collectively form its macrostructure and include such features as fissures, root holes, bedding patterns, silt and sand seams or lenses and other discontinuities. A study of the macrostructure is important as it usually has an effect on the behaviour of the soil mass. For example, the strength of an unfissured clay mass is much stronger than along a crack.
- Microstructure; The structural arrangement of microscopic sized clay particles, or groups of particles, defines the microstructure of a clay deposit. Clay deposits have been laid down under water and were created by the settlement and deposition of clay particles out of suspension. Often during their deposition, the action of Van der Waals forces attracted clay particles together and created flocculant, or honeycombed, structures which, although still microscopic, are of considerably greater volume than single clay particles. Such groups of clay particles are referred to as clay flocs. Figure 1.4 shows natural appearing dry clay.

Figure1.4. Clays (Boughton)

1.1.3. Properties and behaviour of soft soils

The three main properties on which emphasis is laid when dealing with soft soils are permeability, bearing resistance (capacity) and deformability.

1.1.3.1. Permeability

The permeability of a soil represents the rate at which water or any other liquid finds its way through it. In the case of our research, we will focus on how water interacts with the soil (soft). The parameter used to define the permeability of a material is its **coefficient of permeability, $\boldsymbol{k}$.** It is important to realise that when a soil is said to have a certain coefficient of permeability, this value only applies to water (at 20°C). Although changes in temperature cause changes in the value of k, this change is less appreciable in most soil works, (Smith, 2014). The coefficient of permeability (measured in ms^{-1}) can be defined from Darcy's law as the rate of flow of water per unit area of soil when under a unit hydraulic gradient (Smith, 2014). This can be illustrated by the following equations.

$$v = ki \quad \text{(E1.1)}$$

or

$$Q = Atki \quad \text{(E1.2)}$$

or

$$q = Aki \quad \left(where\ q = quantity\ of\ unit\ flow = \frac{Q}{t}\right) \quad \text{(E1.3)}$$

The permeability of soils (soft soils) can be determined in the laboratory through three different tests that are, the **constant head permeameter**, the **falling head permeameter** and the **hydraulic consolidation cell (Rowe cell)** while in the field, two different tests enable the determination of the permeability which are the **pumping out tes**t and the **pumping in test**, (Wiley, 2014). The permeability of soils is often obtained reliably via in situ and laboratory tests in two components namely the vertical (k_v) and horizontal (k_h) components which are related to the in situ current void ratio and the structural condition of the natural deposit.

In the laboratory, standard incremental-loading oedometer tests are customarily used to determine the consolidation coefficient c_v for predicting in situ consolidation time. Vertical permeability k_v can be calculated using the relationship, given by Terzaghi's one-dimensional theory of consolidation, between c_v, determined here by Taylor's method, and the constrained modulus M using the relationship in equation 1.4

$$k_v = \frac{\gamma_w c_v}{M} \quad \text{(E1.4)}$$

where γ_w is the unit weight of water.

The coefficient of permeability of soft soils generally vary from $\mathbf{10^{-6}}$ to $\mathbf{10^{-11}}$ ms^{-1} (AMSOL, 2010) which is relatively very small compared to that of sand and gravel explaining why consolidation takes place slowly in soft soil ($c_v \propto k_v$ from E1.4)

1.1.3.2. Resistance

Soft soils generally are characterised by very low mechanical resistances. The compressibility properties, undrained shear strength, and stress history are essential for reliable calculations of settlement and bearing capacity of soft soils. The oedometer test is used to determine the later parameters as discussed in the following lines;

The compressibility of soft soil (e.g. Clay) can be represented by one of the following coefficients.

- The coefficient of volume compressibility (m_v); defined as the volume change per unit volume per unit increase in effective stress. The units of m_v are the inverse of pressure (m^2/MN). The volume change may be expressed in terms of either void ratio or specimen thickness. If, for an increase in effective stress from σ_0' to σ_1' , the void ratio decreases from e_0 to e_1, then

$$m_v = \frac{1}{1+e_0}\left(\frac{e_0-e_1}{\sigma_1'-\sigma_0'}\right) \qquad \text{(E1.5)}$$

$$m_v = \frac{1}{H_0}\left(\frac{H_0-H_1}{\sigma_1'-\sigma_0'}\right) \qquad \text{(E1.6)}$$

The value of m_v for a particular soil is not constant but depends on the stress range over which it is calculated. BS 1377 (British Standard 1377) specifies the use of the coefficient m_v calculated for a stress increment of $100\ kN/m^2$ in excess of the effective overburden pressure of the in-situ soil at the depth of interest, although the coefficient may also be calculated, if required, for any other stress range.

- The compression index (Cc); is the slope of the linear portion (portion bc) of the $e-\log\sigma'$ plot in Figure 1.5 and is dimensionless. For any two points on the linear portion of the plot.

$$C_c = \frac{e_0-e_1}{\log\left(\frac{\sigma_1'}{\sigma_0'}\right)} \qquad \text{(E1.7)}$$

The expansion part of the $e-\log\sigma'$ plot can be approximated to a straight line, the slope of which is referred to as the **expansion index C_e** as can be seen in Figure 1.5.

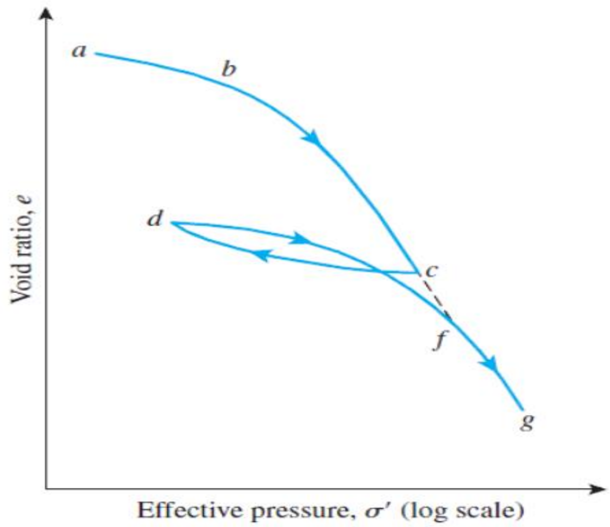


Figure1.5. Graph of void ratio versus logarithm of effective vertical stress (BRAJA M, 2010)

The **Undrained shear strength** of soft soils (e.g. Clay) can be obtained by the piezocone test data (other tests like the vane shear test can be used) using a number of equations (Lunne et al., 1997; Schnaid, 2009). The most commonly-employed equations relate the corrected cone resistance (q_T) to the cone factor (N_{kt}), and pore pressure to the cone factor ($N_{\Delta u}$). Therefore, an accurate determination of the cone factor values is quite important for a precise estimation of undrained strength using piezocone test data.

Profile of the soft soil's undrained strength (S_u) is then obtained from the results of vane shear test correlated with piezocone tests data and by using the corresponding empirical cone factors stated earlier. Undrained strength can also be estimated using a mathematical derivation proposed by Mantaras et al. (2015) which takes into account a number of critical factors affecting the S_u values including soil stress history, clay stiffness, and excess pore pressure during piezocone tests. The method of Mantaras et al. (2015) uses the following equation to determine the undrained shear strength:

$$S_u = \frac{\Delta u_{max}}{4.2 log I_r} \quad \text{.....(E1.8)}$$

Where $\Delta u_{max} = u_{2,max} - u_0$, in which $u_{2,max}$ is excess pore pressure measured in cone tip and u_0 is the hydrostatic pore pressure. The data provided by the three piezocone tests can then used to estimate the undrained strength using the Equation 1.8 according to Mantaras et al. (2015).

Soft soil sensitivity (S_t), defined as the ratio of peak strength (S_u) to remoulded strength (S_{ur}), can then be obtained using vane test data.

The stress history of the soft soil is another important parameter that influences the resistance of a soft soil as described in the following paragraph.

A careful determination of pre-consolidation stress (σ_c') is particularly important for a precise estimation of the settlement of embankments over soft clay deposit. The pre consolidation stresses can directly be determined using oedometer consolidation tests data. The stress history is basically established by the overconsolidation ration (OCR) which is defined as the maximum value of effective stress in the past divided by the present value. Figure 1.6 shows the variation of void ratio with effective vertical stress.

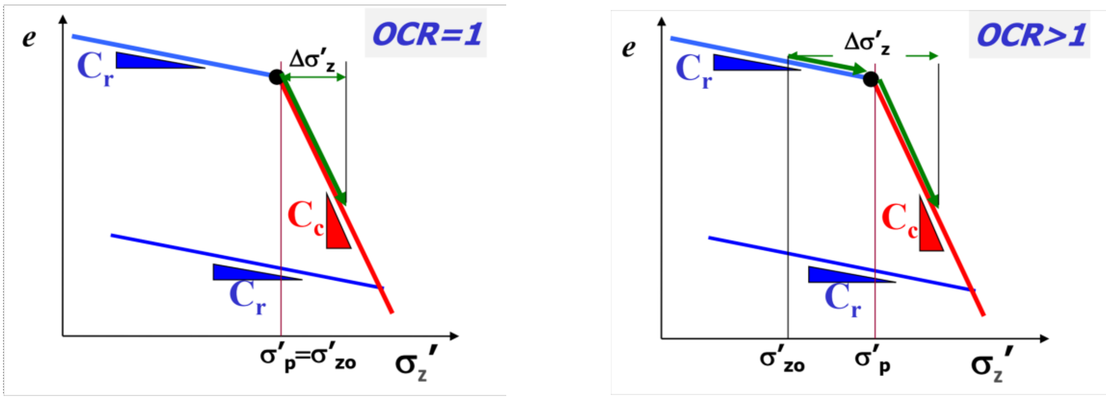


Figure1. 6. Void ratio versus effective vertical stress for both normally and overconsolidated clay from oedometer test

1.1.3.3. Deformability

When external or internal forces acting on a solid body vary, the body changes its stress state and can experience:

- a rigid displacement or rotation
- deformation composed of volume variation and form variation

In the case of embankments constructed on soft soil, the main effect of the variation of stress state is the deformation composed of volume variation and form variation.

When on the lateral sides of an infinitesimal volume of soil (with size x, y and z) only normal forces act (i.e. normal stress without shear stress), the volume reduce itself with a movement of the side normally to the stress (only volume variation without distortion of the cube). We indicate with $\Delta x, \Delta y, \Delta z$ the displacements of each side parallel to the reference axis x, y, z. We can define longitudinal strains the ratios (Budhu, 2011):

$$\varepsilon_x = \frac{\Delta x}{x},\ \varepsilon_y = \frac{\Delta y}{y},\ \varepsilon_z = \frac{\Delta z}{z} \quad \text{(E1.9)}$$

When on the lateral sides of an infinitesimal volume of soil only shear forces act (i.e. shear stress without normal stress), the body deforms changing its form without change in volume. We indicate with $\Delta x, \Delta y, \Delta z$ the displacements of each side parallel to the reference axis x, y, z. We can define angular strains the ratios:

$$\gamma_{zx} = \arctan\left(\frac{\Delta x}{z}\right) \approx \frac{\Delta x}{z},\ \gamma_{xy} = \frac{\Delta y}{x},\ \gamma_{yz} = \frac{\Delta z}{y} \quad \text{(E1.10)}$$

For the hypothesis of small strains

The total vertical movement at the surface resulting from the load, generally oriented downward, is called **settlement**. On the contrary, when a soil deposit is unloaded (excavation) the movement of ground surface is upward and we speak of **swelling**.

Since soft soils are mostly completely saturated, the change in volume can occur only if water is forced out of the voids. As pore fluid squeezed out:

- Soil grain rearrange themselves in a more stable and denser configuration
- Decrease in volume and surface settlements are observed
- Higher degree of rearrangement and compression are observed compared to granular soils.
- Compression of soft (fine) soils is a very time dependent process, compression occurs very slowly due to low permeability: we call consolidation the slow water squeezing process, coupled with deformation occurrence.

Given the vertical strain definition we can calculate the settlement (ΔH) in different ways, depending on OCR and the vertical stress increment

$$\Delta H = H_0 \frac{\Delta e}{1+e_0} = \begin{cases} \frac{H_0}{1+e_0} C_c \log\left(\frac{\sigma'_{z0}+\Delta\sigma'_z}{\sigma'_{z0}}\right) & if\ OCR = 1 \\ \frac{H_0}{1+e_0} (C_r \log\frac{\sigma'_P}{\sigma'_{z0}} + C_c \log\left(\frac{\sigma'_{z0}+\Delta\sigma'_z}{\sigma'_P}\right) & if\ OCR > 1\ and\ \sigma'_{z0} + \Delta\sigma'_z > \sigma'_P \end{cases}$$

(E1.11)

1.2. Embankments

Embankments are widely used in civil engineering works, a better understanding of their definition, use and behaviour on soft soil will help better treat this work.

1.2.1. Definition and use

An embankment is defined as an earthmoving material implemented by compacting and intended to raise the profile of a piece of land or to fill in an excavation. It is therefore an earthmoving operation which consists of raising the level of a road in relation to the existing natural soil by means of a volume of earth which is placed and then compacted. The fill is usually made of soil material, but it can also be made of aggregates and rocks.

1.2.2. Embankment behaviour on soft soils (failure mechanisms)

The construction of an embankment results in increases in total stress, both within the embankment itself as successive layers of fill are placed and in the foundation soil (Craig, 2004). The initial pore water pressure (u_0) depends primarily on the placement water content of the fill. The construction period of a typical embankment is relatively short and, if the permeability of the compacted fill is low, no significant dissipation is likely during construction. Dissipation proceeds after the end of construction with the pore water pressure decreasing to the final value in the long term (Figure 1.7). The factor of safety of an embankment at the end of construction is therefore lower than in the long term. Shear strength parameters for the fill material should be determined from tests on specimens compacted to the values of dry density and water content to be specified for the embankment. The stability of an embankment may also depend on the shear strength of the foundation soil. The possibility of failure along a surface such as that illustrated in Figure 1.8 should be considered in appropriate cases.

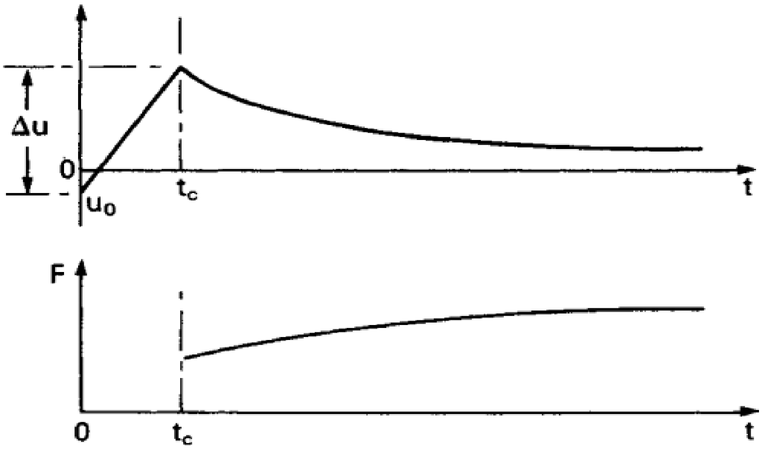


Figure1.7. Pore pressure dissipation and factor of safety in an embankment (Craig, 2004)

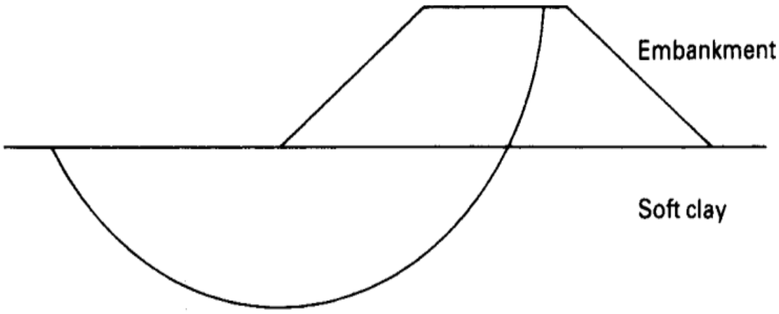


Figure1.8. Failure below an embankment (Craig's, 2004)

Embankments are sometimes built on weak foundation materials. **Sinking, spreading** and **piping** failures may occur irrespective of the stability of the overlying embankment material. Consideration of the internal stability of an embankment-foundation system, rather than just the embankment, may be necessary. A simple rule of thumb based on bearing capacity theory can be used to make a preliminary estimate of the factor of safety against circular arc failure for an embankment built over a clay foundation. The rule is (Cheney and Chassie, 1982)

$$FS = \frac{6c}{\gamma_{fill} \times H_{fill}} \qquad \text{(E 1.15)}$$

Where $FS = factor\ of\ safety$

$c = cohesion\ of\ foundation\ clay$

$\gamma_{fill} = unit\ weight\ of\ embankment\ fill$

$H_{fill} = height\ of\ embankment\ fill$

The factor of safety computed using this rule serves only as a rough preliminary estimate of the stability of an embankment over a clay foundation and is not be used for final design. The simple equation does not take into consideration factors such as fill strength, strain incompatibility between embankment fill and the underlying foundation soil, and the fill slope angle. In addition, it does not identify the location of a critical failure surface. If the factor of safety using the rule-of-thumb equation is less than 2.5, a more sophisticated stability analysis is required (Chaney and Chassie, 1982).

Figure 1.9 shows the variations in safety factor, strength, pore pressures, load and shear stresses with time with time for an embankment constructed over deposit. Over time, the excess pore pressure in the clay foundation diminishes, the shear strength of the clay increases, and the factor of safety for slioe failure increases.

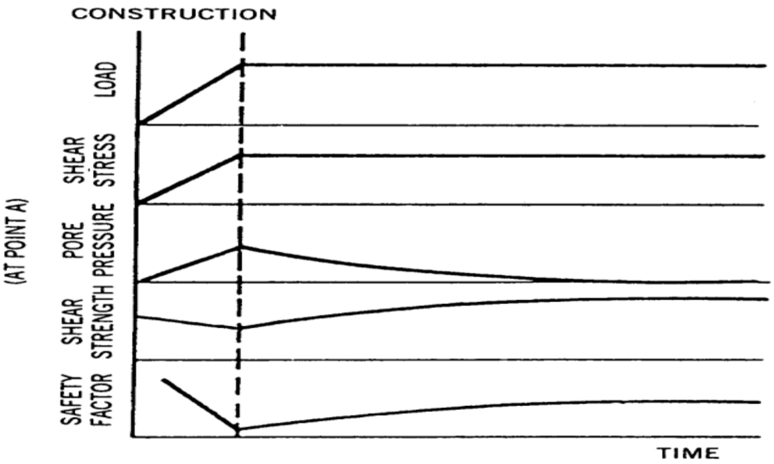


Figure1.9. Stability condition for an embankment slope over a clay foundation(W. Abramson, 2002)

Embankment fills over soft clay foundations are frequently stronger and stiffer than their foundations. This leads to the possibility that the embankment will crack as the foundation deforms and settles under its own weight and to the possibility of progressive failure because of the stress-strain incompatibility between the embankment and its foundation. Design charts developed by Chirapuntu and Duncan (1977), using finite element method (FEM) analyses, depict the effects of cracking and progressive failure on the stability of embankments on soft foundations. These charts may be used as a supplement to conventional stability analyses. The use of geosynthetic reinforcement in the fill may prevent the initiation of cracking and subsequent failure in these cases.

Alternatively, it may be necessary to remove the soft foundation materials or locate in the fill at another site. Figure 1.10 Stress-train compatibility of an embankment on soft clay.

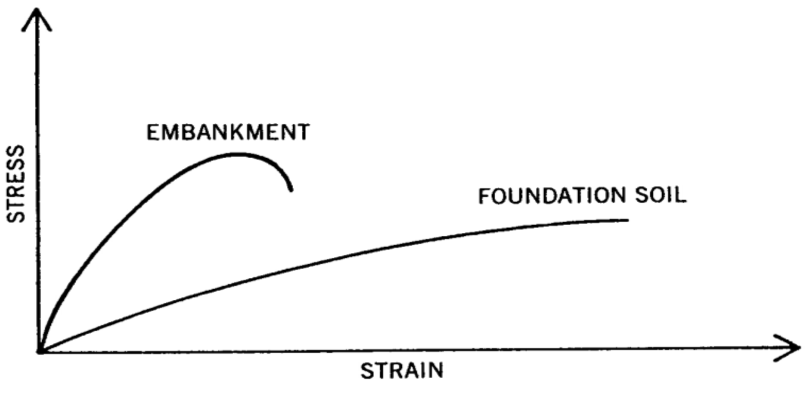


Figure1.10. Stress-strain compatibility of an embankment on soft clay. (W. Abramson, 2002)

Peak strengths of the embankment and the foundation soils cannot be mobilized simultaneously because of stress-strain incompatibility (Figure1.10). Hence, a stability analysis performed using peak strengths of soils would overestimate the factor of safety. Many engineers perform stability analyses using soil strengths that are smaller than the peak values to allow for possible progressive failure.

The construction of embankments on soft soils poses four particular types of problems: instability, settlement, horizontal movement and "parasitic" stresses on neighbouring structures.

1.2.2.1. Instability

Embankments on soft soils experience two forms of instability, including instability capacity by punching and instability by rotation.

a) Instability capacity by punching

Here, the entire embankment sinks by pushing the soil to either sides (Figure 1.11). This type of instability occurs in very soft soil layers from the surface (estuary silt, peat bogs, etc.).

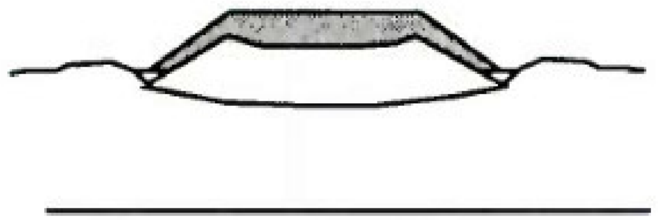


Figure.1.11. Instability capacity by punching (Dadouche, 2002)

b) Instability by rotation

This involves the rotation of part of the embankment and soft soils on a failure surface with a cylindrical shape, with formation of an escarpment in the embankment and a bulge of foot (figure1.12.).

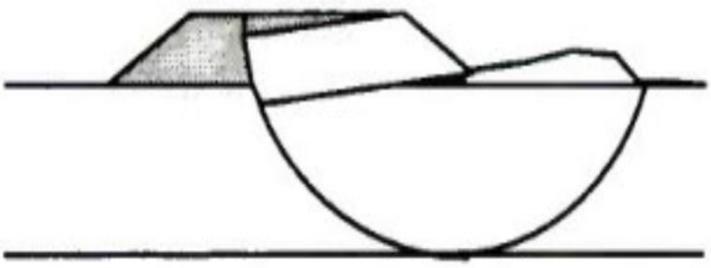


Figure1.12. Instability by rotation (Dadouche, 2002)

Most failures are of the "rotational" type. Most instabilities occur at "short term", during construction of the embankment (or excavation at the foot of the embankment...). In order to assess the stability of an embankment on soft soils, it is necessary to determine the strength of the soil in the short term (undrained cohesion). Construction by stages, which plays on the increase the resistance of the soil over time under the already constructed embankment and the installation of side benches to prevent punching or rotational failure are two of the most common methods to solve the problem of instability by rotation.

1.2.2.2. Settlement

The rules for designing embankments on soft soils limits the load on the soil to values for which its deformations (settlements and horizontal movements) are finished, even if they are large and can last for very long periods of time.

Under normal conditions, a small part of the settlement occurs during the construction of the embankment, mainly during the so-called primary consolidation phase and for the rest during the so-called secondary compression or creep period.

Immediate settlement (during the placement of the successive layers of the embankment) generally develops at constant volume, so that it is accompanied by horizontal displacements of equivalent amplitude. The primary consolidation settlement tends towards its final value following an exponential law. Over pore water pressure remain throughout the process of soil deformation, including during the final creep phase. It should be borne in mind that this three-phase process resets itself each time a new load is applied on the soil i.e. in particular when you come to "reload" the embankment in order to bring it back to its theoretical level.

The process of "preloading" or overconsolidation of the soil (application during the construction work with a load greater than the final weight of the fill and what it carries) is one of the techniques to control this process.

The final settlement amplitudes are usually derived from compressibility tests at the oedometer. The total settlement is usually 10 to 20% higher than the settlement deducted from the oedometric compressibility curve, which corresponds to the effects of creep and the horizontal movements of the soil. However, in heterogeneous soils with alternating sandy and clayey soils, tests carried out on the most clayey part of the cores may give a pessimistic image of the deformability of the soil and also of its permeability.

Variations in pore water pressure in soft soils under embankments accompany consolidation settlement and allow the state of the effective stresses in the soil to be monitored and therefore its resistance. They can also be used to control the stability of the embankment during construction works.

The calculated settlements must be taken into account in the total thickness of the embankment to be placed in place to eventually obtain the altitude desired in the project's specifications. In particular, stability must be analysed taking into account the total thickness of the embankment, including settlements.

If the embankment is constructed in stages with significant settlements at each stage, the width of the embankment platform must be considered at each stage: a geometrical analysis shows that to obtain the desired width of the embankment at the end of the construction, it is unwise to define each step by cutting the theoretical profile of the embankment into horizontal slices, but stiffen the slopes of the embankments or build the embankment on a wider right-of-way.

1.2.2.3. Horizontal movements

The maximum amplitude of the horizontal displacement of soft soils under the embankment generally accounts for 15% of the settlement amplitude. These displacements maintain the same form during consolidation, making it easier to forecast and control them by means of inclinometric measures. Horizontal movements may be more important during the embankment construction (drained conditions). They can be limited by improving the conditions of soil drainage. Horizontal movements of soft soils under embankments are one of the main causes of parasitic stress on neighbouring structures.

1.2.2.4. Parasitic stress on neighbouring structures

Settlements under embankments create negative frictional forces on the piles that are within their influence area (including outside the embankment in some cases). On the other hand, horizontal soil movements also exert horizontal "parasitic" forces on these piles. These additional efforts must be taken into account in the calculation of the deep foundations. They can be limited, or even practically eliminated, if the embankment is constructed sufficiently in advance. Vertical and horizontal movements of the soil may also generate additional forces on the retaining structures located below.

Finally, the construction of an embankment on a soft soil causes the soil surface to settle over a certain distance (depending on the thickness of the soft soil) beyond the foot of the embankment slopes. This settlement can cause the cracking of structures founded superficially in the zone of influence of the embankment. In particular, the failure of embankments on soft soils, which is a complex phenomenon that must be carefully studied.

1.3.OVERVIEW ON STABILITY AND GROUND IMPROVEMENT

This part deals with an overview on stability in its first section and ground improvement techniques in the second section.

1.3.1. Overview on stability (slope)

To easily go through this overview, we shall talk of the definition of slope, limit equilibrium analysis, undrained analysis in clay and numerical methods of analysis of slopes.

1.3.1.1. Definition of slope stability analysis

Slope stability is performed to assess the safe design of a human-made or natural slopes (embankments, road cuts, open-pit mining, excavation, landfills and so on) and the equilibrium conditions. Slope stability refers to the resistance of an inclined surface to failure by sliding or collapsing. The numerical approaches of slope stability analysis can be categorized as either limit (plasticity-type, Limit Equilibrium Method [LEM]) formulation or displacement formulations such as the finite element method (FEM). Limit formulations provide a theoretical context for understanding the range of answers that can be expected from a slope stability analysis (Mendelson, 1968). The goals of slope stability analysis consist of the assessment of the following:

- ➢ Critical slip surface-surface for which the resistance ratio between available and mobilized resistance is the lowest;
- ➢ Safety factor-resistance ratio between available and mobilized resistance;
- ➢ Deformations, displacements and rate.

However, Design is based on the requirement to maintain stability rather than on the need to minimize deformation. If deformation were such that the strain in an element of soil exceeded the value corresponding to peak strength, then the strength would fall towards the ultimate value. Thus, it is appropriate to use the critical-state strength in analysing stability. However, if a pre-existing slip surface were to be present within the soil, use of the residual strength would be appropriate. (Craig, 2004).

Thus, we have the following three types of slope stability analysis for landslides:

- Limit equilibrium theory; static and pseudo static analysis;
- Un-drained analysis in clay;
- Continuum theory; numerical model.

1.3.1.2. Limit Equilibrium analysis

Limit equilibrium analysis is an important analysis in evaluating the stability of slopes in geotechnical engineering.

a) Assumptions of the limit equilibrium analysis

The limit equilibrium analysis is based on the following assumptions (Nash, 1987);

- The mass of soil is considered perfectly rigid,
- Slope instability occurs for sliding of a mass of soil along a surface,
- The resistance of the soil is fully mobilized along the entire surface (F=constant).
- All limit equilibrium methods use the Mohr-Coulomb expression to determine the shear strength (τ_f) along the sliding surface. The shear stress at which a soil fails in shear is defined as the shear strength of the soil. Mathematically, we have that

  $\tau_f = c' + \sigma' tan\phi'$……………………………………………………..….(E 1.16)

  Where $c' = effective\ cohesion\ of\ the\ soil$

  $\sigma' = effective\ stress\ of\ the\ soil$

  $\phi' = effective\ frictional\ angle\ of\ the\ soil$

The factor of safety (FOS) is determined by using the relation given by equation 1.17

$FOS = \frac{Resistant\ force}{driving\ force}$…….……………………………………......……(E 1.17)

These analyses can either be carried out in un-drained condition (short term) where the shear resistance is evaluated in terms of total stress i.e. $C_u \neq 0$ (un-drained cohesion) and $\phi_u = 0$ (un-drained frictional angle) or drained condition (long term) where the shear resistance is evaluated in terms of effective stress i.e. $c' \neq 0$ and $\phi' \neq 0$

There are various approaches for the determination of the factor of safety as it is shown in Figure1.13.

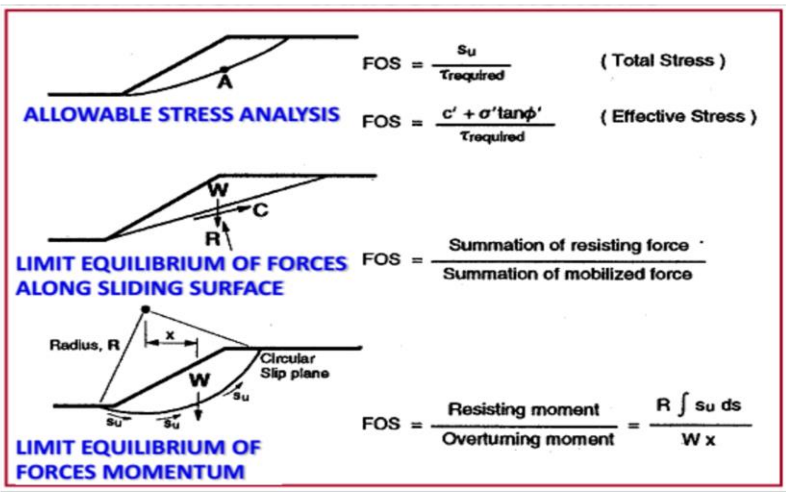


Figure1.13. Various approaches for analyzing factor of safety

b) Types of limit equilibrium analysis

Equilibrium analysis can be done according to two main types of slopes theories which are infinite and finite slopes theory.

i) Infinite slope theory

The infinite slope stability analysis is a 2-D analysis in which the following inputs are required for stability calculation; slope geometry, slope angle in degrees (β) and vertical height of potential failure element(h). The limit equilibrium method is particularly applicable to slope failures where a soil mantle slides downslope over the underlying bedrock or where a permeable root zone or sod layer slides downslope over underlying subsoil layer, which is much less permeable and may temporary induce a perched water table.

It is assumed that the potential failure surface is parallel to the surface of the slope and is at a depth that is small compared with the length of the slope. The slope can then be considered as being of infinite length, with end effects being ignored. Consider that the slope is inclined at angle to the horizontal and the depth of the failure plane is z, (Figure 1.14). The water table is taken to be parallel to the slope at a height of $m \times z$ $(0 < m < 1)$ above the failure plane. Steady seepage is assumed to be taking place in a direction parallel to the slope as shown in Figure 1.14. The forces on the sides of any vertical slice are equal and opposite, and the stress conditions are the same at every point on the failure plane. In terms of effective stress, the shear strength of the soil along the failure plane (using the critical-state strength) is

$\tau_f = (\sigma - u)tan\phi'$………………………………………………………...………(E 1.18)

and the factor of safety is $FOS = \frac{\tau_f}{\tau}$

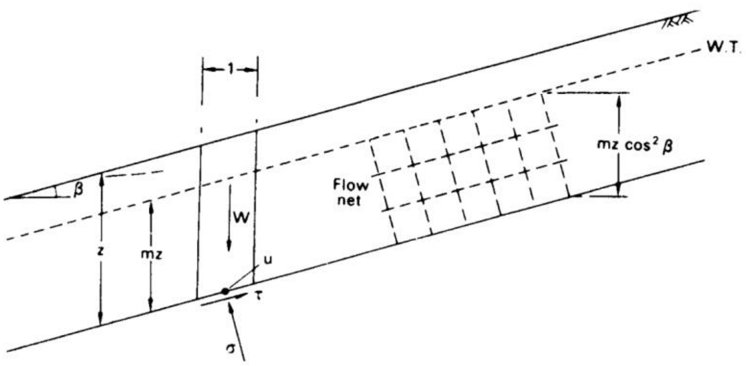


Figure1.14. Plane translational slip for infinite slope (Craig, 2004)

Resolving all the forces acting on the slice, we have

The expressions for σ, τ and u are

$\sigma = \big((1 - m)\gamma + m\gamma_{sat}\big)zcos^2\beta$…………………………………………............……(E 1.19)

$\tau = \big((1-m)\gamma + m\gamma_{sat}\big)z sin\beta cos\beta$……………………………………..……..…..(E 1.20)

$u = mz\gamma_w \cos^2\beta$…………………………………………………………......…(E 1.21)

Therefore;

$FOS = \frac{\tau_f}{\tau} = \frac{(\sigma-u)tan\phi'}{((1-m)\gamma+m\gamma_{sat})zsin\beta cos\beta} = \frac{\big(((1-m)\gamma+m\gamma_{sat})zcos^2\beta - mz\gamma_w cos^2\beta\big)tan\phi'}{((1-m)\gamma+m\gamma_{sat})zsin\beta cos\beta}$…...…(E 1.22)

So generally, for infinite slope,

$FOS = \frac{\gamma tan\phi'}{((1-m)\gamma+m\gamma_{sat})tan\beta}$…………………………………………………….....(E 1.23)

- In dry conditions ($m = 0$)

$FOS = \frac{tan\phi'}{tan\beta}$…………………………………….……………...………....(E 1.24)

- In fully saturated conditions ($m = 1$)

$FOS = \frac{\gamma tan\phi'}{\gamma_{sat} tan\beta}$……………………………………………………..............(E 1.25)

For a total stress analysis the shear strength parameter c_u is used ($\phi_u = 0$) and the value of u is zero.

If the saturated unit weight is twice the specific unit weight of water, the factor of safety in dry condition will be twice the factor of safety in wet condition. Thus, a slope is more stable when dry than when it is wet.

ii) Method of slices (for limited slopes)

In this method the potential failure surface, in section, is again assumed to be a circular arc with centre O and radius r. On the soil mass (ABCD) on Figure 1.15, a trial failure surface (AC) is divided by vertical planes into a series of slices of width b, (Figure 1.15). The base of each slice is assumed to be a straight line. For any slice the inclination of the base to the horizontal is and the height, measured on the centreline, is h. The analysis is based on the use of a lumped factor of safety(FOS), defined as the ratio of the available shear strength (τ_f) to the shear strength (τ_m) which must be mobilized to maintain a condition of limiting equilibrium, i.e

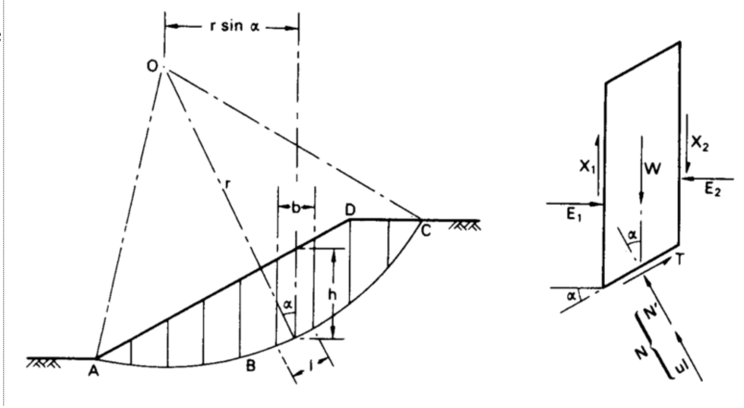


Figure1.15. The method of slice (Craig, 2004)

$FOS = \frac{\tau_f}{\tau_m}$……………………………………………………………………..…..(E 1.26)

The factor of safety is taken to be the same for each slice, implying that there must be mutual support between slices, i.e. forces must act between the slices. The forces (per unit dimension normal to the section) acting on a slice are:

- The total weight of the slice, $W = \gamma bh$ (γ_{sat} where appropriate).
- The total normal force on the base, N (equal to σl). In general this force has two components, the effective normal force N' (equal to $\sigma' l$) and the boundary water force U (equal to ul), where u is the pore water pressure at the centre of the base and l the length of the base.
- The shear force on the base, $T = \tau_m l$.
- The total normal forces on the sides, E_1 and E_2.
- The shear forces on the sides, X_1 and X_2. Any external forces must also be included in the analysis. The problem is statically indeterminate and in order to obtain a solution assumptions must be made regarding the interslice forces E and X; in general the resulting solution for factor of safety is not exact. Considering moments about O, the sum of the moments of the shear forces T on the failure arc AC must equal the moment of the weight of the soil mass ABCD. For any slice the lever arm of W is $r\ sin\alpha$, therefore

$\sum Tr = \sum Wrsin\alpha$…………………………………………………………..……..(E 1.27)

Now

$T = \tau_m l = \frac{\tau_f}{FOS}$…………………………………………………….………….……(E 1.28)

$\therefore \sum \frac{\tau_f}{FOS} l = \sum Wsin\alpha \Longrightarrow FOS = \frac{\sum \tau_f l}{\sum Wrsin\alpha}$……………………………...……...........(E 1.29)

For effective stress analysis (In terms of tangent parameter c' and ϕ':

$FOS = \frac{\sum(c' + \sigma' tan\phi')l}{\sum Wsin\alpha}$ …………………………………………...……………………(E 1.30)

Or

$FOS = \frac{c' L_a + tan\phi' \sum N'}{\sum Wsin\alpha}$…………………………………………………...……...…….(E 1.31)

Where L_a is the arc length AC. Equation2.8 is exact but approximations are introduced in determining the forces N'. For a given failure arc the value of FOS will depend on the way in which the forces N' are estimated. However, the critical-state strength is normally appropriate in the analysis of slope stability,$i.e\ c' = 0$ therefore the factor of safety is given by

$$FOS = \frac{tan\phi' \sum N\prime}{\sum Wsin\alpha} \quad \text{(E 1.32)}$$

The following are some solutions proposed by different authors for the analysis of slope stability by the method of slices.

ii.1) The Fellenius (or Swedish) solution

In this solution it is assumed that for each slice the resultant of the interslice forces is zero. The solution involves resolving the forces on each slice normal to the base, i.e.

$$N' = Wcos\alpha - ul$$

Hence the factor of safety in terms of effective stress is given by

$$FOS = \frac{c'L_a + tan\phi' \sum(Wcos\alpha - ul)}{\sum Wsin\alpha} \quad \text{(E 1.33)}$$

The components $Wcos\alpha$ and $Wsin\alpha$ can be determined graphically for each slice. Alternatively, the value of α can be measured or calculated. Again, a series of trial failure surfaces must be chosen in order to obtain the minimum factor of safety. This solution underestimates the factor of safety: the error, compared with more accurate methods of analysis, is usually within the range 5–20%.

For an analysis in terms of total stress the parameter c_u is used in E 1.33 ($with\ \phi_u = 0$) and the value of u is zero. The factor of safety then becomes

$$FOS = \frac{c_u L_a}{\sum Wsin\alpha} \quad \text{(E 1.34)}$$

As N' does not appear in (E 1.34), an exact value of F is obtained.

The use of the Fellenius method is not now recommended in practice. (Craig, 2004)

ii.2) The Bishop routine solution

In this solution it is assumed that the resultant forces on the sides of the slices are horizontal, i.e

$$X_1 - X_2 = 0 \quad \text{(E 1.35)}$$

For equilibrium the shear force on the base of any slice is

$$T = \frac{1}{FOS}(c'l + N'tan\phi')$$

Resolving forces in the vertical direction gives us:

$$N' = \frac{\left[W - \left(\frac{c'l}{FOS}\right)sin\alpha - ulcosa\right]}{\left[cos\alpha + \frac{tan\phi' sin\alpha}{FOS}\right]} \quad \text{(E 1.36)}$$

It is convenient to substitute

$$l = bcos\alpha$$

From equation 2.17, after some arrangement,

$$FOS = \frac{1}{\sum W sin\alpha} \sum \left[\{c'b + (W - ub)tan\phi'\} \frac{sec\alpha}{1 + \frac{tan\alpha tan\phi'}{F}} \right] \text{……………….…………….(E 1.37)}$$

Bishop also showed how non-zero values of the resultant forces $(X_1 - X_2)$ could be introduced into the analysis but this refinement has only a marginal effect on the factor of safety.

The pore water pressure can be related to the total 'fill pressure' at any point by means of the dimensionless pore pressure ratio, defined as

$$r_u = \frac{u}{\sigma_v} = \frac{u}{\gamma h} \ (\gamma_{sat} \ where \ appropriate). \text{……………………...............………..…..(E 1.38)}$$

For any slice $r_u = \frac{u}{W/b}$

Hence Equation 1 can be written as

$$FOS = \frac{1}{\sum W sin\alpha} \sum \left[\{c'b + W(1 - r_u)tan\phi'\} \frac{sec\alpha}{1 + \frac{tan\alpha tan\phi'}{FOS}} \right] \text{…………………………….…….(E 1.39)}$$

As the factor of safety occurs on both sides of E 1.39, a process of successive approximation must be used to obtain a solution but convergence is rapid.

Due to the repetitive nature of the calculations and the need to select an adequate number of trial failure surfaces, the method of slices is particularly suitable for solution by computer. More complex slope geometry and different soil strata can be introduced.

The factor of safety determined by this method is an underestimate but the error is unlikely to exceed 7% and in most cases is less than 2%.

Spencer proposed a method of analysis in which the resultant interslice forces are parallel and in which both force and moment equilibrium are satisfied. Spencer showed that the accuracy of the Bishop routine method, in which only moment equilibrium is satisfied, is due to the insensitivity of the moment equation to the slope of the interslice forces.

1.3.1.3. Undrained analysis in clay

In the case of cohesive layer located above a rigid material, the possible position of the sliding surface is limited below. Taylor have solved this case furnishing a non-dimensional abacus for the current use (Figure 1.16).

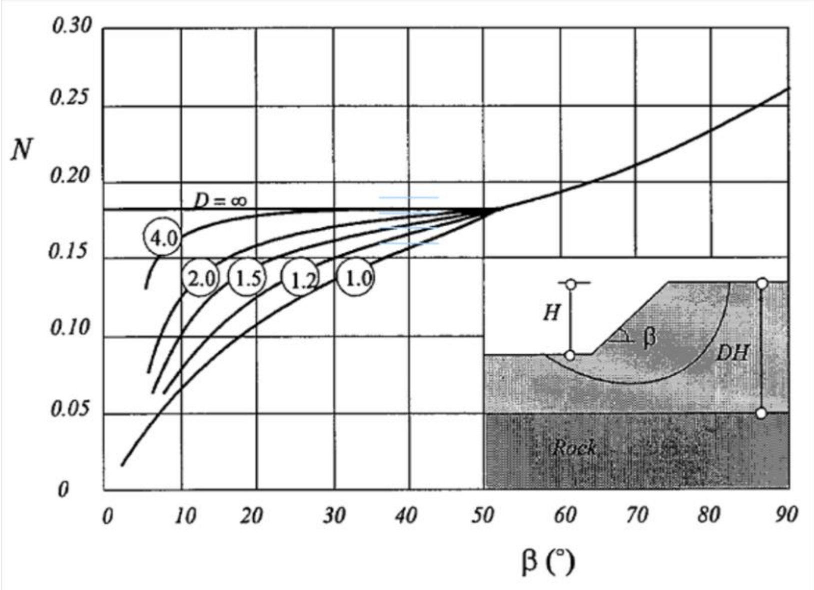


Figure1.16. Taylor's stability coefficients for ϕ_u=0. (Boston society of Civil Engineers)

In the y-axis, there is a factor N given as:

$N = \frac{Cu}{\gamma H.FOS}$ and this is a function of the slope angle,β and the ratio of $D = \frac{Hb}{H}$ with Hb the depth of the rigid base. In condition of limit equilibrium, $FOS = 1$.

1.3.1.4. Numerical methods of analysis

Numerical modellings based on either the LEM or FEM provide an approximate solution to problems which cannot be solved by conventional methods, e.g. complex geometry, material anisotropy, nonlinear behaviour, in situ stresses. Numerical analysis allows for material deformation and failures, modelling of pore water pressures, creep deformation, dynamic loading and assessing effects of parameter variations and so on. However, numerical modelling is restricted by some limitations. For instance, input parameters are not usually measured and availability of these data is generally poor. Numerical methods used for slope stability analysis can be divided into three main groups: continuum; discontinuum and hybrid modelling. Here, we will discuss only the continuum modelling which is the most used model.

Modelling of the continuum is suitable for the analysis of soil slopes, massive intact rock or heavily jointed rock masses. This approach consists of the finite-difference and finite element methods that discretized the whole mass to finite number of elements with the help of generated mesh (Figure 1.17).

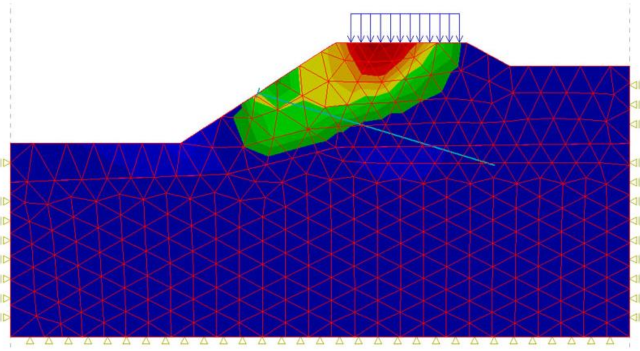


Figure1.17. Finite element mesh

In finite difference method (FDM) differential equilibrium equations (i.e. strain displacement and stress-strain relations) are solved. FEM uses the approximations to the connectivity of elements, continuity of displacement and stresses between elements. Most of the numerical codes allows modelling of discrete fractures e.g. bedding planes, faults. Several constitutive models are usually available e.g. elasticity, elasto-plasticity, strain softening, elasto-viscoplasticity and so on.

Some of the software that can be used consist of the following:

- PLAXIS 2D and 3D- general purpose software for geotechnical applications including slope stability. Based on the Finite Element approach
- OptumG2(2014)-general purpose software for geotechnical applications including slope stability. Based on both the Limit Equilibrium and Finite Element approach.
- GEOSTUDIO- general purpose software for geotechnical applications including slope stability. Based on both the Limit Equilibrium and Finite Element approach.
- Limit state GEO (2008)-general purpose software for geotechnical applications based on discontinuity layout optimization for plane strain problems including slope stability. Based on the Limit Equilibrium approach.
- GEO5 slope stability (1989)-program is used to perform slope stability analysis of embankments, road cuts, anchored retaining structures and MSE walls. Based on the Limit Equilibrium approach.

1.3.2. Ground improvement

The special nature of soft soil deposits is arguably the most interesting soil to work with from the geotechnical point of view. Soft soils are fairly widespread all over the world and they are mostly found in important cities. There are two main problems encountered when undertaking civil constructions on soft soil deposits; excessive settlement and possible instability. Due to large void ratio and inherent compressibility of such clays, consolidation and displacements can be noticeable under construction loads and equally over a lengthy period after implementation of the structure. Low shear strength is particularly hazardous when constructing large embankment on soft clay base, facilitating potential circular or sliding failure planes. Hence, the need of ground improvement schemes is very necessary. With time and as a function of the difficulties faced, many techniques have been developed in other to allow safe construction on soft soil.

1.3.2.1. Some ground improvement techniques

When it comes to soil improvement techniques, there is a wide variety of methods which allow constructors to improve the soil structure below or around the projected construction. It would be impossible to describe all the available methods of soil improvement, but in Figure 1.18 is present a summary that could be strongly useful. The most appropriate construction method to be used in each project is associated with factors such as geotechnical characteristics of deposits, use of the area, construction deadlines and costs involved. Figure 1.18 presents some construction methods of embankments on soft soils (Lerouil, 1997). Some methods contemplate settlement control and others stability control, but most methods contemplate both issues. In the case of very soft soils, it is common to use geosynthetic reinforcement associated with most of the alternatives presented in Figure 1.18. Time constraints may render inadequate techniques such as conventional embankments (Figure 1.18A, B, C, D, M) or embankments over vertical drains (Figure 1.18K, L), favouring embankments on pile-like elements (Figure 1.18F, G, H) or lightweight fills (Figure 1.18E), which, however, may have higher costs. Removal of soft soil can be used when the layer is not very thick (Figure 1.18I, J) and the transport distances are not considerable. In urban areas, it is difficult to find areas for the disposal of excavated material, considering the environmental issue associated with this disposal. Space constraints can also prevent the use of berms (Figure 1.18B), particularly in the case of urban areas. The geometry of the embankments and the geotechnical characteristics are highly variable factors and the construction methodology must be analysed case by case.

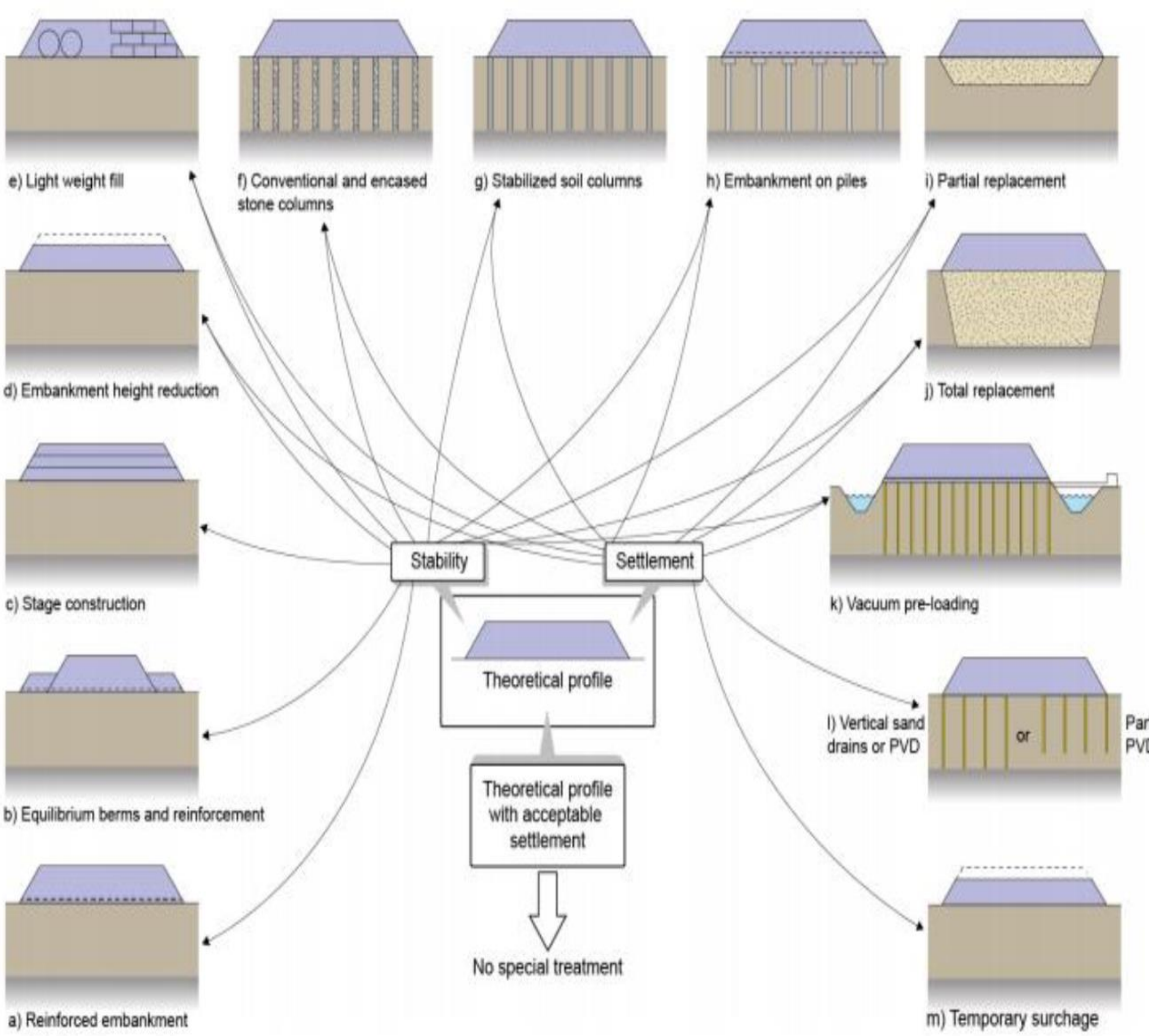


Figure1.18. Improvement techniques on soft soil (adapted from Leroueil, 1997)

Over the techniques shown in figure 1.18, the Stone columns are one of the most economical solutions for soil improvement, since it mainly uses the available material.

For this research, the characteristics of the soft foundation soil will be improved by the used of stone columns while the stability of the embankment will be assured on one hand the use of a geosynthetics (geotextile) at its base and on the other hand by the stable foundation soil.

1.3.2.2. Stone Columns

Stone column supported embankments are time dependent foundation technique used to increase the structural stability and to reduce the structural deformations.

a) Description and installation method

Stone column ground improvement involves adding vertical columns of stone into the ground to a depth of at least 4m below the ground surface. A layer of compacted gravel can then be put over the top of the columns, ready for the construction of new embankment. Stone columns help to limit the amount and consequences of future liquefaction by:

- Densifying the soil through vibration and introducing stone into the soil
- Reinforcing the soil creating a stiff composite soil mass.

By achieving this, the non-liquefying soil crust is thickened and stiffened to reduce the likelihood of undulations, tilt and uneven ground surface subsidence from liquefaction of the underlying soil layers, therefore reducing damage to the embankment.

In addition, stone columns provide the soil with an increased drainage path to help reduce excess pore water pressure that can lead to liquefaction, so the columns can reduce the consequences of liquefaction when this occur as shown in Figure1.19.

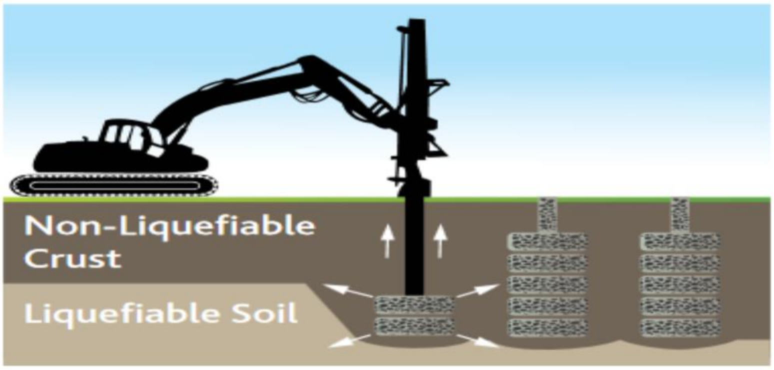


Figure1.19. Stone columns effect on liquefiable soil

They are realized with coarse materials (sand or gravel with $5mm < D < 150mm$) put in site as shown in Figure1.20 and Figure1.21 by:

- Vibroflotting (deep vibrator)
- Casing pile installed by vibration
- Casing pile installed by a screw

Figure1.20. Stone column realisation

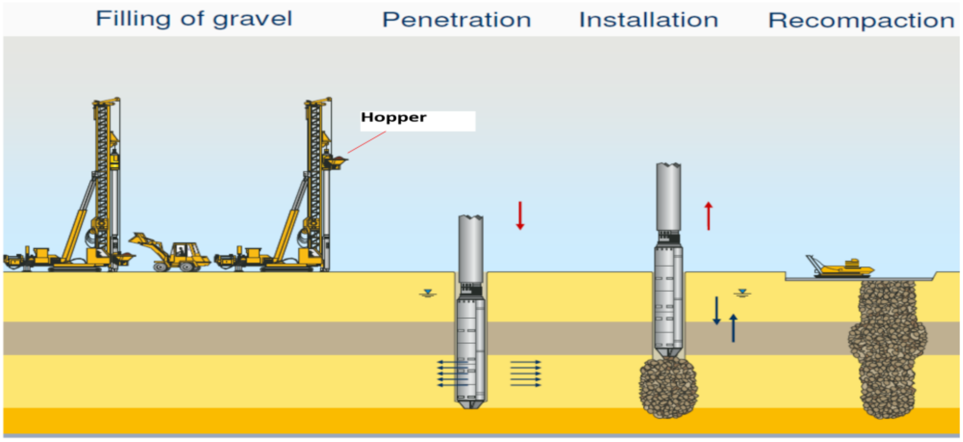


Figure1.21. Stone column realisation stages (TRH, 1986)

They are used to :

- Reduce the consolidation time (are like sand drains)
- Reduce the settlement entity
- Increase the overall resistance of the system (bearing capacity)
- Reduce Liquefaction potentials
- Enhance the stability of embankments and natural slopes

Used in soft soil generally with $c_u < 50\ kPa$ (make care in sensitivity soil).

Typical sizes:

$D = 0.7 - 1.2\ m, diameter$

$L\ = 5 - 20\ m, length$

$i = 1.5 - 3.5\ m., spacing$

The material used for stone columns are of the following characteristics;

- Diameter in the range 5-150 mm
- 60-70% in weight in the range 40-80 mm
- Every type of rounded degree
- no crushable material
- eventually addition of cement or other materials (secondary materials)

If the columns have a drainage function also we have to add sand in order to avoid the fine erosion from soils with subsequent filling of the column pores.

The grain-size distribution has to respect the Terzaghi criteria for filters (1922):- permeability criterion: $D_{15f}\ >\ 4 \cdot D_{15b}$

- retention criterion: $D_{15f}\ <\ 4 \cdot D_{85b}$

Drain blanket:

% Passing to #200 ≤5%

% sand ≤50%

$D_{max} \leq 150\ mm$

$E \geq 60\ MPa$ for $p = 100 - 300\ kPa$ in the load test with the 60 cm diameter plate.

b) Analysis and calculation method

Stone columns effectiveness considerably result from the use of an adequate analysis and calculation method.

i) Analysis

Most studies in the field of ground improvement by stone columns have focused on unit cell and full-scale analyses. The unit cell pattern is convenient for single stone column, and full-scale analyses are appropriate for group stone columns. The unit cell conception has been widely investigated by many researchers. According to research of Ballam & Booker (1981), there are three various patterns of preparation of single stone column in unit cell condition as follows:

- Square pattern,
- Triangular pattern,
- Hexagonal pattern.

Parameters

The most important parameter in a stone column treatment is the area replacement factor (a_s), which represents the area of soft soil replaced or displaced by the stone columns (Jorge Castro, 2017) given by;

$a_s = \frac{A_s}{A}$……………………………………………………………..……….(E 1.40)

Where $a_s = Area\ ratio, A_s = Total\ area\ of\ a\ unit\ cell, A = Total\ area\ of\ columns$

Equation 1.41 gives the relationship of total area of column with respect to diameter and spacing of columns.

$A_s = C_1 \times \left(\frac{D}{S}\right)^2$ …………………………………………………………..…… (E 1.41)

Where $D = Column\ Diameter, S = Column\ spacing,$

$C_1 = constant\ corresponding\ columns\ pattern$

Figure 1.22 provides the unit cell arrangement in three patterns.

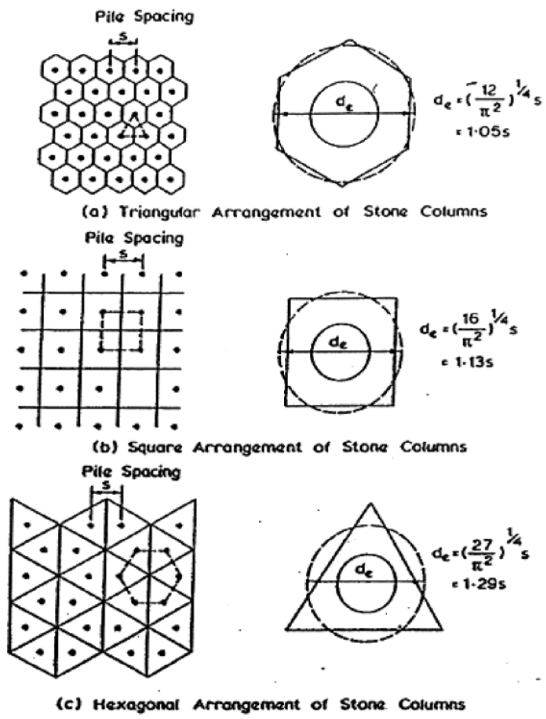


Figure1.22. Various patterns of unit cell: (a) Triangular, (b) Square, and (c) Hexagonal (Weber, 2008)

ii) Calculations method

Generally, the problems in modelling and designing stone columns are;

- The mutual actions at the contact gravel-soil are unknown and there is a compenetrating among the material
- The external load subdivides between columns and soil not proportionally to the area
- The settlements of columns and soil are different
- The installation modifies the initial stress distribution

It is usually assumed that:

- The columns have the tip in a more resistant layer
- The settlement of the ground surface is constant and the vertical strain is equal everywhere (the adsorbed load is proportional to the stiffness)
- The gravel is in plastic state (c indicates the column quantities):

$\sigma'_{hc} = K_{ac}\sigma'_{vc}$……………………………………………………….……………(E 1.42)

Settlement analysis (by De Beer-Van Impe, 1983) defines the Equivalence of area to have a plane strain model in which every column line is transformed in a wall with a thickness d_f.

Being $\alpha = area\ ratio$:

$\alpha = \frac{\pi D}{4ab} = \frac{d_f}{b} \Leftrightarrow d_f = \alpha b$……………………………………….............. (E 1.43)

Where D i Figure 1.22 provides the unit cell idealization in three patterns.

the column diameter and a and b being the longitudinal and transverse interspaces

Figure1.23 shows and illustration of the load distribution on the soil-columns combination.

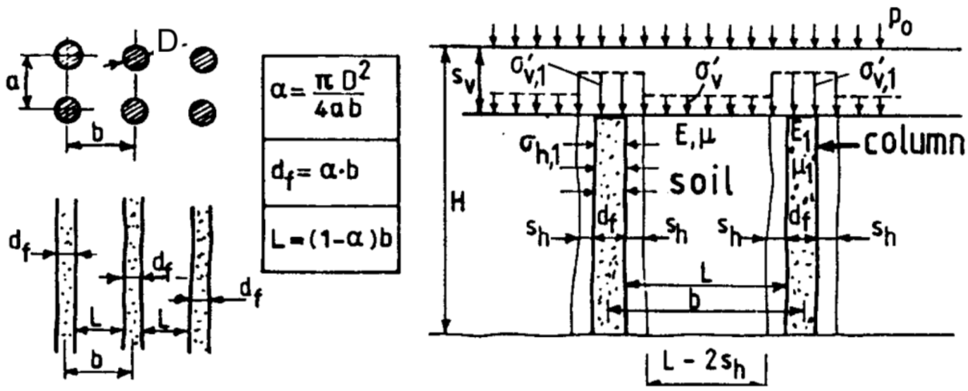


Figure1.23. Stone columns and load distribution (De Beer-Van Impe, '83)

For this analysis, the following are considered;

We neglect Self-weight of soil and the Shear stress in the lateral side of column (horizontal and vertical stresses are principal).

And we assume that the column strain at constant volume and he soil parameters are E_t *and* ν_t.

We can write the following relations (symbol in the previous scheme):

i) Constant column volume (s_v = *vertical settlement*, s_h = *lateral displacement*):

$d_f H = (d_f + 2s_h)(H - s_v) \Rightarrow s_v = \frac{2s_h H}{d_f + 2s_h}$.. (E 1.44)

ii) Elastic strain in plane strain condition of the soil:

$s_v = \frac{H}{2E_t}(1 - \nu_t^2)\left(\sigma'_{ht} - \frac{\nu_t}{1-\nu_t}\sigma'_{\nu_t}\right), \ E_t = \frac{(1-\nu_t)(1-2\nu_t)}{1-\nu_t}E_{ed,t}$.............................. (E 1.45)

iii) Active plastic state for the column:

$\sigma'_{hc} = K_{ac}c = \sigma'_{ht}$.. (E 1.46)

iv) Subdivision of external load:

$p_0 b = (d_f + 2s_h)\sigma'_{vc} + (L - 2s_h)\sigma'_{vt}$.. (E 1.47)

The equations may be transformed in non-dimensional shape and then solved:

i') $\frac{s_v}{H} = f_1\left(\frac{s_h}{\alpha b}\right)$..(E 1.48)

ii') $\frac{\sigma'_{vt}}{p_0} = f_2(\nu_t, \frac{s_h}{\alpha b}, \frac{p_0}{E_t})$...(E 1.49)

iii') $\frac{\sigma'_{ht}}{p_0} = f_3(\nu_t, \frac{s_h}{\alpha b}, \frac{p_0}{E_t})$...(E 1.50) iv') $f_4\left(\frac{\sigma'_{vt}}{p_0}, \frac{\sigma'_{ht}}{p_0}, K_{pc}, \alpha\right) = 0 \Longrightarrow f_5\left(\frac{p_0}{E_t}, \nu_t, K_{pc}, \frac{s_h}{\alpha b}\right) = 0$……………………………….(E 1.51)

Being known p_o, E_t, ν_t, K_{pc} and α, we solve the last equation for attempts and we obtain the ratio $\frac{s_h}{\alpha b}$. This permits to obtain the stress in column and soil and the vertical settlement.

$m = \frac{F_1}{F_{tot}} = \alpha\left(\frac{\sigma'_{vc}}{p_0} + 2\frac{s_h}{\alpha b}\right)$ and $\mathcal{K} = \frac{S_v}{S_{vo}}$…………………………………………..…. (E 1.52)

With m and $\mathcal{K}$ being the "efficiency parameters" where F_1 = vertical load on one column, F_{tot} = *total applied on the competent cell*, S_{vo} = *settlement without columns*, s_v = *settlement with columns.*

Figure 1.24 shows how the efficiency parameters vary with column distribution

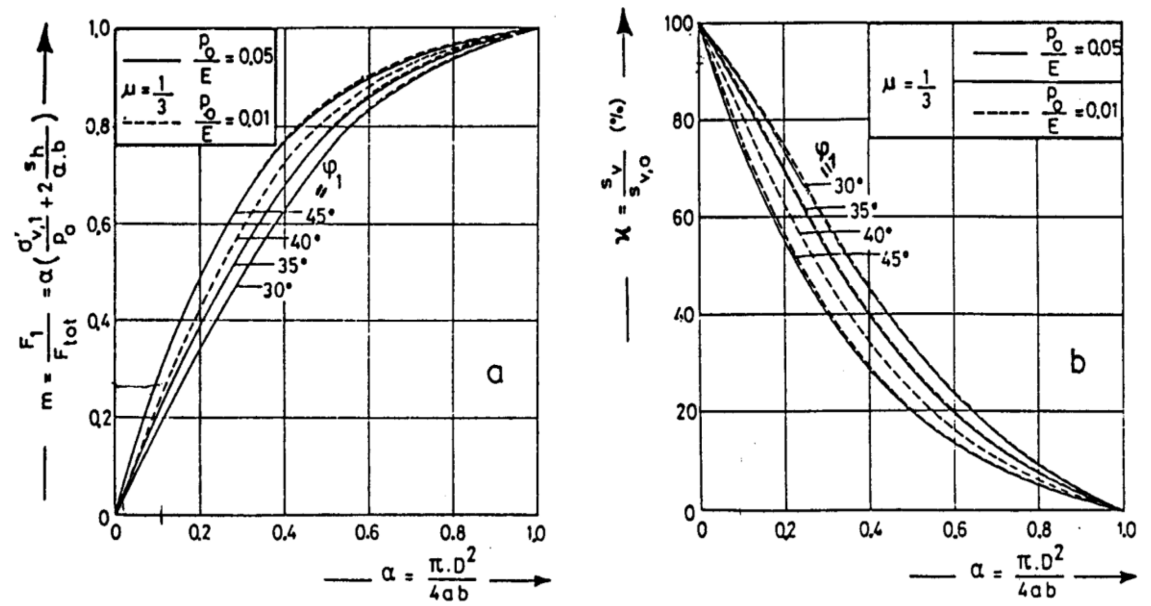


Figure1.24. Efficiency parameters variation with column distribution (De Beer-Van Impe, '83)

The bearing capacity analysis stipulates that the columns could reach failure according to 3 mechanisms as shown in Figure1.25;

(1) Excess lateral expansion due to small lateral confinement
(2) Shear along a inclined sliding plane
(3) Punching failure of the underlying soil

There are some formulae but they are not largely used because the columns are prevalently used for settlement reduction, drainage and reinforcement below embankment.

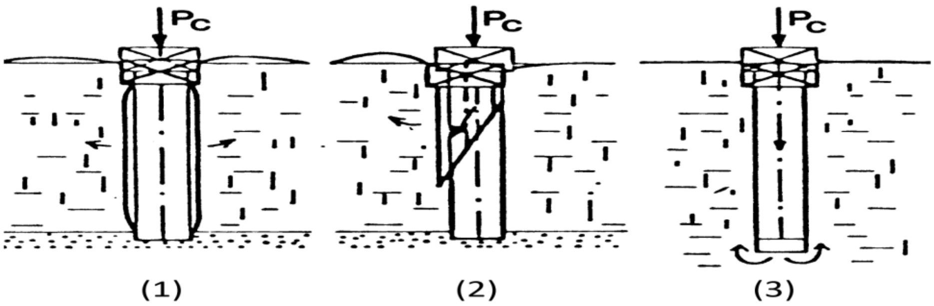


Figure1.25. Bearing capacity analysis of a stone column reinforced soil

The stability analysis can be assessed taking into account the columns in three ways:

- Using plane strain LEM analysis drawing every column lines eventually substituted by the equivalent walls. The LEM code will divide in homogeneous slices;
- Using plane strain LEM analysis but transforming the layer with the columns inside in an equivalent material with shear strength parameters $\phi^{*} and\ c^{*}$:

  $\phi^{*} = \arctan\ (\alpha tan\phi_c')$, $\quad c^{*} = (1-\alpha)c_u$.. (E 1.53)
- Using FEM analysis with column simulated as in (1) or (2).

Figure 1.26 shows parameters affecting the stability of stone column reinforced soil

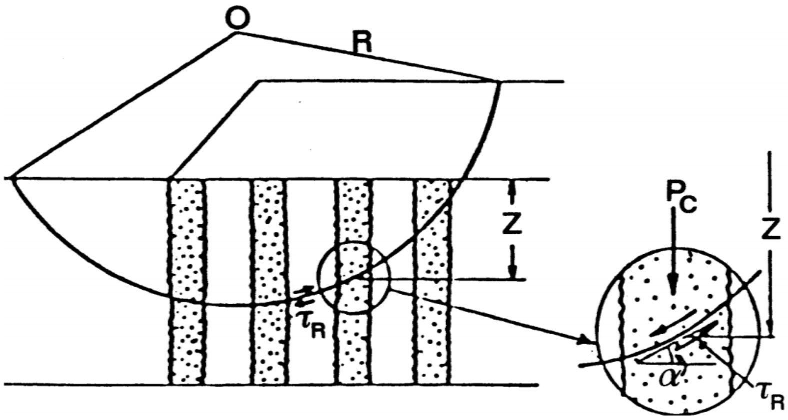


Figure1.26. Stability analysis of a stone column reinforced soil.

An example of analysis on an embankment on soft soil with stone columns was done by Fattah et al. in 2014 and the following observations where made;

- Improvement of the soft clay with stone columns leads to significant effect in the behaviour of the soft clay, both in undrained conditions and during consolidation stages.
- Increasing the stone column's diameter leads to a considerable decrease in the settlement beneath the embankment, especially below the centre line.

- Consolidation analysis showed that the presence of stone columns leads to a decrease in the differential settlement below the embankment. The reduction in settlement continues during the consolidation process.
- The excess pore water pressure increases immediately after construction of each lift of the embankment and then falls. The excess pore water pressure dissipates suddenly when using stone columns. This means that these columns function as **drains**.

c) Method for numerical modelling of Stone columns

The modelling of stone columns is mainly function of the type of analysis or the kind of the finite element analysis that we want to study. Finite element (FE) analysis of the test embankment is performed using both two dimensional axisymmetric and plane strain simulations. The axisymmetric analysis is carried out by modelling half of a stone column accompanied by the surrounding soft soil. The diameter of the unit cell was determined assuming an average center-to-center spacing in the middle area of the test embankment. The plane strain analysis was conducted by simulating the full-scale embankment on stone columns in which the column diameter was transformed into an equivalent wall. The simulation procedure, material properties, and results of the numerical analyses will be described in the following chapter.

1.2.2.2. Geotextile

Geotextiles belong to the family of geosynthetics which are widely used materials in the engineering world and more precisely in Geotechnical Engineering.

a) Description of geotextiles and use

It is a planar, flexible, permeable, polymeric (synthetic or natural) textile material, thickness 1-10mm, which may be nonwoven, knitted or woven, used in contact with soil/rock and/or any other geotechnical material in civil engineering applications. The figure 1.27 shows a variety of geotextiles ranging from woven to non-woven to knitted.

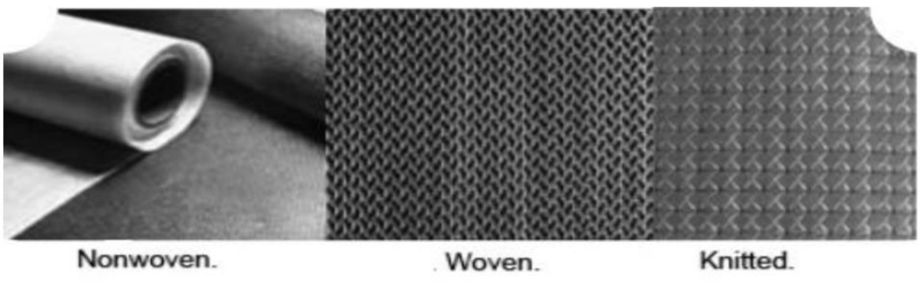


Figure1.27. General Geotextiles (Zornberg et al, 1995)

Among the different geosynthetic products, geotextiles are the ones that present the widest range of properties. They can be used to fulfill all the distinct functions of geosynthetics for many different geotechnical, environmental, and hydraulic applications.

Secondly subgrade stabilization is one of the most common geotextile applications. The geotextile, placed at the interface between the soft subgrade and the granular fill material, prevents the loss of the granular fill into the soft subgrade, thereby maintaining the structural integrity of the fill as shown in Figure1.28.

Figure1.28. Geotextile used in subgrade stabilization. (Source: Geosynthetic international)

Geotextiles used for subgrade stabilization must meet specific mechanical and hydraulic requirements to perform properly. Two external factors govern the mechanical requirements of the geotextile. These are the strength of the soft subgrade beneath the geotextile, and the type of granular fill material placed above the geotextile. The geotextile has to be more robust than the weaker subgrade below, and the larger the stone sizes used in the granular fill material above. The hydraulic properties of the geotextile must ensure adequate movement of groundwater out of the soft subgrade.

b) Composition of Geotextile

The most common types of fibers or filaments used in the manufacture of geotextiles are monofilament, multifilament, staple filament, and slit-film. If fibers are twisted or spun together, they are known as a yarn. Monofilaments are created by extruding the molten polymer through an apparatus containing small-diameter holes. The extruded polymer strings are then cooled and stretched to give the filament increased strength. Staple filaments are also manufactured by extruding the molten polymer; however, the extruded filaments are cut into 25 to 100 mm portions. The staple by knives or lanced air jets. Slit-film filaments have a flat, rectangular cross-section instead of the circular cross-section shown by the monofilament and staple filaments. filaments or fibers may then be spun into longer yarns. Slit-film filaments are manufactured by either extruding or blowing a film of a continuous sheet of polymer and cutting it into filaments (Zornberg, 2007).

c) Manufacture of geotextiles

Geotextiles are manufactured from polymer fibers or filaments that are later formed to develop the final product. Approximately 85% of the geotextiles used today are based on polypropylene resin. An additional 10% are polyester and the remaining 5% is a range of polymers including polyethylene, nylon, and other resins used for specialty purposes. As with all geosynthetics, however, the base resin has various additives, such as for ultraviolet light protection and long-term oxidative stability (Zornberg ,2007).

Woven geotextiles are manufactured using traditional weaving methods and a variety of weave types. Nonwoven geotextiles are manufactured by placing and orienting the filaments or fibers onto a conveyor belt, which are sub-sequent bonded by needle punching or by melt bonding. The needle-punching process consists of pushing numerous barbed needles through the fiber web. The fibers are thus mechanically interlocked into a stable configuration. As the name implies, the heat (or melt) bonding process consists of melting and pressurizing the fibers together (Zornberg ,2007).

Common terminology associated with geotextiles includes machine direction, cross machine direction, and selvage. Machine direction refers to the direction in the plane of fabric in line with the direction of manufacture. Conversely, cross machine direction refers to the direction in the plane of fabric perpendicular to the direction of manufacture. The selvage is the finished area on the sides of the geotextile width that prevents the yarns from unraveling. Adjacent rolls of geotextiles are seamed in the field by either overlapping or sewing. Sewing is generally the case for geotextiles used as filters in landfill applications but may be waived for geotextiles used in separation. Heat bonding may also be used for joining geotextiles used in filtration and separation applications.

Conclusion

This chapter presented in its first part, soft soils, their origins, their typology and their properties followed by the notion of embankments with emphasis laid on the definition and use in the engineering world and finally their behaviour when constructed on soft soils including some failure mechanisms. This part reveals that embankments constructed on soft soils should be treated with attention on its long term stability prior to the relatively low quality properties of soft soils. The second part started with an overview on stability, different solutions for calculation of the factors of safety according to different researchers, the notion of soil improvement techniques, different methods for soil improvement with particularly attention paid on the stone column method due to its efficiency and multipurpose were studied. This solution was associated with the use of a geosynthetic (geotextile) for the reinforcement and seperation purposes at the embankment base. The following chapter will be focused properly on how the predicted results of the technique chosen can be attained.

Chapter 2 : METHODOLOGY

Introduction

As seen in the previous chapter, analysing the stability of embankments on soft soils requires a good knowledge of the type of material in place (its properties), the geometry of the problem (embankment and foundation soil), the modes of failure and possible causes, and the use of well-established numerical and analytical methods in order to verify the stability and to propose solutions in case the embankment is unstable. As far as this chapter is concerned, it will focus on the working methodology, i.e. the set of steps or procedures to be followed in order to achieve the main objective of this research which is to access the stability of a high embankment constructed on a soft soil on the Bangangté -Foumbot - Bamendjing - Galim road section with the stabilization method chosen. For this reason, the procedure starts with a general recognition of the study area followed by an observation visit to the site. Subsequently, a data collection will be necessary for the modelling of the embankment. The analytical method of stone column design and analysis will then be presented, followed by the presentation of the numerical analysis software, different design models and necessary parameters for the design.

2.1. Site recognition

With the aim to correctly find a solution to our thesis problem, many information have been obtained about the project location and all the characteristics (physical and socio-economic) of the villages concerned. The general recognition of the site is done through a documentary research. The objective is to provide on a map the geographical location of the project area, its relief, the climatic conditions (temperatures and precipitations of the area which are factors influencing the consolidation process), the geology, the seismicity, the hydrology, the vegetation, the fauna and the flora but also the socio-economic parameters and activities carried in the region.

2.2. Site visit

After the documentary research, the site visit has been done by observing the study area. the observation of the site consists at taking pictures of it and asking questions, in order to have a better and wide set of information necessary to properly understand the problem. In fact, with the embankment already in construction phase, we have consulted and used available project reports, as well as documented or academic reports related to our study area for more information.

2.3. Data collection

The collection of geotechnical and geometric characteristics of selected samples on a road section from **PK 12+650 to PK 12+950** (PK stands for "*Point Kilométrique*"-road length) will enable a two-dimensional and possibly a three-dimensional modelling and analysis using analytical and numerical methods later described in this chapter.

2.2.1. Geometrical data

The geometrical data were also given by the same company. These data present the longitudinal profile of the embankment area as well as the cross-sections of the typical profiles.

2.2.2. Geotechnical data

Part of the geotechnical data of the problem were acquired from the laboratory of the company in charge of the construction works of the project entitled "*Route de desenclavement du basin agricol de l'Ouest Lot 2 Bangangté Foumbot-Bamendjing-Galim*". These data present the physical, mechanical and compressibility characteristics of the material forming the foundation and embankment soil.

2.4. Analytical design of Stone columns

The analytical method used to design our stone columns is that proposed by De Beer-Van Impe in 1983 as described in the previous chapter. The analysis is done is the unit cell analysis.

This method consists in verifying the stresses and settlements in the column and surrounding soil.

The geotechnical data needed for the design are ;

- Wet and dry densities (γdry / γ')
- Friction angle, drained and undrained shear strength ($\phi'/c'/Cu$)
- Eodometric modulus and poisson's ratio ($Eoed$ / ν)
- Permeability (K)

With the method, we consider a rectangular arrangement of the columns and we fine an equivalence of area to have a plane strain model in which every column line is transformed in a wall with a thickness d_f. Details about the column pattern is described in Figure 1.23

With $\alpha = area\, ratio$:

$$\alpha = \frac{\pi D^2}{4ab} = \frac{d_f}{b} \Leftrightarrow d_f = \alpha b \quad \text{(E 2.1)}$$

Where D is the column diameter and a and b being the longitudinal and transverse interspaces For this analysis, the following are considered;

We neglect Self-weight of soil and the Shear stress in the lateral side of column (horizontal and vertical stresses are principal).

We assume that the column strains at constant volume, the installation method is displacement and the soil parameters are E_t and ν_t.

We can write the following relations (symbol in the previous scheme):

i) Constant column volume ($s_v = vertical\ settlement, s_h = lateral\ displacement$):

$$d_f H = (d_f + 2s_h)(H - s_v) \Rightarrow s_v = \frac{2s_h H}{d_f + 2s_h} \quad \text{(E 2.2)}$$

ii) Elastic strain in plane strain condition of the soil:

$$s_v = \frac{H}{2E_t}(1 - \nu_t^2)\left(\sigma'_{ht} - \frac{\nu_t}{1-\nu_t}\sigma'_{\nu_t}\right), \quad E_t = \frac{(1+\nu_t)(1-2\nu_t)}{1-\nu_t} E_{eod,t} \quad \text{(E 2.3)}$$

iii) Active plastic state for the column:

$$\sigma'_{hc} = K_{ac}\sigma'_{vc} = \sigma'_{ht} \quad \text{(E 2.4)}$$

$$\sigma'_{ht} = K_{at}\sigma'_{vt} \quad \text{(E 2.5)}$$

iv) Subdivision of external load:

$$p_0 b = (d_f + 2s_h)\sigma'_{vc} + (L - 2s_h)\sigma'_{vt} \quad \text{(E 2.6)}$$

Knowing p_o, E_t, ν_t, K_{ac}, a, b, α, and H we solve the previous equations to obtain the stresses and vertical settlement in column and soil.

2.5. Numerical analysis in Plaxis 2D

In the framework of our work, numerical analysis will be carried out namely to simulate and constate the initial state of the problem and then show the effectiveness of our proposed solutions.

The one-dimensional consolidation equation can be solved numerically by the Finite Element Method (FEM). The method has the advantage that any pattern of initial excess pore water pressure can be adopted and it is possible to consider problems in which the load is applied gradually over a period of time. The errors associated with the method are negligible and the solution is easily programmed for the computer (Craig, 2004).

The analytical analysis method using FEM is well implemented in the PLAXIS software and the presentation of the different moduli of the software and how they are used in the analysis will be done in the next part.

2.5.1. Input program

In the Input program of Plaxis the geometry is given by entering different soil layers, structural parts, and external loads etc. A choice between various available material models: Linear model, Mohr-Coulomb, Hardening Soil, Hardening Soil Model with Small-Strain Stiffness, Soft Soil and Soft Soil Creep, is made at the input for each material. The material is given relevant material properties, such as stiffness and density, which are assigned to elements together with appropriate boundary conditions. Also, the model in whole is assigned boundary conditions. When the model is complete, a mesh is generated, and initial stresses and pore water pressures are initiated before moving to the Calculation program.

2.5.1.1. Material properties

Soil properties and material properties are stored in material data sets. There are four different types of material sets amongst which we have data sets for soil and interfaces, plates, geogrids and anchors. More explicitly, there are different material models proposed by plaxis for modelling the soil namely the mohr-coulomb model, the linear-elastic model the hardening soil model, the soft soil model, the soft soil creep model, and the jointed rock model. In our study, focus will be made on the model which can take into account many characteristics of soft soil and embankment material. Two models will be presented in this part: Mohr coulomb model (MC) and soft soil creep model (SSC).

a) Mohr-Coulomb model (MC)

The elastic-plastic Mohr-Coulomb model represent a first order approximation of the soil or rock behaviour as for our embankment, our substitution material. This model involves five inputs parameters which are: Young's modulus (E) and Poisson's ratio (ν) for soil elasticity, Friction angle (φ) and Cohesion (c) for soil plasticity, and dilatancy angle (ψ).

b) Soft soil creep model (SSC)

Generally, all soils exhibit some creep and the primary compression is thus followed by a certain amount of secondary compression. The high degree of compressibility, creep and the secondary compression are dominant in soft soils such as normally consolidated clay, silt and peat as in our case. These are best demonstrated by oedometer test data. Therefore, plaxis implemented a model under the name of Soft Soil Creep which is a relatively new model and it has been developed for application of settlement problems, embankment, etc. The proper initial soil conditions are essential when using Soft Soil Creep Model. It also includes data on the preconsolidation stress to take in consideration the effect of over-consolidation. Some basic characteristics of the Soft Soil Creep model are;

- Stress dependent stiffness (Logarithmic compression behaviour)
- Distinction between primary loading and unloading-reloading
- Secondary (time dependent) compression
- Memory of pre-consolidation
- Failure behaviour according to Mohr Coulomb criterion

The Soft Soil Creep Model requires the following main parameters:

- Failure parameters as in the Mohr-Coulomb model: Cohesion(c); Friction angle(φ);

Dilatancy angle (*ψ)*

- Parameters of the Soft Soil Creep model: Modified compression index (λ^*); Modified swelling index (κ^*); Modified secondary compression index (μ^*).

These parameters can be obtained both from an isotropic compression test and an oedometer test. When plotting logarithm of the stress as a function of strain, the plot can be approximated by two straight lines, as shown in Figure2.3. The slope of the normal consolidation line gives the modified compression index, λ^* and the slope of the unloading or swelling line can be used to compute the modified swelling index, κ^*. There is a difference between the modified indices λ^* and κ^* and the original Cam- Clay parameters λ and k. The later parameters are defined in terms of the void ratio e instead of the volumetric strain εv. The parameter μ^* can be obtained by measuring volumetric strain on the long term and plotting it against the logarithm of time, as shown in Figure2.3.

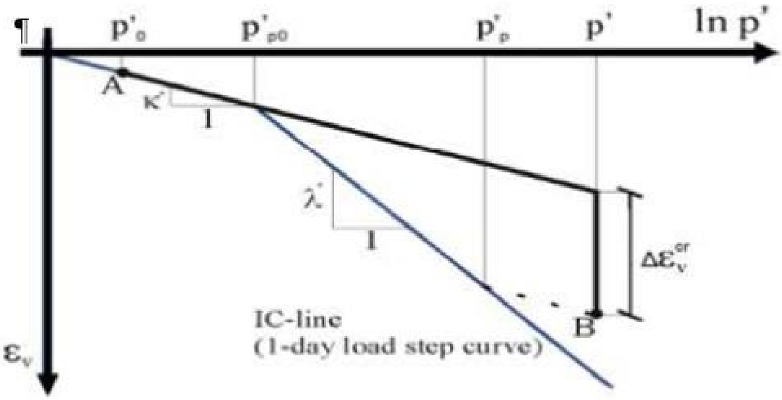


Figure2.3. Idealized stress-strain curve from oedometer test with division of stress increments into elastic and creep component (Plaxis Manual 2008)

Relationship to Cam-Clay parameters:

Relationship to internationally normalized parameters:

$\lambda^* = \frac{C_c}{2.3\times(1+e)}, \ \kappa^* \approx \frac{2C_s}{2.3\times(1+e)}, \ \mu^* = \frac{C_\alpha}{2.3\times(1+e)}$ ……………………………………….(E 2.7)

Where;

$\boldsymbol{C_c}$ is the compression index,

The compression index (C_c) is the slope of the straight line on the e-log σ' plot, and is dimensionless (Figure 1.6). For any two points on the linear portion of the plot the equation for Cc may be written as:

$$C_c = -\frac{e_1 - e_0}{\log(\sigma_1'/\sigma_0')} \quad \ldots\ldots (E\ 2.8)$$

C_r is the recompression index,

The equation of recompression Index Cr (Figure 1.6) is given by the following relation:

$$C_r = -\frac{e_2 - e_1}{log(\sigma_2'/\sigma_1')} \quad \ldots\ldots (E\ 2.9)$$

$\boldsymbol{C_\alpha}$ is the secondary compression index.

This parameter is obtained by the creep curve. It is the plot of void ratio versus the logarithm of time from our experimental data. The secondary compression Index Cα may be written as:

$$C_\alpha = -\frac{e_1 - e_p}{lo\,g\left(\frac{t}{t_p}\right)} \quad \ldots\ldots (E\ 2.10)$$

Where (t_p, e_p) is the coordinate at the intersection of the tangents of the primary consolidation and secondary compression parts of the void ratio versus logarithm of time curve and (t, e_1) is the coordinate of any point on the secondary compression curve.

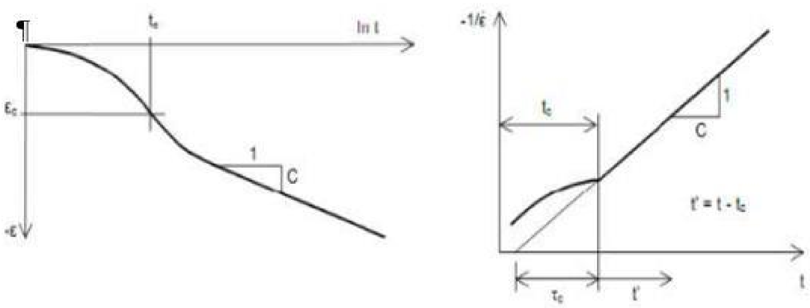


Figure2.4. Consolidation and creep behaviour in standard oedometer test (Plaxis Manual 2008).

2.5.1.2. Material behaviours

In principle, all model parameters in Plaxis are meant to represent the effective soil response, i.e. the relation between stresses and strains of the soil skeleton. An important feature of the soil is the presence of pore water. Pore pressure significantly influences the soil response. To enable incorporation of the water-skeleton interaction in the soil response Plaxis offers for each model a choice of three types of behaviour;

- Drained behavior *;* Using this setting no excess pore water pressure is generated. This is clearly the case for dry soils and also for full drainage due to high permeability as in sand or a low rate of loading. This option may also be used to simulate long-term soil behaviour without the need to model the precise history of the undrained loading and consolidation.
- Undrained behavior ; This setting is used for a full development of excess pore water pressure. This occurs when a soil has low permeability as in clays or under a high rate of loading. The undrained behavior is usually followed by consolidation in loading phases.
- Non-porous behavior ; Using this option neither initial nor excess pore-water pressure is taken into account. Application for this option may be found in modelling of concrete and rock or structural behavior. Non-porous behavior is often used in combination with linear elastic model. In this study, we are study both the drained and undrained behavior to well establish the behavior of stone columns.

2.5.1.3. Model generation

As mentioned above, two types of material elements have been used to model the reinforced soft soil with stone columns.

Soil elements have been used to simulate the stone column material; the surrounding soft soil, the blanket layer and the embankment fill in the whole embankment modelling. The soft foundation soil has been modelled (in undrained and consolidation, and drained conditions) by the Soft Soil Creep Model. The blanket layer and the embankment fill have been modelled by the Mohr Coulomb Model in drained conditions. The stone column material have been modelled also by using the Mohr Coulomb Model in drained conditions. J. PIVARČ (2011) using Plaxis

a) Geometry, loading and boundary conditions

When setting up the geometry of the models, each model was divided into ten clusters. The first four and the fifth cluster represent the foundation soil and the stone column. While the sixth and the remaining clusters represent the blanket fill layer and the embankment fill. However, the three last clusters were divided into sub-clusters to represent the stages of the embankment construction. The geometry of each model was controlled by the following conditions;

- Loading entire area of the stone column and the surrounding soft soil by applying embankment loads in intervals.
- The volume of the surrounding soft soil has been varied to investigate the influence of spacing distance between columns on the behavior of the soft soil foundation.

The whole case study is modelled and standard fixities are assigned to the boundary conditions which is available in Plaxis.

b) Mesh generation

Plaxis uses unstructured mesh, which is generated automatically with options for global and local mesh refinement. Plaxis provides five choices of mesh density ranged from very coarse mesh to very fine mesh. In this research, medium mesh was chosen. Mesh was refined in zones which stresses, and strains are expected to be high i.e. the upper part of the stone column and the surrounding soil.

c) Initial conditions

Once the geometry of the model has been created and the finite element mesh has been generated, the initial situation must be specified. Plaxis provides an option to specify the initial conditions. This option consists of two modes: one mode for the generation of the initial water pressure and the other mode for the specification of the initial geometry configuration and the generation of the initial effective stresses.

2.5.2. Calculation Program

After generation of a finite element model, calculation can be executed, and calculation type must be specified in this step.

2.5.2.1. Types of calculations

Choices between diverse ways of analysis of the actual problem are made in the Calculation program. Distinction is made between three basic types of calculations, a plastic calculation, Consolidation analysis and Phi-c reduction (safety analysis).

- Plastic calculation should be selected to carry out an elastic-plastic deformation analysis in which it is not necessary to take excess pore pressures with time into account. The plastic calculation does not take time effect into account, except when the Soft Soil Creep model is used.

- Consolidation analysis should be selected when it is necessary to analyse the development or the dissipation of excess pore pressures in water-saturated clay-type soils as a function in time. Plaxis allows for true elastic-plastic consolidation analyses. In general, a consolidation analysis without additional loading is performed after an undrained plastic calculation. It is also possible to apply loads during a consolidation analysis. Varying time spans can be considered by choosing *Consolidation* and then enter the desired number of days. If full consolidation analysis is wanted, *Minimum Pore Pressure* should be selected, where all excess pore pressure is reduced. The plastic calculation and the consolidation analyses have been used in the current study.

- Phi-c reduction (safety analysis) can be executed by reducing shear parameters. A safety analysis can be performed after each individual calculation phase and thus for each construction stage to calculate the safety factor. However, the Phi-c reduction cannot be used as a starting condition for another calculation phase because it ends in a state of failure. All the analyses stated above will be used been used in the current study.

2.5.2.2. Loading types

After specifying calculation type, the loading has to be specified. The following types of loading can be selected:

- Staged construction is the most important type of loading. In this Plaxis feature it is possible to change the geometry and load configuration by deactivating or reactivating loads, volume clusters and structural objects as created in the geometry input. Staged construction enables an accurate and a realistic simulation for various loading, construction and excavation processes To carry out a stage construction calculation, it is first necessary to create a geometry model that includes all the objects that need to be used during the calculation. A stage construction can be executed in Plastic calculation or in Consolidation analysis which both of them have been used in the current research.

- Total multipliers type is used to specify the ultimate values of external loads. When the total multiplier loading is selected, the ultimate values of external loads will be applied exactly at the end of calculation.

- Incremental multiplier type is selected when the external load is applied incrementally. Before entering a load increment, an increment of time can be entered. Increments of time are not relevant when using plastic calculation except when time dependent models are used. The input of time increments is essential when using consolidation analysis.

2.5.3. Output

When the calculations are completed the results can be viewed in the Output program. A large amount of data can be obtained from a finite element calculation such as stresses, pore pressures and displacements for soils, and displacement.

2.5.4. Curves

In the Calculation program, there is an option to pre-select points of interest in the model. If such a point is pre-selected, the displacement, the stress or the pore pressure of the point for each iteration, step or time can be viewed in the sub program Curves. The results can be viewed in either a table or as a graphic curve.

The methodology of our work is then base on the good selection of parameter and configuration of the software and from it we can now simulate different initial condition and boundary condition as mentioned above.

2.6. Cost analysis

The cost analysis will be done to confirm that the ground improvement technique proposed in this work is economically acceptable. The cost will be obtained by multiplying the quantities needed by the unit prices gotten from enterprise.

The volume of material needed by the columns used will be obtained from the formula,

$$V = \frac{\pi D^2}{4} \times H \times N \quad \text{(E 2.11)}$$

Where D, H and N are respectively the diameter, height and number of the columns. The surface area of the geotextile used corresponds to the surface area of the road section considered, given by the formula,

$$A = L \times W \quad \text{(E 2.12)}$$

The volume of the pouzzolana (drainage blanket) will also be obtained by multiplying the surface area of the section by the thickness of the pouzzolana layer according to the formula,

$$V = A \times t \quad \text{(E 2.13)}$$

Conclusion

The study of the behaviour of our embankment will be applied on the construction of a 6m high embankment made of reddish clay. This embankment is extended over 300m long and is constructed on soft soil made up of clay with low strength. The height of the embankment causes problems like bearing capacity and instability. As mentioned in this chapter, the data were obtained from laboratory and in situ geotechnical test realized by the enterprise. Since the access to laboratory for experimental simulations is expensive and complicated, the numerical simulation was chosen and the methodology of this simulation was specified in this chapter after the use of an analytical method to have an idea on column characteristic. The next chapter deals with the presentation of the result from the site visit, the data obtained, the implementation of an analytical procedure to evaluate the settlement of a single column in axisymmetric and a numerical procedure to evaluate and present results of settlement, pore pressure and factor of safety for slope stability under different conditions in plane strain.

Chapter 3 : RESULTS AND INTERPRETATION

Introduction

This chapter reveals the results of the researches carried out during this work. It goes from the presentation of the site using data obtained from site visit and project documentation, through presentation of technical data to the analytical and numerical analysis of the embankment supported by a stone column improved soft soil under different loading conditions.

The soft soil is reinforced by stone columns to assess the stability of the high embankment. The FEM package of Plaxis 8 program analysis has been used to provide all the valuable information, which is required to confirm this stability. In the following sections, the modelling of the stone columns, the soft soil and the embankment, and the discussion of the results of the parametric study are presented. The discussion contains the effect of the stone columns on stability of the embankment by factor of safety analysis. The analysis of the system will be launched for undrained and drained conditions.

3.1. General presentation of the site

The presentation of the site hosting the case study of this research work will be based on two main aspects, namely the physical parameters (geographical location, climate, topography, geology...) and the socio-economic parameters (population, agriculture).

3.1.1. Physical parameters

This The physical parameters are geographical location, climate, relief, hydrology, hydrogeology, geology, vegetation and seismicity.

3.1.1.1. Geographical location of the project

The project entitled "*Routes de desenclavement du basin agricol de l'Ouest* " is a social project designed to facilitate the evacuation of agricultural production. Situated in the western region of Cameroon precisely in the eastern zone, the second of this road project links over 107 km the towns of Bangangté-Foumbot-Bamendjing-Galim, this subdivided into 2 sections (Figure 3.1). Section 1 extends over a length of 60 km from PK 0 to PK 60 and links the city of Bangangté, capital of Ndé division, and Foumbot city in the Noun division. Bangangté is connected to the capital city of the west region (Bafoussam) by the national road N°4 over about 50km, while the national road N°6 connects Foumbot to Bafoussam over 26.1km. As for section

2, it links over 47 km (from PK 0 to PK 47) the town of Foumbot to the towns of Bamendjing and Galim both located in the Bamboutos division. In terms of geographical coordinates, the project extends in longitude from 10° 24' 00" to 10° 31' 38" East, and in latitude from 5° 41' 59" to 5° 08' 46" North.

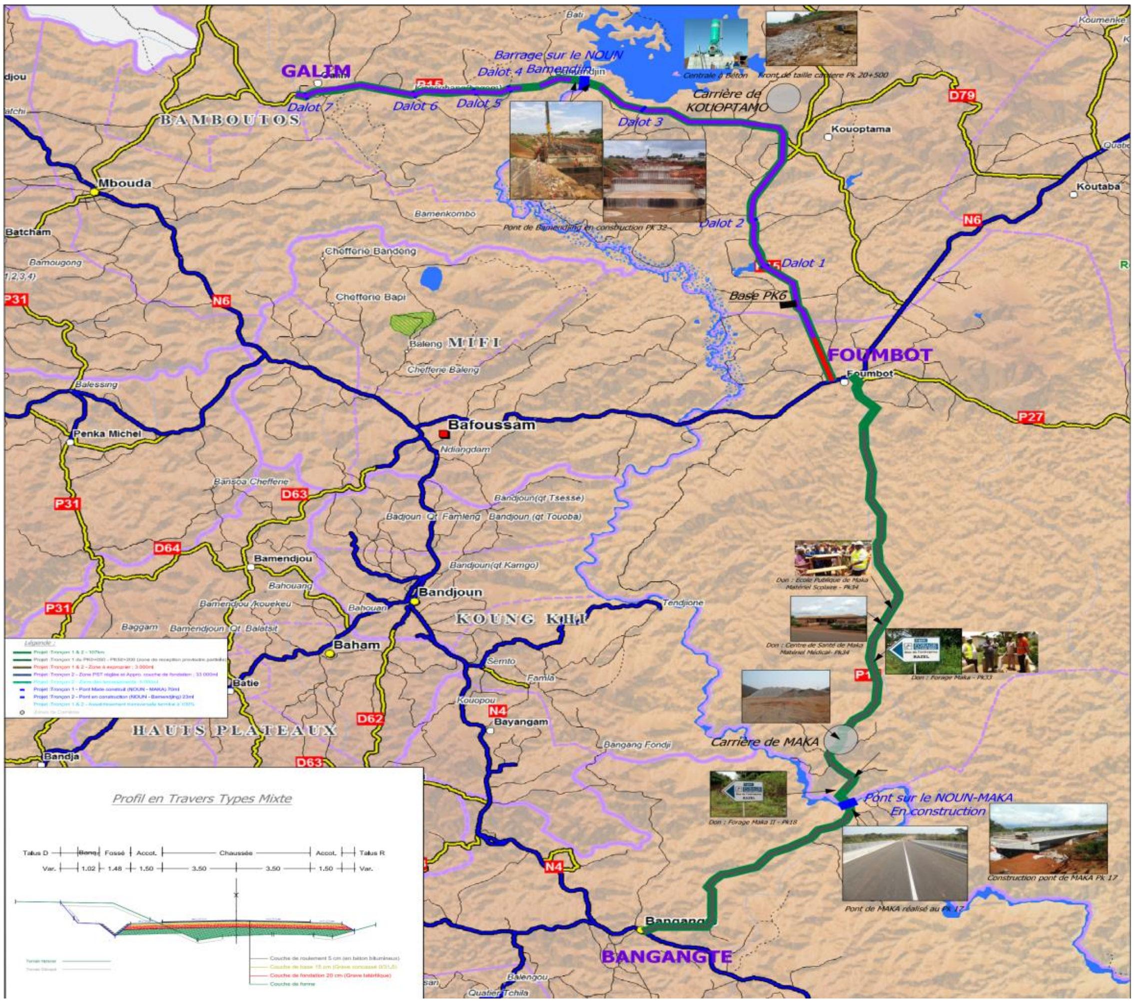


Figure3.1. Localization of the study area (Project document)

3.1.1.2. Climate

The climate of the west region is in general equatorial of the Cameroon sub variety and has two main seasons: a dry season from October/November to March/April characterised by a strong evapotranspiration and a rainy season covering the rest of the year. Among the towns in the study area, the commune of Foumbot has the highest rainfall value, which varies between 2,500 and 5,000 mm of rain per year (IRAD, 2013). Temperatures oscillate around 21°C with maxima of 32°C and minima of 14°C. As for humidity, the study area has a high average relative humidity of 80% with peaks in August and September. The region is also subject to strong winds that change direction and strength according to the seasons, sometimes causing damage to fragile plantations such as coffee trees.

3.1.1.3. Topography

On the whole, the Western region of Cameroon presents a mountainous topography with altitudes ranging from 500 to more than 2500 m which extend along the Cameroonian fault. The highest peak in the region, standing at 2740 m, is part of the chain of the Bamboutos Mountains which are dormant volcanoes on the western side of the town of Mbouda, 22.4 km from Galim. The territory of the Foumbot municipality is made up in places of isolated mounds and residual hills of very low height, the western hillside of Mount Mbapit (2352m of altitude) is installed in this territory. In general, the hilly areas are excellent places for large livestock while the valleys and abundant plains are used for seasonal crops. The present topography forms are the result of a long and complex volcanic action that occurred in the area.

3.1.1.4. Geology

Most of the soils in the study area are the result of volcanic activity. In the plains of the commune of Foumbot, there are very porous and fertile black amorphous alluvial soils resulting from volcanic projections (pozzolan). These soils have an important agronomic value due to their high nitrogen, phosphorus and potassium content (PNDP, 2014). Due to their overexploitation and the difficulty of managing rain and wind erosion, this soil is becoming increasingly poor. There are also several other types of soil in the region, namely hydromorphic alluvial soils mainly in the lowlands, reddish lateritic soils remarkably present on the slopes of some peaks in the commune of Bangangté and sandy-clay soils in the marshy areas (PNDP,2015).

3.1.1.5. Hydrology and hydrography

The territory of West Cameroon, due to its mountainous topography and the depth of the valleys, is watered by a dense hydrographic network made up of tortuous rivers with regular and seasonal regimes. These rivers, all part of the Atlantic basin, experience a high-water period during the rainy season and a low water period during the dry season. Among these rivers, the most important in the study area is the Noun River which is fed by smaller rivers such as the Kon, Ngam and Ndé and flowing from the central region around Bafoussam to the Bamendjing reservoir. This artificial lake is created by a dam on the river Noun, which contributes to the regulation of the Sanaga at Edéa in the Littoral region. Most of the lakes in the region are crater lakes formed as a result of the collapse of volcanoes. One example is Lake Baleng, northeast of Bafoussam, and the twin lakes of Foumbot.

3.1.2 Social and economic characteristics

A knowledge about the population and agricultural behaviour is capital for the social and economic characterisation.

3.1.2.1. Population

According to the last census of the population of Cameroon in 2005, the western region had about 1,720,047 inhabitants, making with respect to its surface area, one of the most densely populated regions with 124 inhabitants/km². As far as the Foumbot commune is concerned, its population is estimated at about 76,486 inhabitants with 38,891 women and 37,595 men. With a growth rate of 2.6%, this would give a population of 90,406 in 2012 PNDP, 2014). This population is essentially made up of Bamilékés, Bamouns, Banso'o and Mbororos who have migrated from the north in search of pasture for their cattle and have settled there permanently.

3.1.2.2. Agriculture

Agriculture occupies an important place in the economic activity of the West Cameroon region in general and the commune of Foumbot in particular. This is due to the richness of the soil in nutrients and fertilizers. Thus, we encounter annual cultures dominated by maize, okra, watermelon, tomatoes; semi-perennial cultures such as plantains and perennial cultures such as coffee trees which are quite widespread in the commune.

3.1.2.3 Mining and quarrying

There are no real mining industries in the commune of Foumbot industries. We can note in this city the presence of a few unorganised operators of pozzolan or sand quarries. The exploitation of these quarries and the removal along the roadsides constitute an economic activity because of the financial income they provide to the populations who work in them, as well as the exploitation fees collected by the Council from operators in certain villages in the communes. Also, it provides building material for housing and public works. Given the rocky outcrops encountered in the municipalities, it can be noted with certainty that the potential of the stone quarries in the communes is still under-exploited. The same is true of the laterite deposits. This is an obstacle to the economic development of the area in view of the mass of jobs and income that their operations would provide for both the local population and the communal institution.

3.2 Phisical description of the site

A visit of the project site was made upon our arrival at the base. From an overview, the work is progressing normally, traffic is very light probably due to the condition of the road, some engineering structures are being built as the bridge over the 'Noun' and a fairly considerable part of the section is being routed. Agriculture is predominant in the cities affected by the project.

Nevertheless, many different geotechnical problems can be observed along the project line, such as water rising due to capillarity (KP 12+500m), pozzolanic stability problems (KP48+900m) and compressible soil stability problems (KP12+650m), settlement problems and also too large time required to consolidate the embankment in compressible soils which are the problem of our case study (KP12+650m). This can be shown in Figure3.2, Figure3.3, Figure3.4, Figure3.5 and Figure3.6.

Figure3.2. Marshy soil of the case study (PK12+650 left)

Figure3.3. Initial state of the case study (PK12+650 right)

Figure3.4. Initial state of the case study (PK12+900 middle)

Figure3.5. Initial state of the case study (PK12+900 right)

Figure3.6. Initial state of the case study (PK12+675 right)

3.3 Data presentation

The geometrical and geotchnical data constitute the main technical data to be used in this study.

3.2.1 Geometrical data

The case study involved in this research is in the second section of the project from PK 12+650 to PK 12+950. The pavement has a width of seven (7) meters and a thickness of forty (40) centimetres divided as follow

- Subbase course (20 cm)
- Base course (15 cm)
- Wearing course (5 cm)

Under the pavement, there is a 6 m high embankment. The embankment has a trapezoidal section with a base of around 20 m and a top of 10 m. This embankment rests on a soft soil of thickness of about 12 m. Figure 3.7 shows a typical transverse profile of the study.

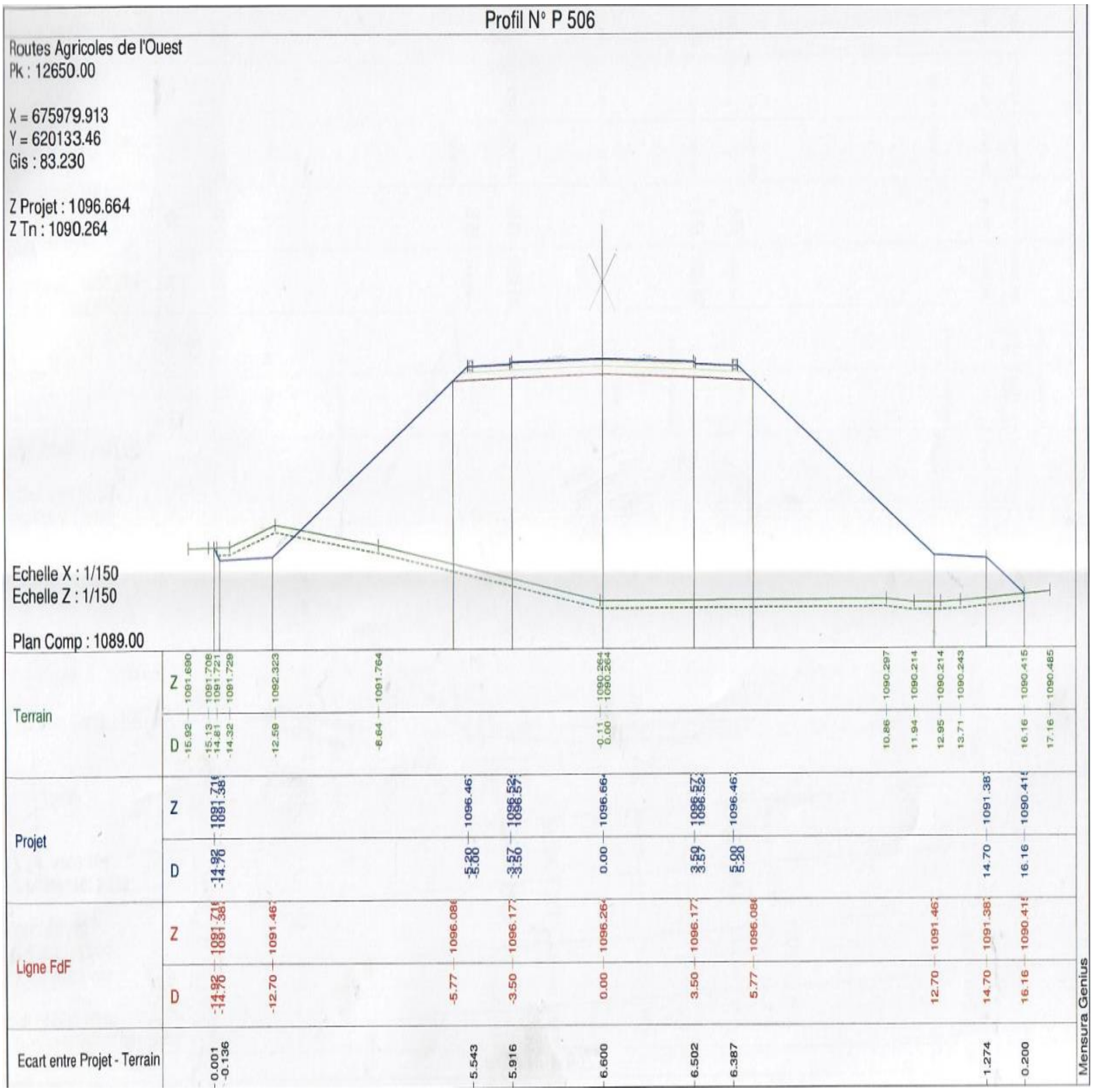


Figure3.7. Typical transverse profile at PK 12+650 (Project documents)

3.2.2 Geotechnical data

For the analysis, the profile chosen is that of the PK 12+650 which corresponds to the embankment with the highest height (6 m). The choice of this profile puts us in the worst conditions and once the stability of this profile is assessed, it is normal that all the other profiles with lower embankment heights will be stable as well.

Geotechnical data obtained for the embankment soil and foundation soil from in situ and laboratory tests will be presented in the next lines.

3.2.2.1 Embankment soil geotechnical data

In this case study, the embankment soil is a reddish clay. The geotechnical parameters of this clay are summarised in Table 3.1

Table3.1. Embankment soil parameters

Soil Layers	γ_{sat} $(kN/m3)$	γ_{dry} $(kN/m3)$	Φ' (°)	Ψ (°)	E (kpa)	v	c', c'_u (kpa)	e_o	K (m/day)	K_o
reddish clay	18.61	16.68	30	0	10.5×10^4	0.3	10	-	0.001	-

3.2.2.2 Foundation soil geotechnical data

In this case study, the foundation soil is a mixture of different soil types. In situ tests like the Cone Penetrometer Test (CPT) was used to define the soil stratigraphy and resistance while laboratory tests such as Direct shear test, Atterberg limit tests, Oedometer test and Permeability tests where used to complete the identification of the soil and the compressibility parameters where obtained. Figure 3.8 shows the stratigraphic profile of the soil obtained from the CPT.

For this analysis, part of the first layer of soil (soft darkish clay) will be replaced by pouzzolana for drainage purposes.

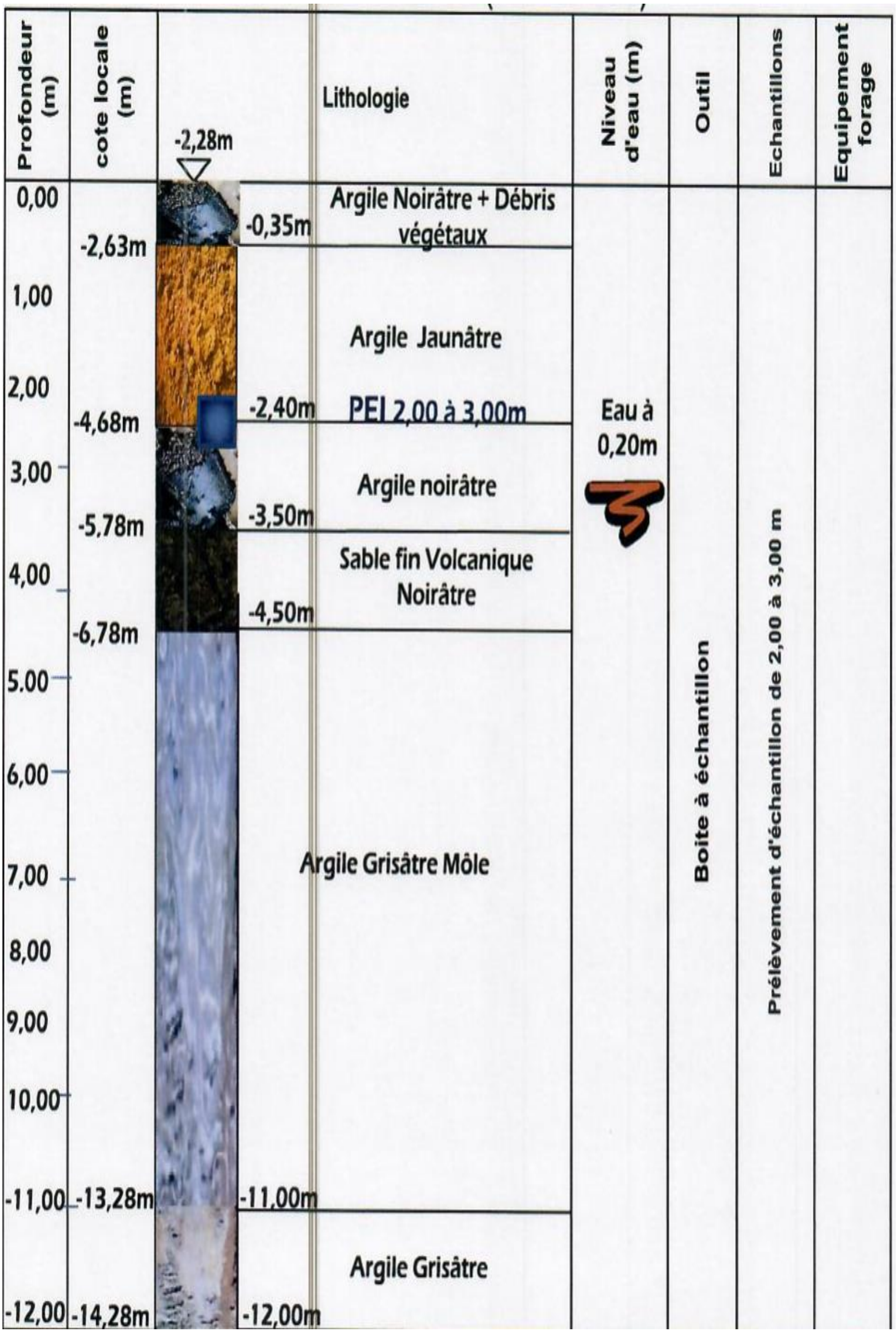


Figure3.8. Stratigraphic profile at PK 12+650 (Project documents)

Table3.2. Foundation soil parameters (Project Documents)

Soil layer	Soft darkish clay	Yellowish clay	Darkish clay	Volcanic darkish fine sand	Soft Greyish clay
Thickness (m)	1.25	0.8	1.1	1.0	7.5
γ_{sat} $(kN/m3)$	15.25	14.77	16.25	17.27	15.68
γ_{dry} $(kN/m3)$	10.07	8.16	11.07	12.23	9.20
K_h $(m\,/day)$	0.001	0.01	0.002	0.002	0.002
K_v $(m\,/day)$	0.001	0.01	0.002	0.002	0.002
E (kpa)	3491.4	4234.3	3491.4	4531.4	2822.9
E_{oed} (kpa)	4700.0	5700.0	4700.0	6100.0	3800.0
v	-	-	-	-	-
c', c'_u $(\,kpa)$	19	19.8	19	33.1	25.3
Φ' (°)	14.4	22.8	14.4	24.2	4.01
Ψ' (°)	-	-	-	-	-
σ_p (kpa)	39.82	29.69	39.82	48.32	21.95
C_c	0.289	0.221	0.289	0.198	0.394
C_s or C_r	0.120	0.091	0.120	0.064	0.232
C_α (computed)	0.0101	0.0077	0.0101	0.0069	0.0138
C_v (m^2/day)	0.014	0.036	0.014	0.029	0.009
e_0	1.376	1.236	1.376	0.954	1.741

The value of C_α is computed using the references in table 3.3. the ratio chosen here is 3.5%

Table3.3. Uniform C_α/C_c ratio (USACE)

Material	C_α/C_c ratio
Inorganic material	$0.025 - 0.065$
Clay	$0.025 - 0.085$
Silt	$0.030 - 0.075$
Peat	$0.030 - 0.085$

3.2.2.3 Stone Column geotechnical data

The gravel present on the site where not tested, for this reason, the parameters for the stone columns in our case study will be gotten from the literature proposed by Dipty and Girish (Table 3.4)

Table3.4. Properties of gravel material for stone column (Dipty and Girish, 2009)

Parameter	$\boldsymbol{E}$ $(\boldsymbol{kpa})$	$\boldsymbol{v}$	c' $(\boldsymbol{kpa})$	$\boldsymbol{\Phi}'$ (°)	$\boldsymbol{\Psi}'$ (°)	$\boldsymbol{K_h}$ $(\boldsymbol{m/day})$	$\boldsymbol{K_v}$ $(\boldsymbol{m/day})$	$\boldsymbol{\gamma}$ $(\boldsymbol{kN/m3})$
value	45000	0.3	0	42	0	6	6	19.4

3.2.2.4 Pouzzolana geotechnical data

Pouzzolana material will be used the drainage layer on top of the stone columns and also as part of the embankment. The parameters of the pouzzolana are summarized in Table 3.5

Table3.5. Pouzzolana parameters (Project Documents)

Parameter	$\boldsymbol{E}$ $(\boldsymbol{kpa})$	$\boldsymbol{v}$	c' $(\boldsymbol{kpa})$	$\boldsymbol{\Phi}'$ (°)	$\boldsymbol{\Psi}'$ (°)	$\boldsymbol{K_h}$ $(\boldsymbol{m/day})$	$\boldsymbol{K_v}$ $(\boldsymbol{m/day})$	$\boldsymbol{\gamma_{sat}}(\boldsymbol{kN/m3})$	$\boldsymbol{\gamma_{dry}}(\boldsymbol{kN/m3})$
value	10000	0.3	0.5	38	0	10	10	14.10	13.01

3.2.2.5 Embankment soil geotechnical data

The embankment soil is composed of pouzzolana and strong reddish clay with the parameters presented in Table 3.6

Table3.6. Embankment soil parameters-Reddish clay (Project Documents)

Parameter	$\boldsymbol{E}$ $(\boldsymbol{kpa})$	$\boldsymbol{v}$	c' $(\boldsymbol{kpa})$	$\boldsymbol{\Phi}'$ (°)	$\boldsymbol{\Psi}'$ (°)	$\boldsymbol{K_h}$ $(\boldsymbol{m/day})$	$\boldsymbol{K_v}$ $(\boldsymbol{m/day})$	$\boldsymbol{\gamma_{sat}}(\boldsymbol{kN/m3})$	$\boldsymbol{\gamma_{dry}}(\boldsymbol{kN/m3})$
value	105000	0.3	10	30	0	0.001	0.001	18.61	16.68

3.2.2.6 Geosynthetic geotechnical data

The geosynthetic material used in this research is the geotextile with parameters shown in Table 3.7

Table3.7. Geotextile parameters (Project Documents)

Parameter	$EA(\boldsymbol{kN/m})$	$\rho\ (g/m^2)$
value	30	400

3.4. Stone Column analytical design

The method of analytical design used in this work is that proposed by De Beer-Van Impe in 1983 as mentioned in chapter 2. The aim of the design is to fine the stresses and vertical settlement in the column and surrounding soil.

The parameters needed for the design are;

p_o: The load on the system

E_t: The modulus of elasticity of the soil

ν_t: Poisson ratio of the soil

K_{ac}: Coefficient of active pressure on the column

a, b,: Horizontal and vertical spacing between the columns

α: Area ratio

H: Height of columns

For the design we will use data from project document and literature presented as follows.

$p_o = 120\ kPa + 10\ kPa + 100\ kPa = 230\ kPa$

$E_t = 3715\ kPa$

$\nu_t = 0.3$

$K_{ac} = 0.33$

$K_{at} = 0.51$

$a = b = 5\ m$

$D = 1\ m$

$\alpha = 0.03$

$H = 11\ m$

With the method, we consider a rectangular arrangement of the columns and we fine an equivalence of area to have a plane strain model in which every column line is transformed in a wall with a thickness d_f. Details about the column pattern is described in Figure 2.1

$d_f = 0.15\ m$

$L = 4.85\ m$

The equations (E 2.1), (E 2.2), (E 2.3), (E 2.4), (E 2.5) and (E 2.6) can now be written with numerical application and the six equations will be solved simultaneously to find the settlements and the stresses. Upon doing the computations, we obtain the following results;

$\boldsymbol{s_v = 2.7 \times 10^{-2}\ m, s_h = 7.3 \times 10^{-4}\ m, \sigma'_{vt} = 228.4\ kPa, \sigma'_{ht} = \sigma'_{hc} = 116.5\ kPa,}$

$\boldsymbol{\sigma'_{vc} = 353.0\ kPa}$

3.5. Numerical analysis results and interpretation

As mentionned in chapter 2, to model a realistic situation of the stone column supported embankment in order to appreciate the effect of stone columns on the stability of the embankment, the MohrCoulomb model in Plaxis 8 program is used for the stone columns material and the Soft Soil Creep model is used to describe the behaviour of the clay. The properties of the stone andsand columns material were adopted from the study of Dipty and Girish (2009) and the properties of clay were adopted from the geotechnical tests performed by the enterprise. The stone column material is modelled in drained condition while the surrounding soft soil is mainly modelled in undrained condition and calculated under plane strain. The properties of these soils are presented in Table 3.2, Table 3.4, Table 3.5, Table 3.6 and Table 3.7. A summary of the material model and loading conditions is shown in Table 3.8

In the present study, a numerical model for initial conditions was first done to have a better appreciation of the efficiency of the method of improvement proposed.

Table3.8. Summary of material used, their material model and loading conditions

Material	Soft compressible soil	Stones for stone columns	Pouzzolana (Drainage material)	Embankment material (reddish clay)
Material model	Soft soil creep model	Mohr coulomb	Mohr coulomb	Mohr coulomb
Loading condition	Undrained	Drained	Drained	Drained

3.5.1 Modelling

Different conditions will be modelled to better understand the effect on the improvement technique chosen.

3.5.1.1 Case for initial conditions (Without soil improvement)

The soft soil layers were defined geometrically and qualitatively based on the parameters in Table 3.2. and Table 3.8. The standard fixities boundary conditions were applied permitting no displacement at the bottom but allowing only vertical displacement on the lateral sides. During construction, the embankment was loaded with a $120\ kPa$ load over a width of $2\ m$ corresponding to the compaction engine loads and its width respectively. This load was replaced by $10\ kPa$ during the interstage consolidation periods (this load stands for the load applied by the vehicles on the embankment during the interstage consolidation period). This load was applied on both senses of the road with the position chosen for the case of maximum deformations. The water table was then defined at the base of the embankment as given by the geotechnical reports. The embankment was modelled using staged construction in three stages. This choice was made based on the number of stages executed by the enterprise on site to have a good basis for comparison of results. Table 3.9 summarises the construction and consolidation time (input) for each phase or stage. The finite element model used to simulate the behaviour of the embankment in initial conditions is shown in Figure3.9 while Figure3.10 and Figure3.11 show respectively the meshing system and calculation steps of the embankment in initial conditions.

Table3.9. Summary of different embankment modelling phases

Material	Pouzzolana (2 Phases)	Reddish clay (2 Phases)	Reddish clay (2 Phases)
Tickness (m)	1.5	3	1.5
Construction Phase (days)	7	21	14
Consolidation Phase (days)	60	60	Till minimum excess pore water pressure

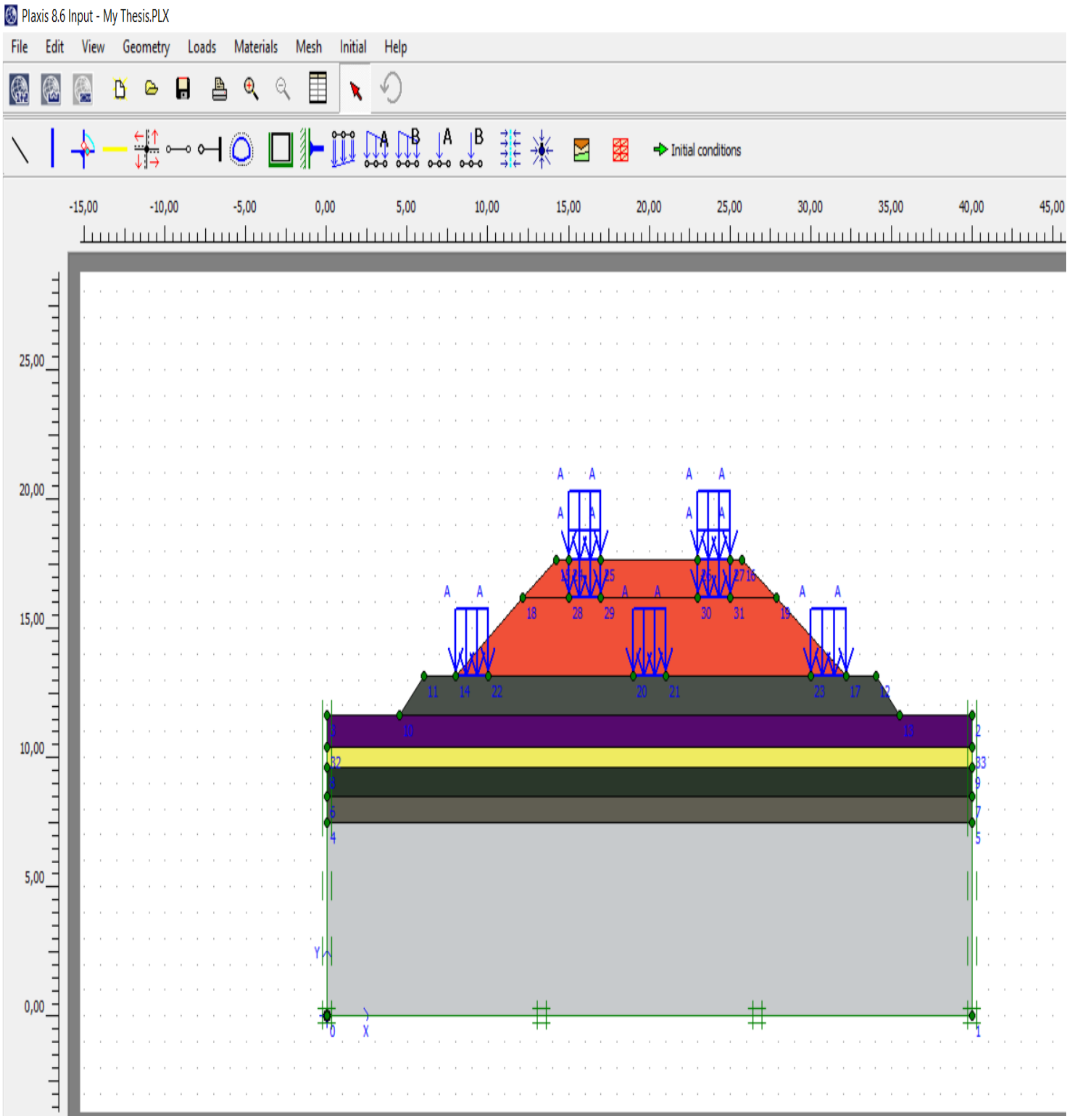


Figure3.9. Model of the embankment constructed on initial state soil (Plaxis 2008)

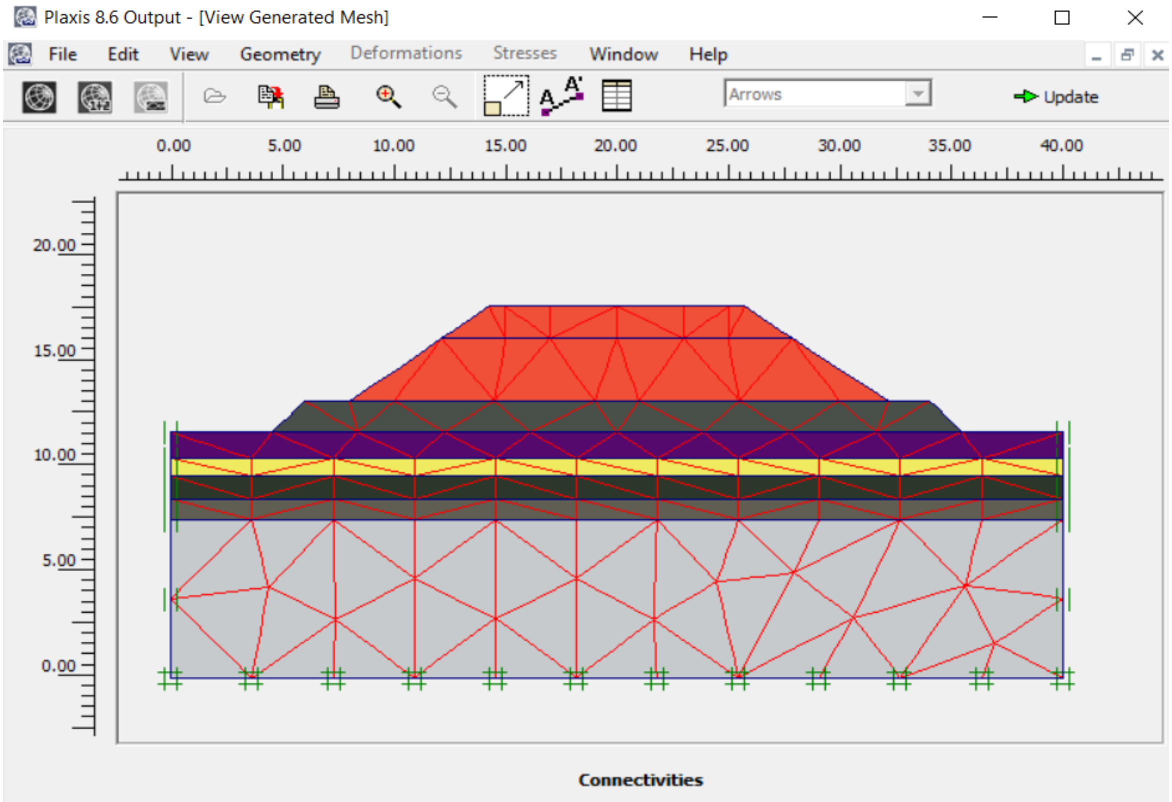


Figure3.10. Meshing system of the embankment constructed on initial state soil (Plaxis 2008)

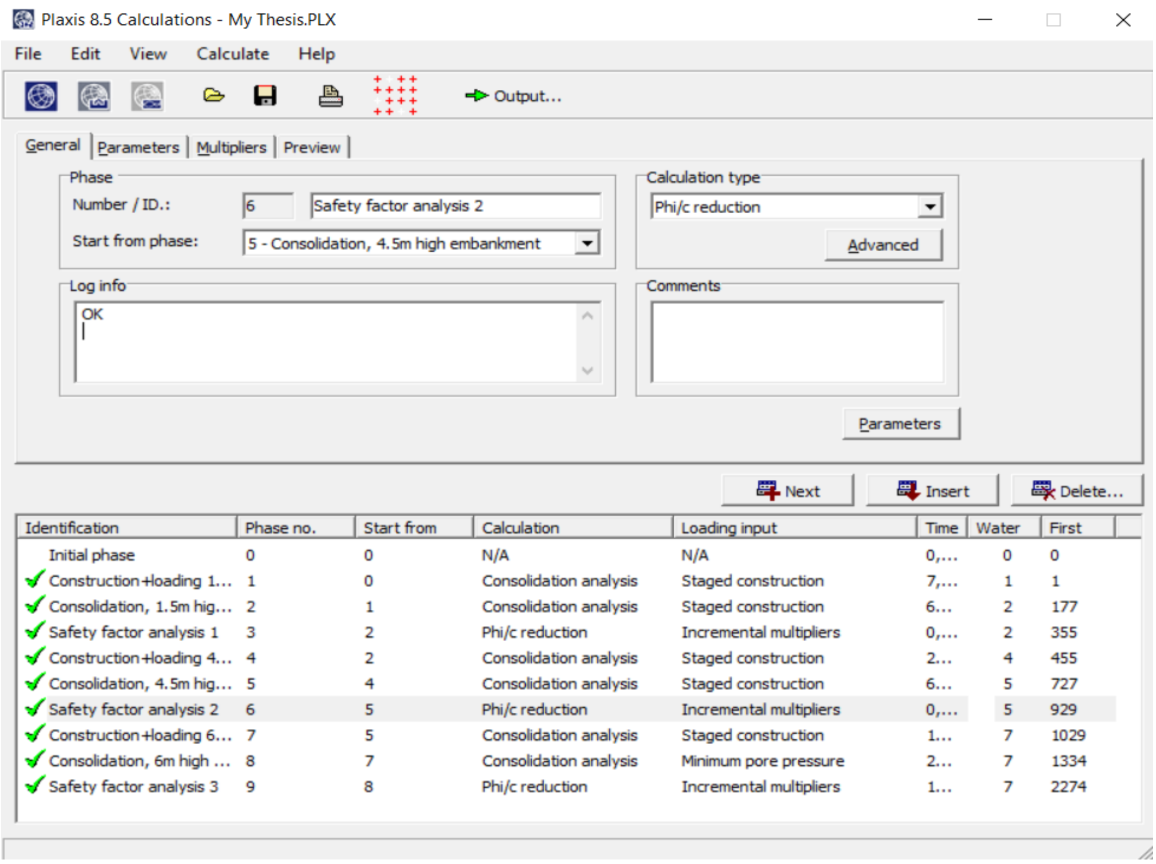


Figure3.11. Calculation steps of the embankment (Plaxis 2008)

3.5.1.2 Case with soil improvement

The system in this case was modelled in a similar way as that in the previous case. The difference lies in the fact that stone columns are inserted into the soft soil with regular spacings of 3 m for the first situation, 4 m for the second situation and 5 m for the third situation with diameters of 1 m, part (65 cm) of the top soft soil was replaced by pouzzolana for drainage purposes, a geotextile was placed at the base of the embankment for separation purposes and all the embankment was made of reddish clay. The modelling of the stone columns, pouzzolana. The finite element model used to simulate the behaviour of the embankment in reinforced conditions is shown in Figure3.13 while Figure3.12 and Figure3.14 show respectively the meshing system and calculation steps of the embankment in reinforced conditions.

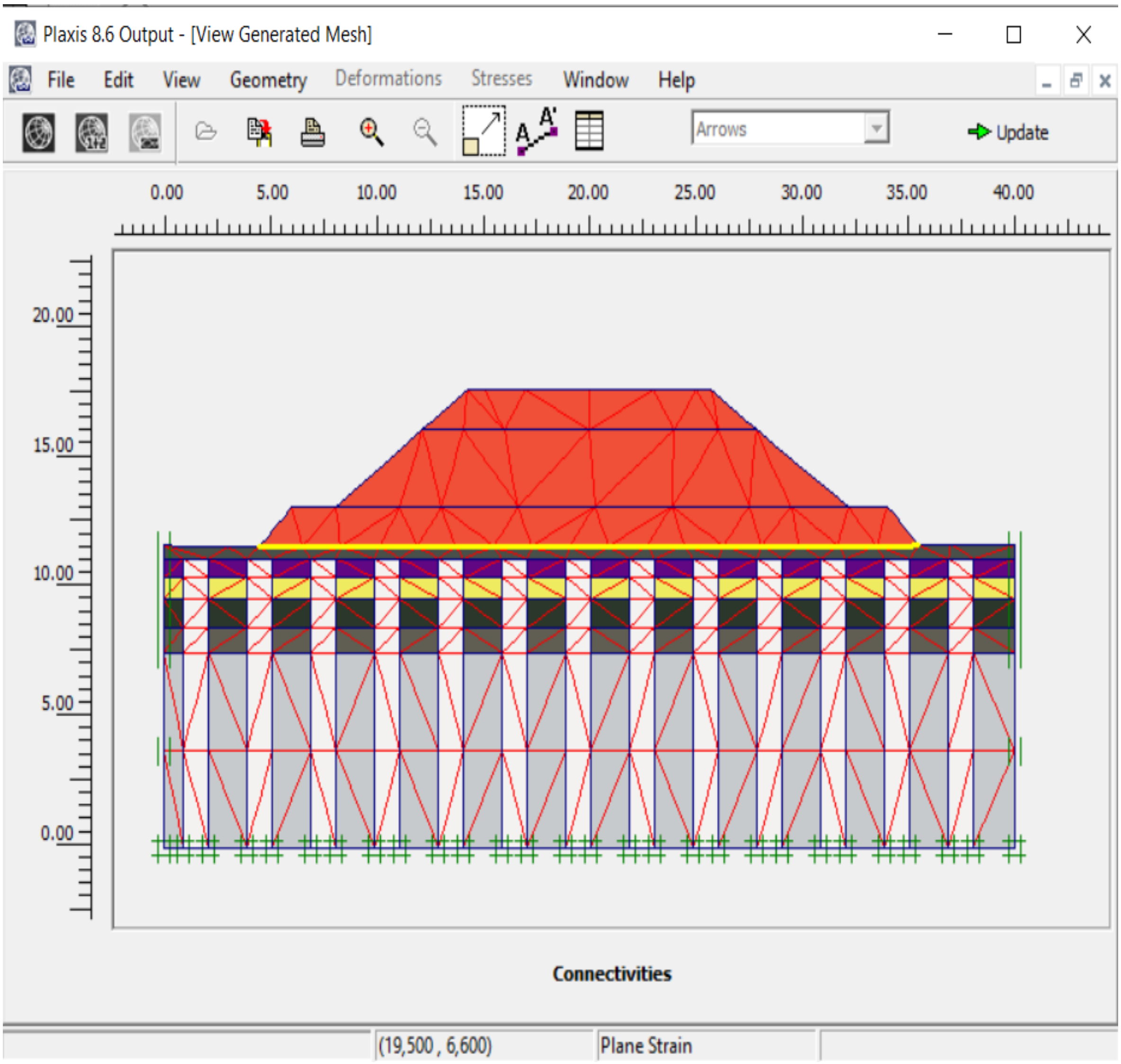


Figure3.12. Meshing system of the embankment constructed on stone columns improved soil (Plaxis 2008)

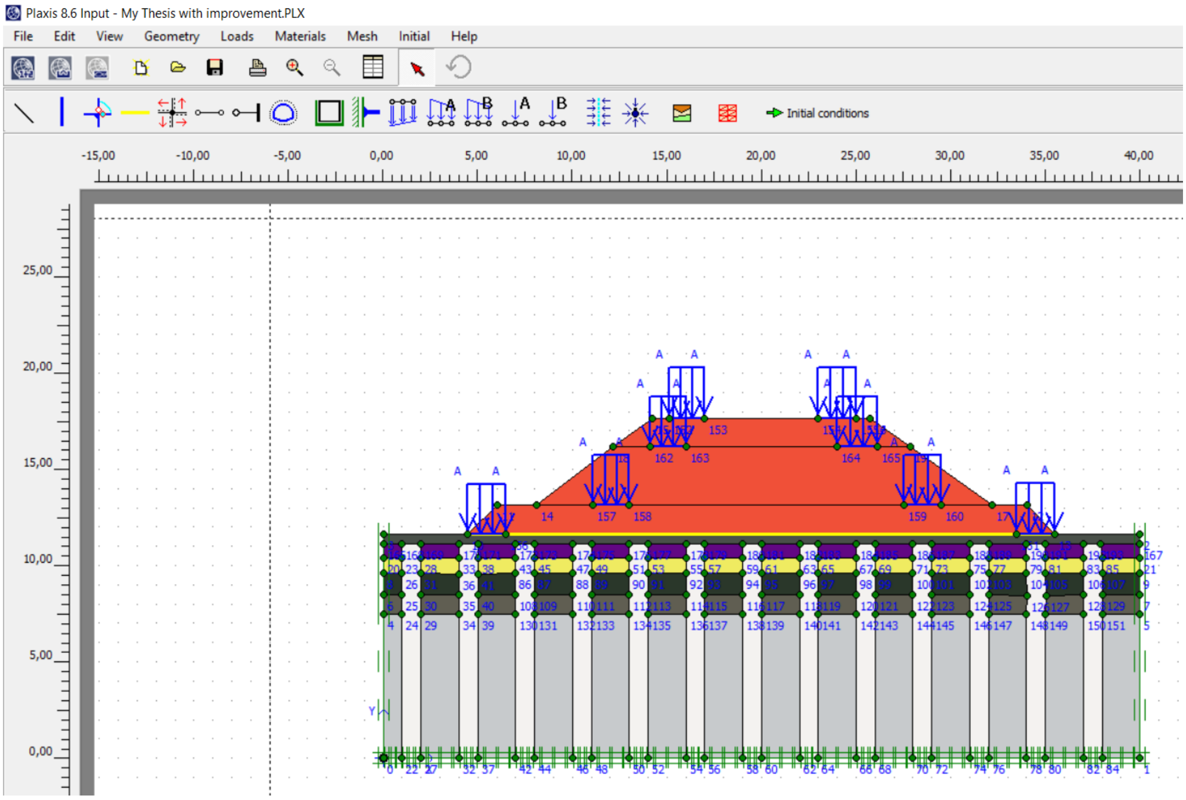


Figure3.13. Model of the embankment constructed on stone columns improved soil (Plaxis 2008)

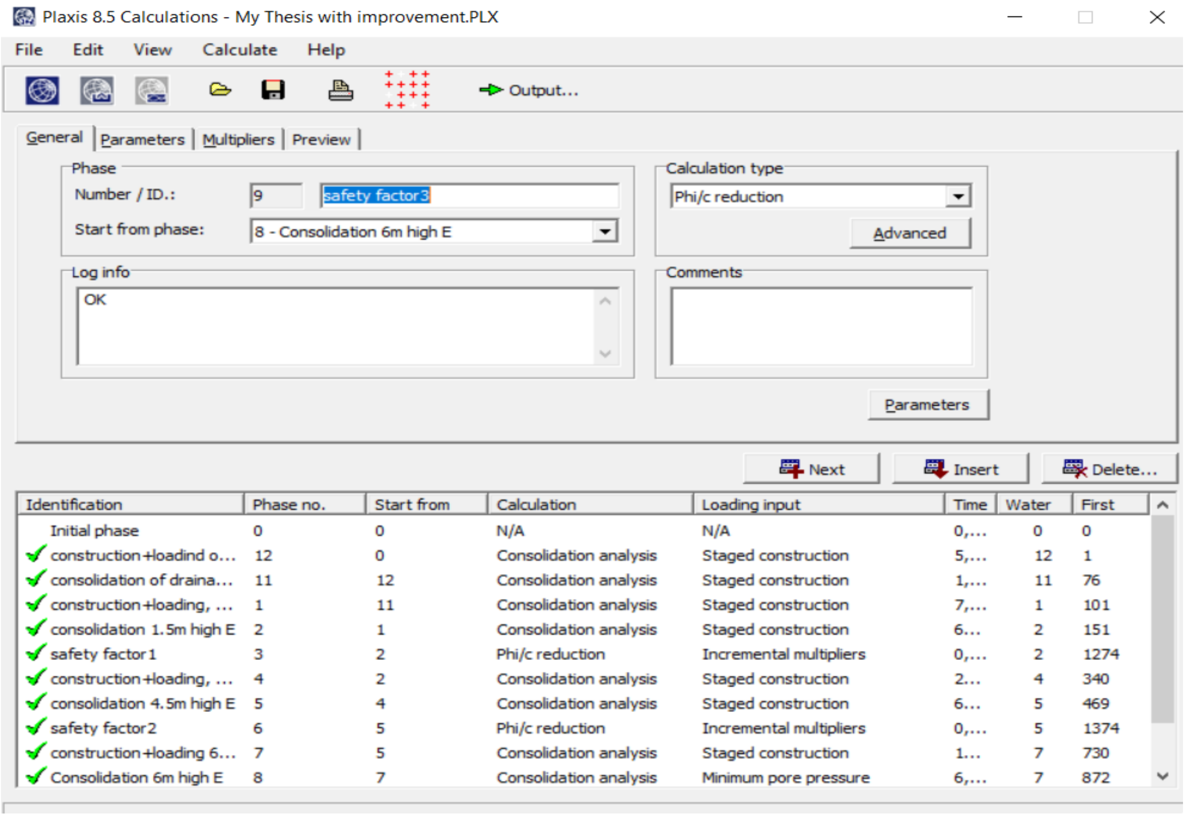


Figure3.14. Calculation steps of the embankment (Plaxis 2008)

3.5.2 Results presentation and interpretation

FEM conception by Plaxis estimated many information about the model amongst which the amount of settlements, excess pore water pressures, consolidation end time, factors of safety of embankment in both the treated and untreated conditions. In numerical analysis, different points were selected to determine the settlements, excess pore water pressures and factors of safety. Figure3.15 shows the stone column distribution. The position of the said points can be seen in Figure3.16.

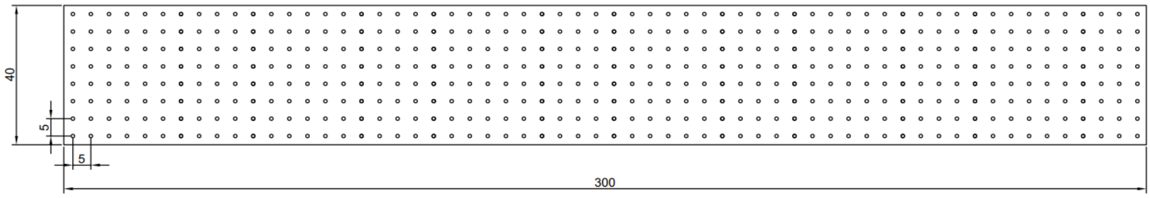


Figure3.15. Stone columns distribution from PK 12+650 to PK 12+950

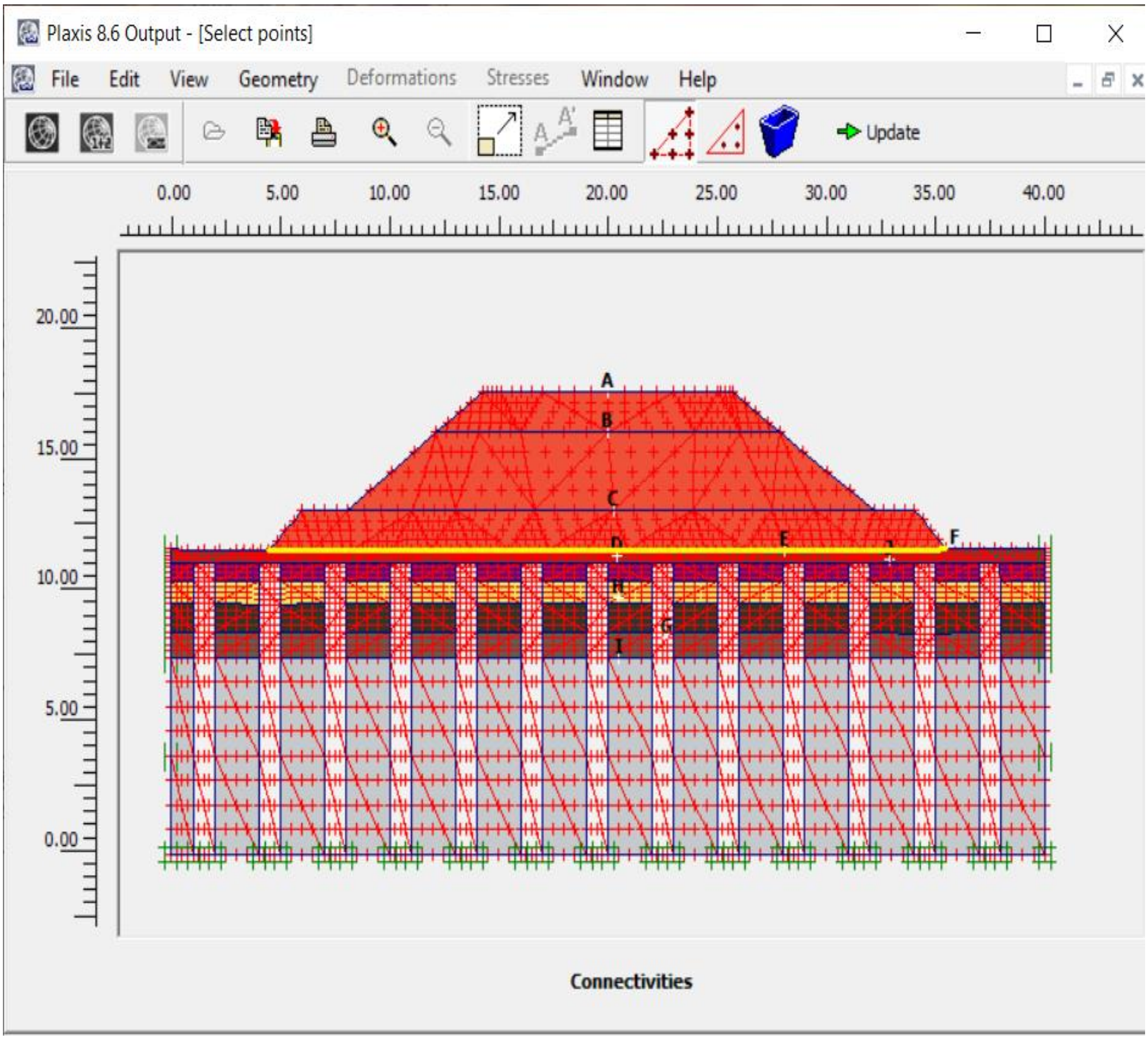


Figure3.16. Points location in FEM analysis (Plaxis 2008)

Four conditions were modelled and Figures3.17 to Figure3.18 present the results acquired from the settlement simulation at different points. From this data, we can see that settlement amount is largest for unreinforced condition, decreasing with column spacing from $S = 5\,m$ through $S = 4\,m$ to $S = 3\,m$. The columns with spacing $S = 5\,m$ reduce the maximum settlement by 52% at point C. However, the detected difference in settlements, between points C, E, and H show that the maximum settlement occurred at point C, the middle of the embankment, while the settlement at point F can be observed as a heave (swelling) for unreinforced columns and the stone columns minimizes the swelling at that point. Nevertheless, at all the chosen points, the settlement reduces with reducing stone column spacing.

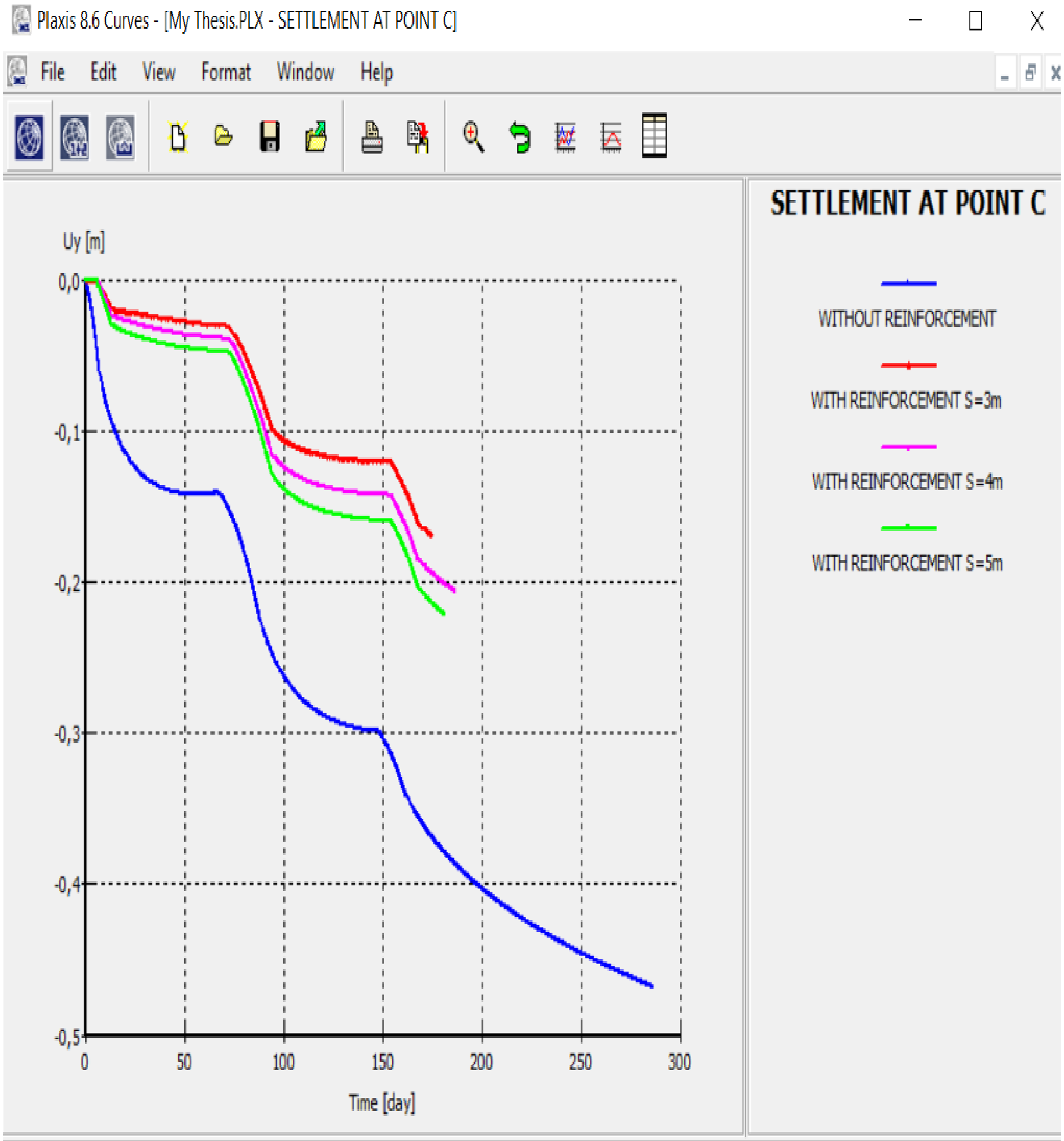


Figure3.17. Settlement versus time at point C (Plaxis 2008)

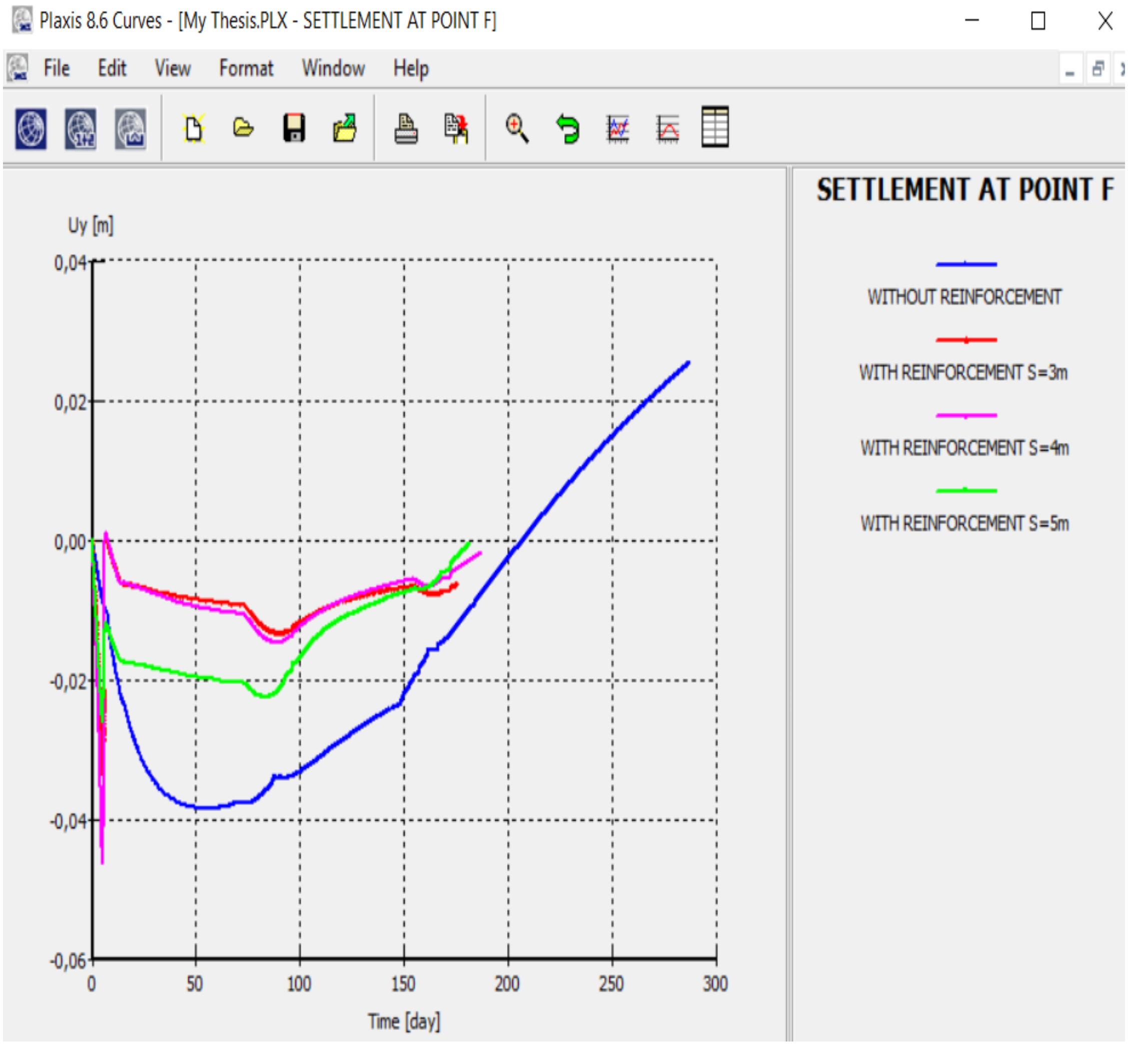


Figure3.18. Settlement versus time at point F (Plaxis 2008)

3.5.2.1 Consolidation end time analysis

The consolidation end time is the time needed to reach a minimum value of excess pore water pressure (In our case $P \leq 1kPa$). It can be seen from Figure 3.19 that the stone column with spacing S = 3 m speeds up the consolidation time in comparison to unreinforced condition from 287 days (9.6 months) to 175 days (5.8 months), the stone column with spacing s =4 m speeds up the consolidation time in comparison with the unreinforced condition from 287 days (9.6 months) to 187 days (6.2 months) and the stone column with spacing s =5 m speeds up the consolidation time in comparison with the unreinforced condition from 287 days (9.6 months) to 182 days (6.1 months). Therefore, from different column spacing, it can be seen that stone columns have a substantial influence on dissipation of excess pore water pressure and on reducing the consolidation time.

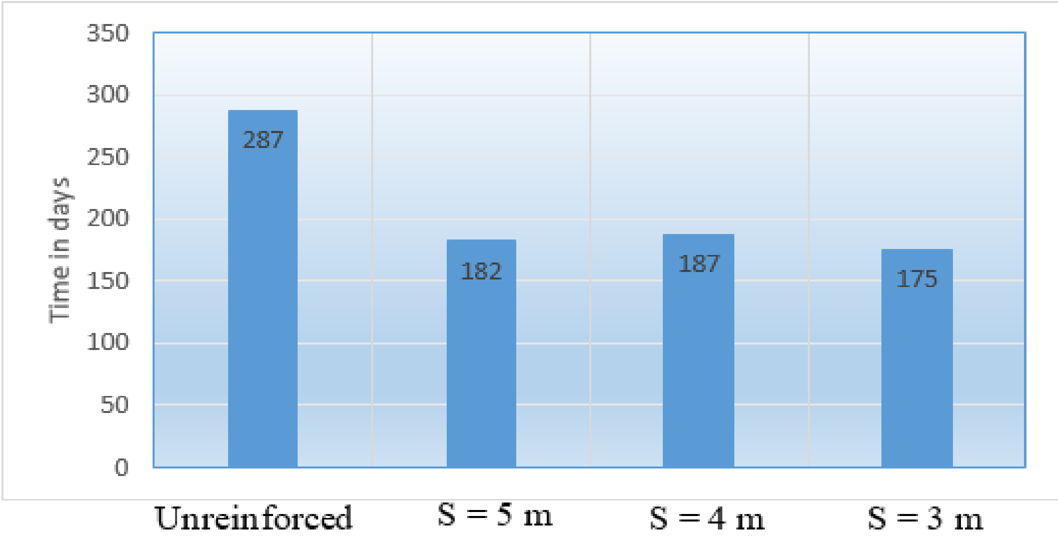


Figure3.19. Consolidation end time analysis (Plaxis 2008)

3.5.2.3 Pore Water Pressure Consideration

To realize the behaviour of excess pore water pressure in time, points D, I and H were selected. Figure3.20 show that excess pore water pressures attains the maximum amount after the construction stage of each step for embankment fill. It will then reduce progressively for each step during the small consolidation period but at the end of construction of the last stage, it will reduce with time until it becomes zero at consolidation end time. The highest excess pore water pressures for our embankment is reached at the end of construction of the first stage. Another remark is that excess pore water pressure increases with increase in depth due to stress increase. Point C is associated with little or no excess pore water pressure since it is located above the water table. Upon comparison between reinforced and unreinforced soil at point I, it is observed that for unreinforced soil, the maximum negative excess Pore Pressure is 36.9 kN/m^2 while for reinforced soil, the maximum negative excess Pore Pressure is averagely 1.7 kN/m^2 justifying the small consolidation end time. Figure3.21 shows the variation of excess pore water pressure with time at pointI for different conditions.

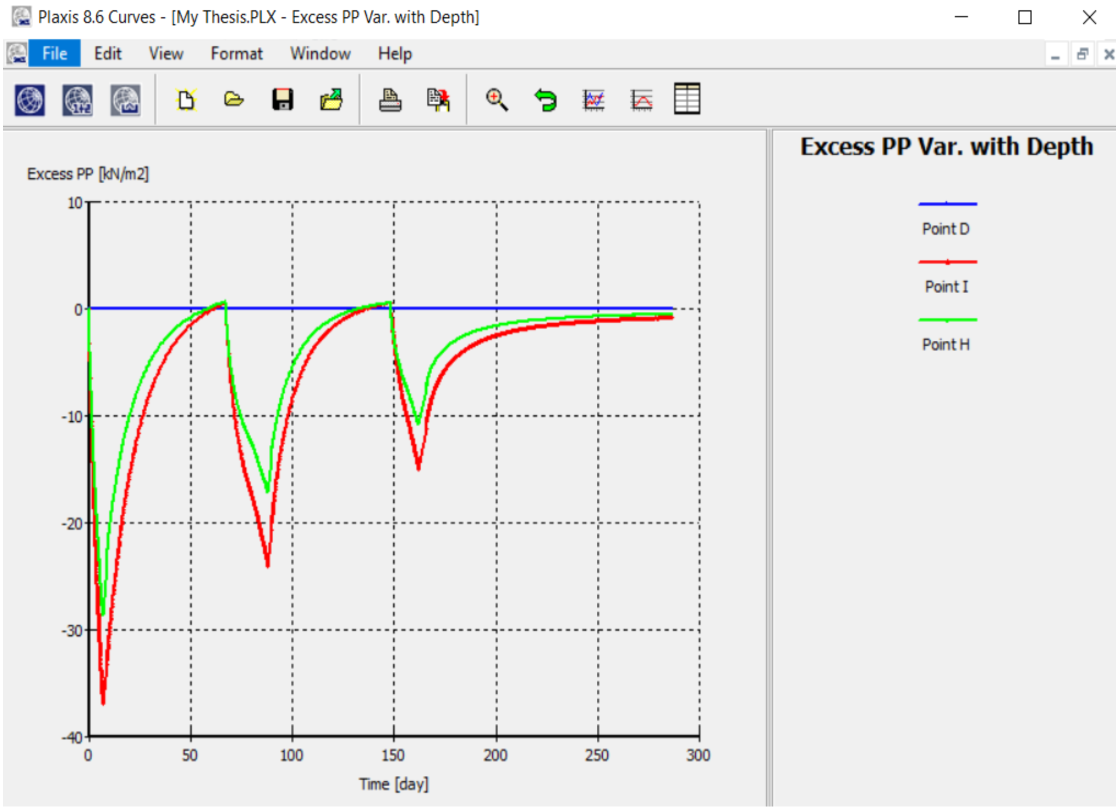


Figure3.20. Excess PP versus time for Unreinforced soil (Plaxis 2008)

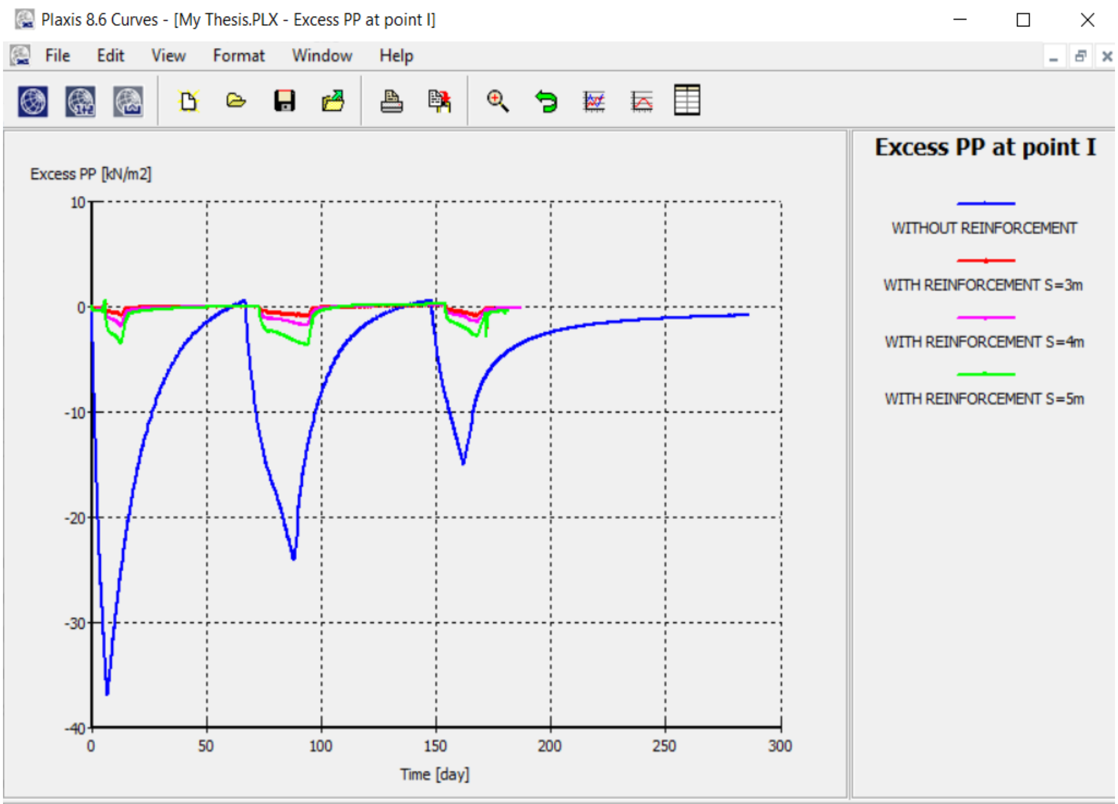


Figure3.21. Excess PP versus time for Unreinforced and Reinforced soil at point I (Plaxis 2008)

3.5.2.4 Safety Factor Analysis

The safety factor analysis is done with the help of the of the PHI/C reduction analysis. The PHI/C reduction analysis is launched at the end of the consolidation phase of each stage of construction. This analysis was done for all the three cases shown in the previous analysis.

The first remark that can be done after this analysis is that, the safety factors (SF) decreases as the stages of construction evolve, this can be explained from the principle of the PHI/C reduction method used in this analysis which consists in reducing gradually the values of the friction angle (phi) and the cohesion (c) up to the first points of failure. It can also be explained from the fact that as the embankment rises the action force on the soil increases thereby causing a need in more mobilised resistance, this leads to a drop in SF. The second remark made after this analysis is that, during the consolidation process, the SF increases with increase in the number of steps, this is directly related to the behaviour of excess pore water pressure. As excess pore water pressure is dissipated, the effective stress in the soil increases leading to higher shear resistance and hence higher SF values. However, the SF to be considered is that at the end of the consolidation process (corresponding to minimum excess pore water pressure).

The analysis done on the unreinforced soil, soil reinforced with stone columns spaced by $5\ m$, $4\ m$ and $3\ m$ reveals that the SF values are 1.13 after 287 days, 1.12 after 182 days, 1.13 after 187 days and 1.12 after 175 days respectively (end of consolidation). The reinforcement by stone columns spaced by $5\ m$ seems therefor satisfactory (technically and economically) since it provides a relatively good SF in a small period of time compared to the case of unreinforced soil as illustrated in Figure3.22. Figure3.23, Figures3.24 and Figures3.25 show how the safety factor varies with the construction steps in different calculation steps.

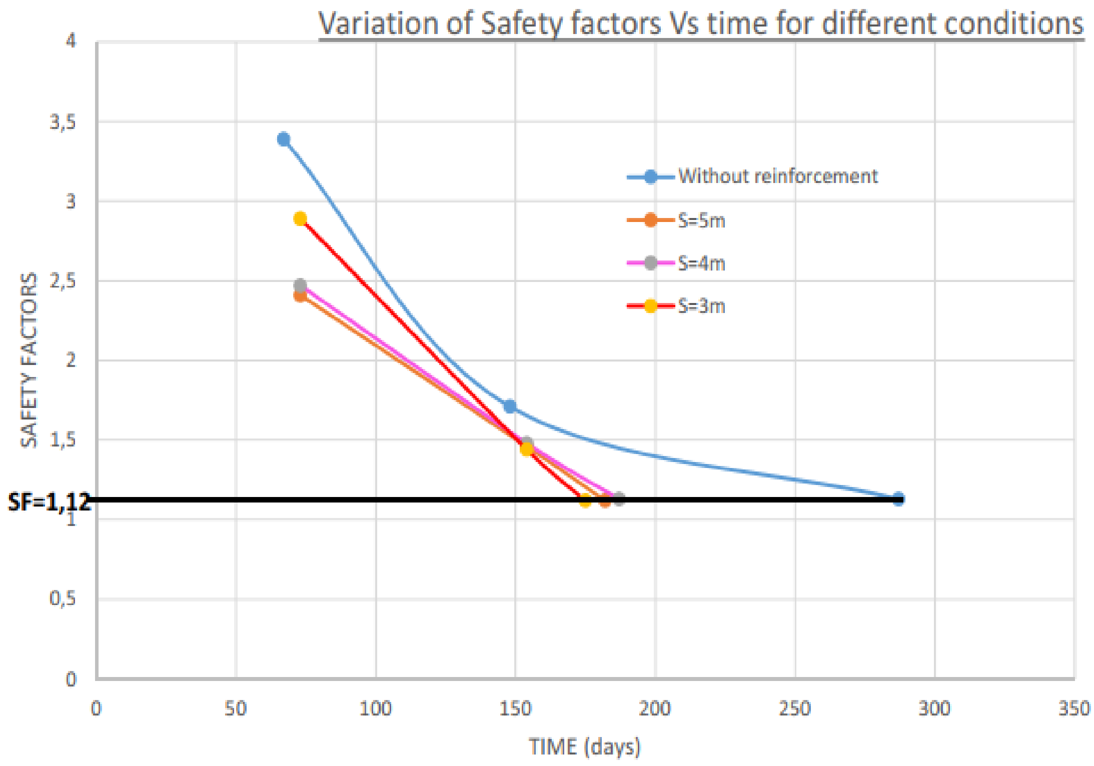


Figure3.22. Variation of Safety factors Vs time for different conditions (Plaxis 2008*)*

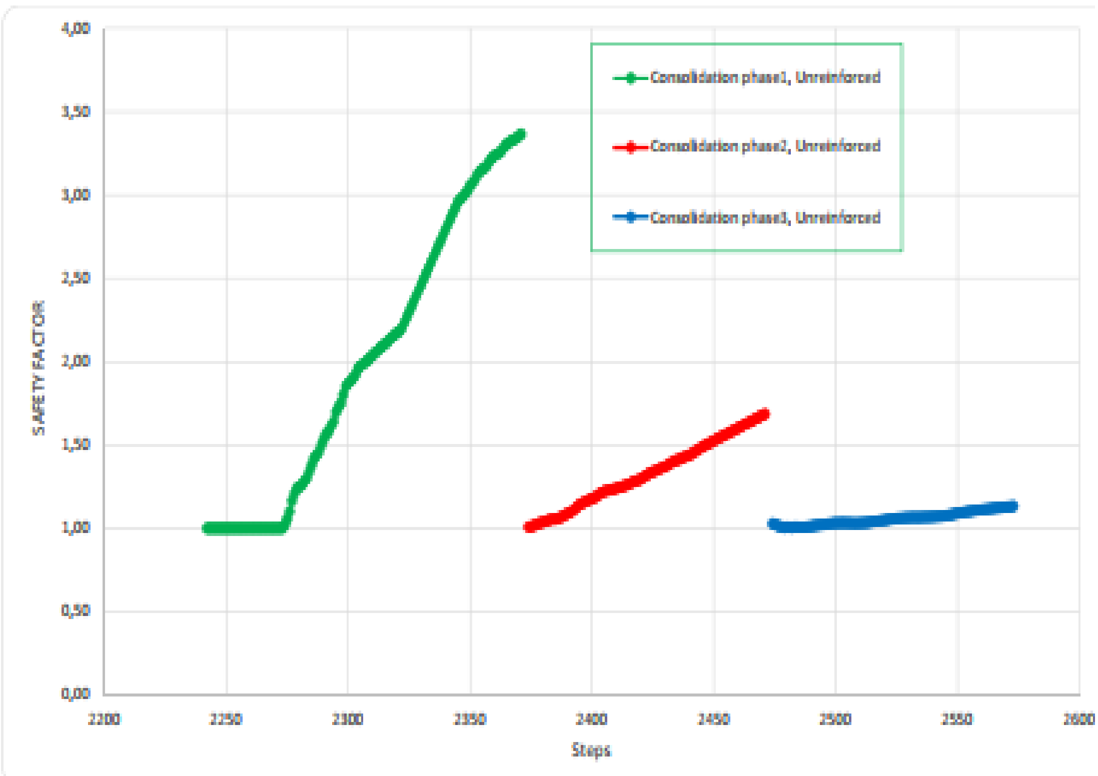


Figure3.23. Safety factor Versus steps of consolidation for unreinforced soil (Plaxis 2008)

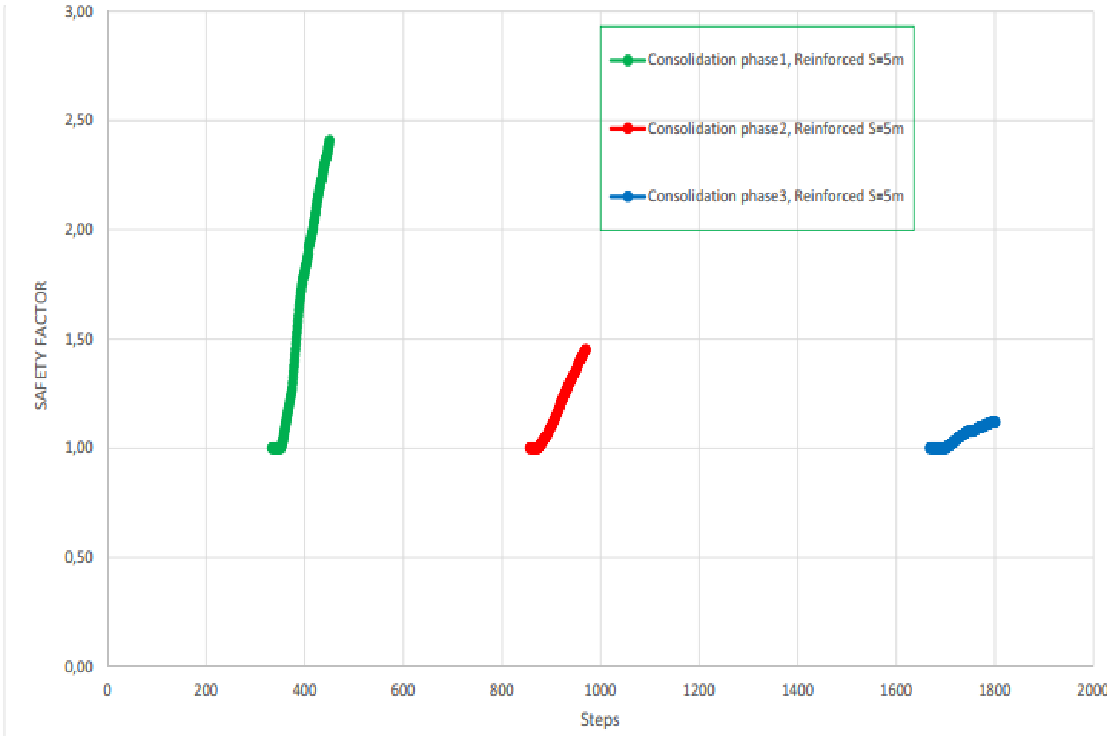


Figure3.24. Safety factor Versus steps of consolidation for reinforced soil (Plaxis 2008)

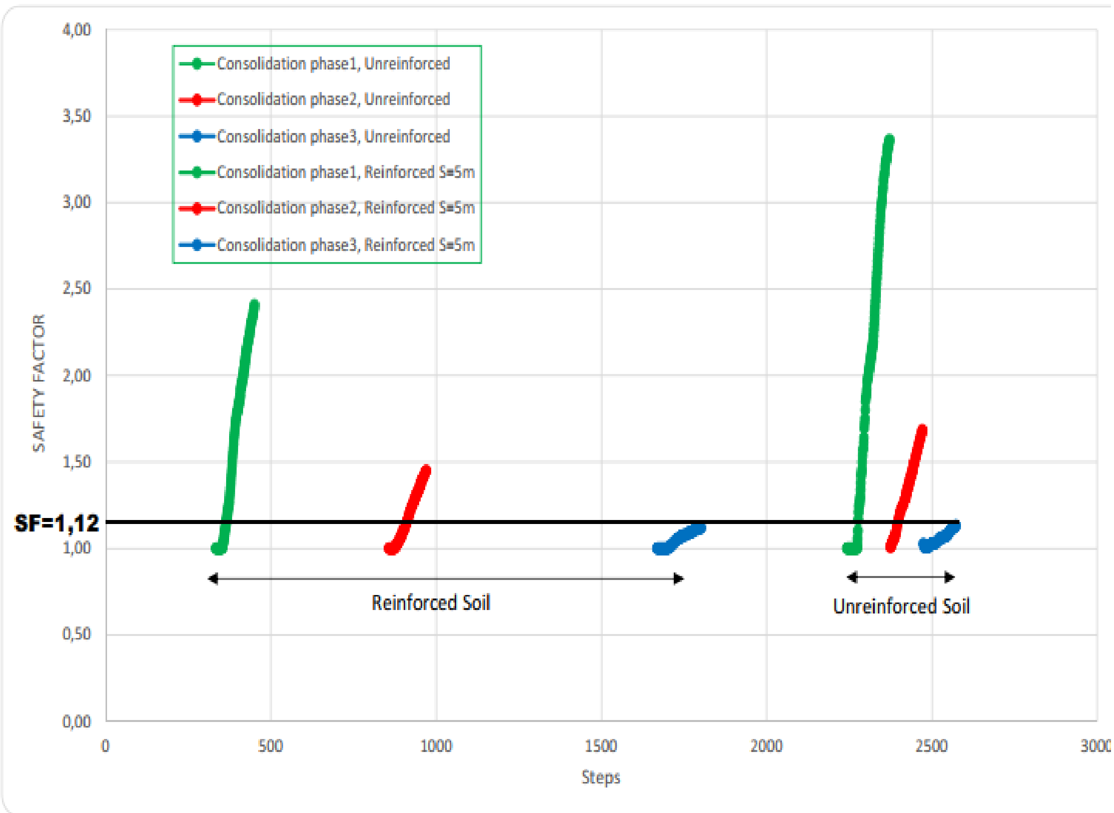


Figure3.25. Safety factor Versus steps of consolidation for unreinforced and reinforced soil (Plaxis 2008)

The failure Surfaces of the slopes for different conditions can also be seen from Figure3.26, Figure3.27, Figure3.28 and Figure3.29.

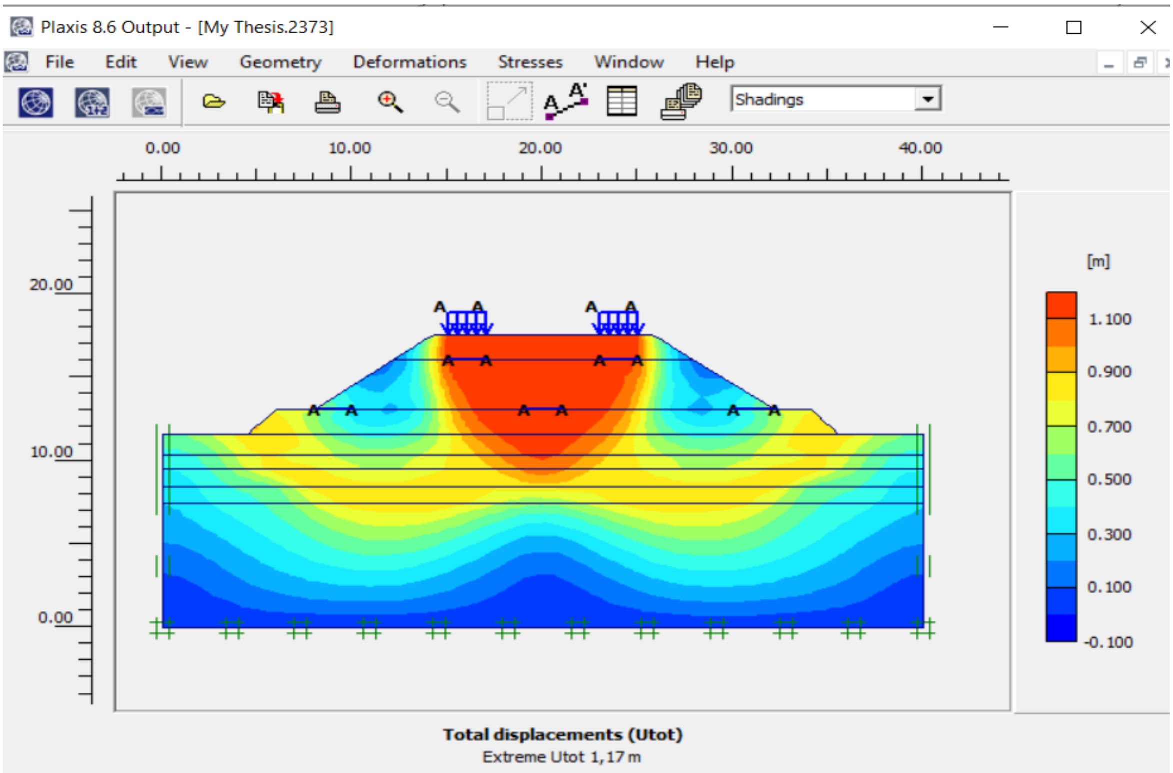


Figure3.26. Failure surface for unreinforced soil (Plaxis 2008)

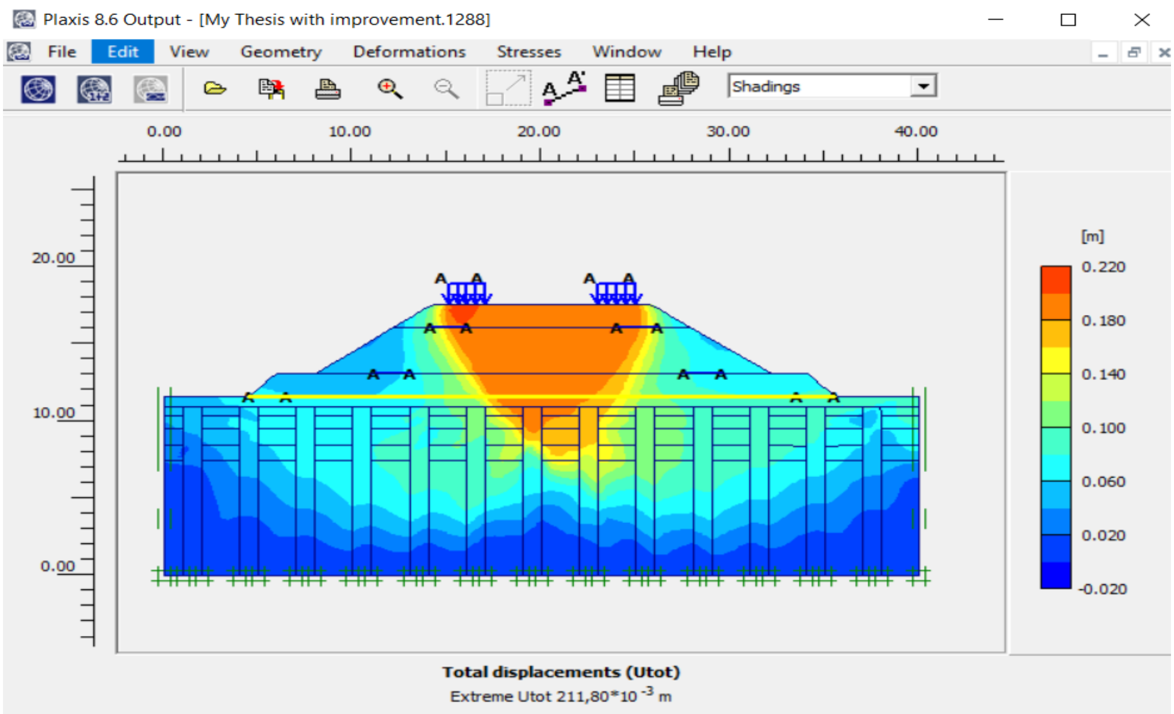


Figure3.27. Failure surface for reinforced soil S=3m (Plaxis 2008)

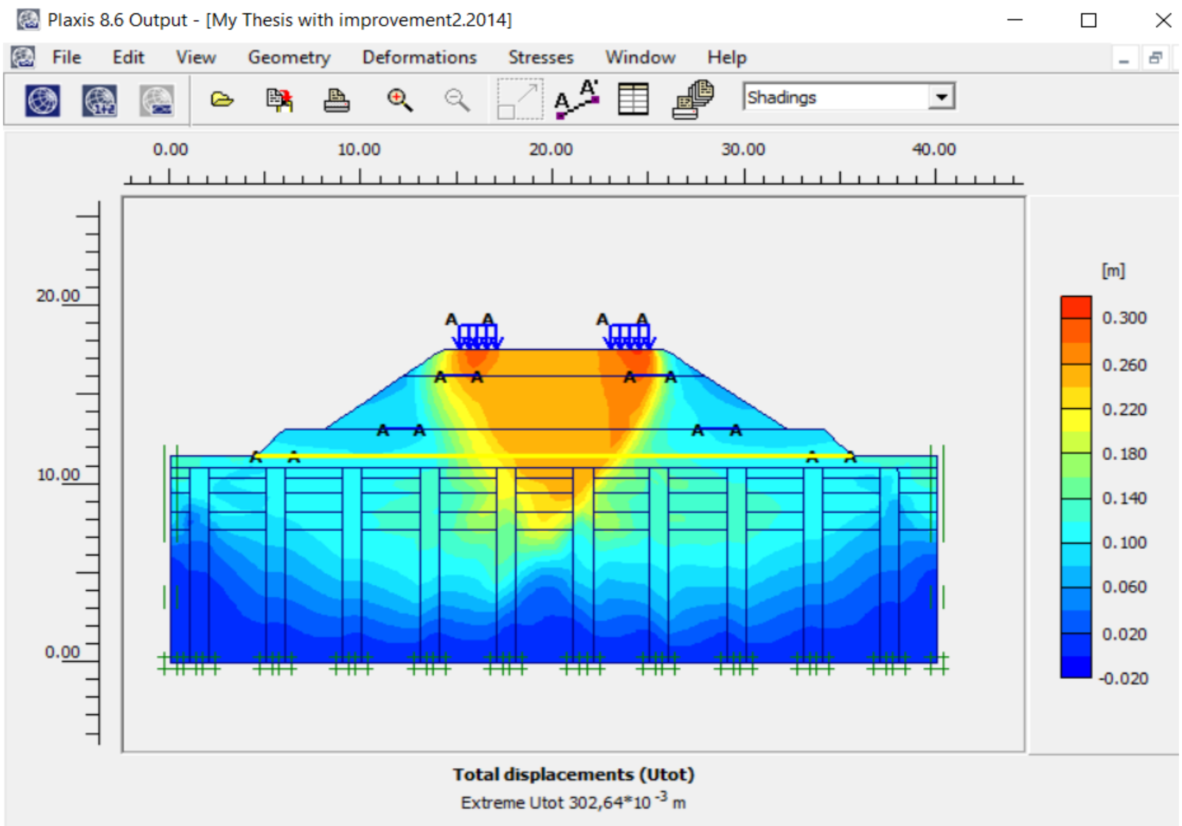


Figure3.28. Failure surface for reinforced soil S=4m (Plaxis 2008)

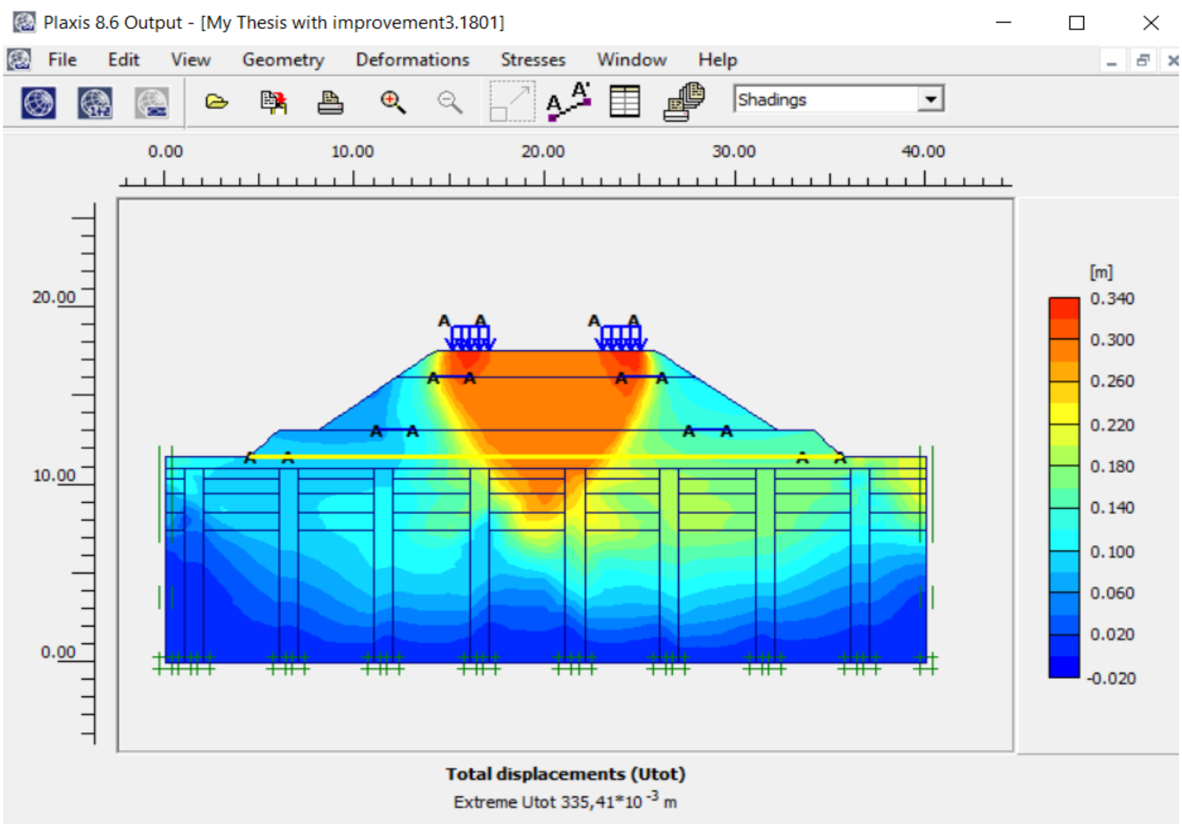


Figure3.29. Failure surface for reinforced soil S=5m (Plaxis 2008)

Table 3.10 gives a summary of the behavior of our embankment under different initial conditions

Table3.10. Summary of the behavior of our embankment under different initial conditions

	Unreinforced soil	Reinforced soil with $S = 5\ m$	Reinforced soil with $S = 4\ m$	Reinforced soil with $S = 3\ m$
Settlement at the end of consolidation (cm)	**47.11**	**22.56**	**20.91**	**17.19**
Consolidation end time (days)	**287**	**182**	**187**	**175**
SF at the end of consolidation	**1.13**	**1.12**	**1.13**	**1.12**
Time gained with treatment, Δt (days)	/	**105**	**100**	**112**

3.5.2.5 Other results from Analysis

Figure3.30, Figure3.31, Figure3.32 and Figure3.33 shows the results of other parameters which helped to better understant the main results presented in the previous parts.

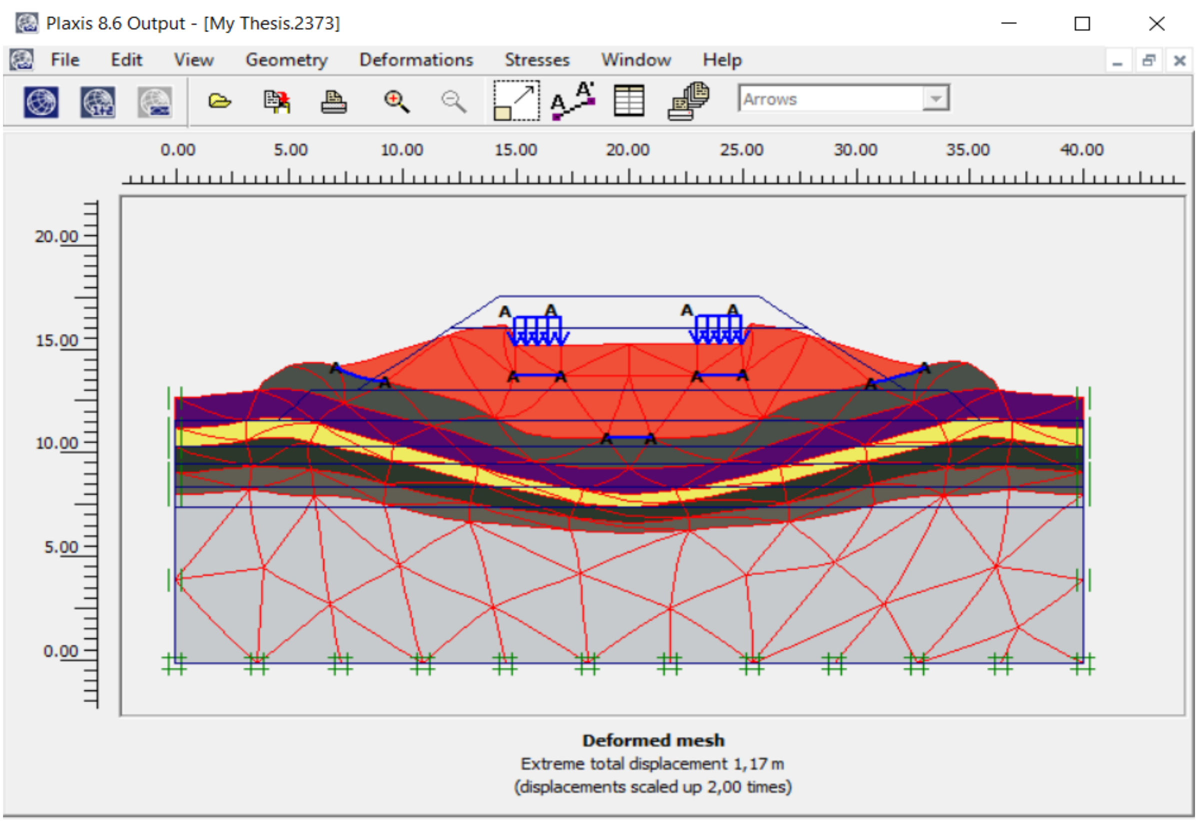


Figure3.30. Deformed mesh in unreinforced soil (Plaxis 2008)

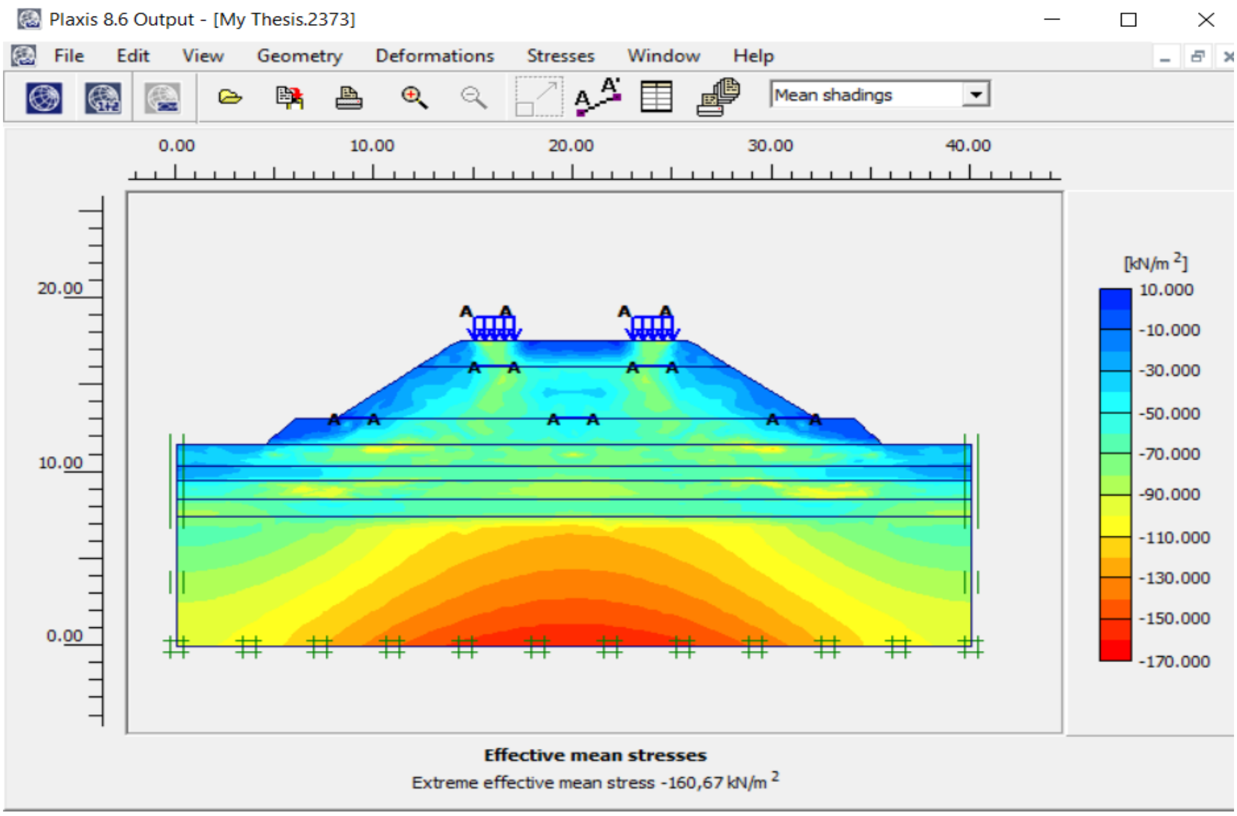


Figure3.31. Effective stress in unreinforced soil (Plaxis 2008)

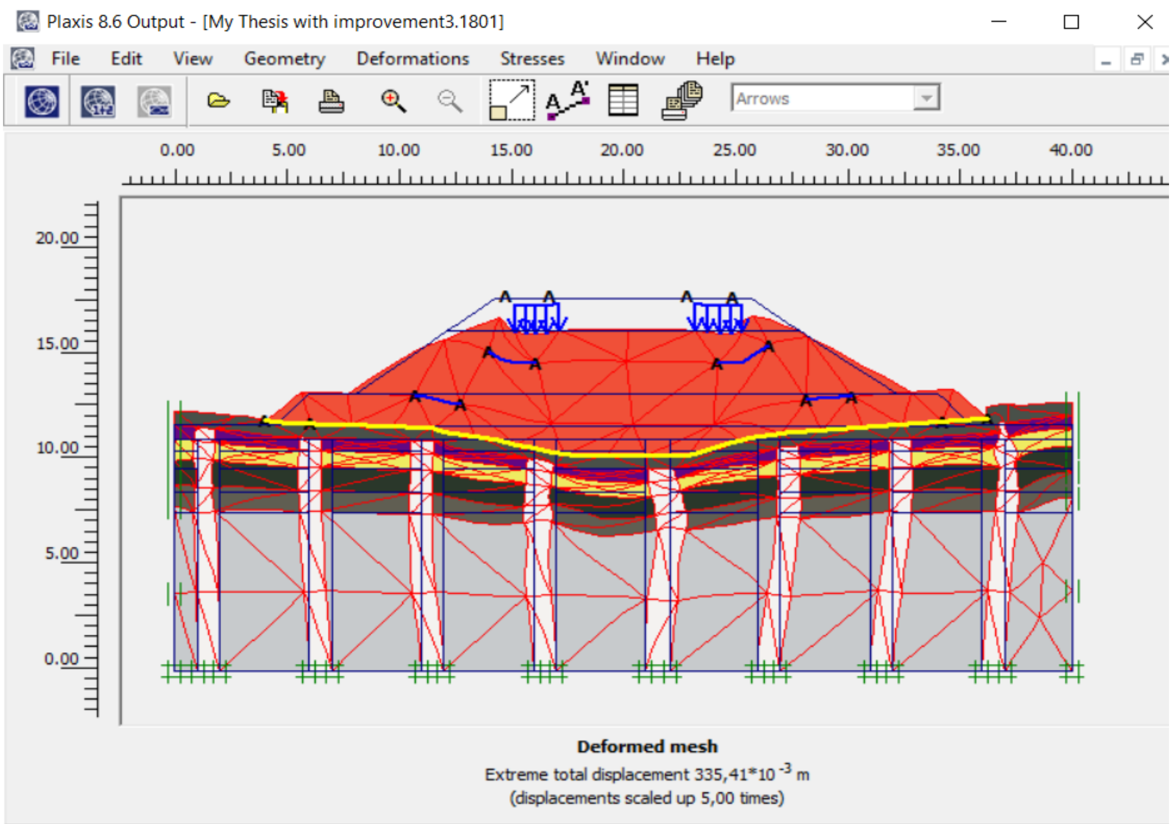


Figure3.32. Deformed mesh in reinforced soil (Plaxis 2008)

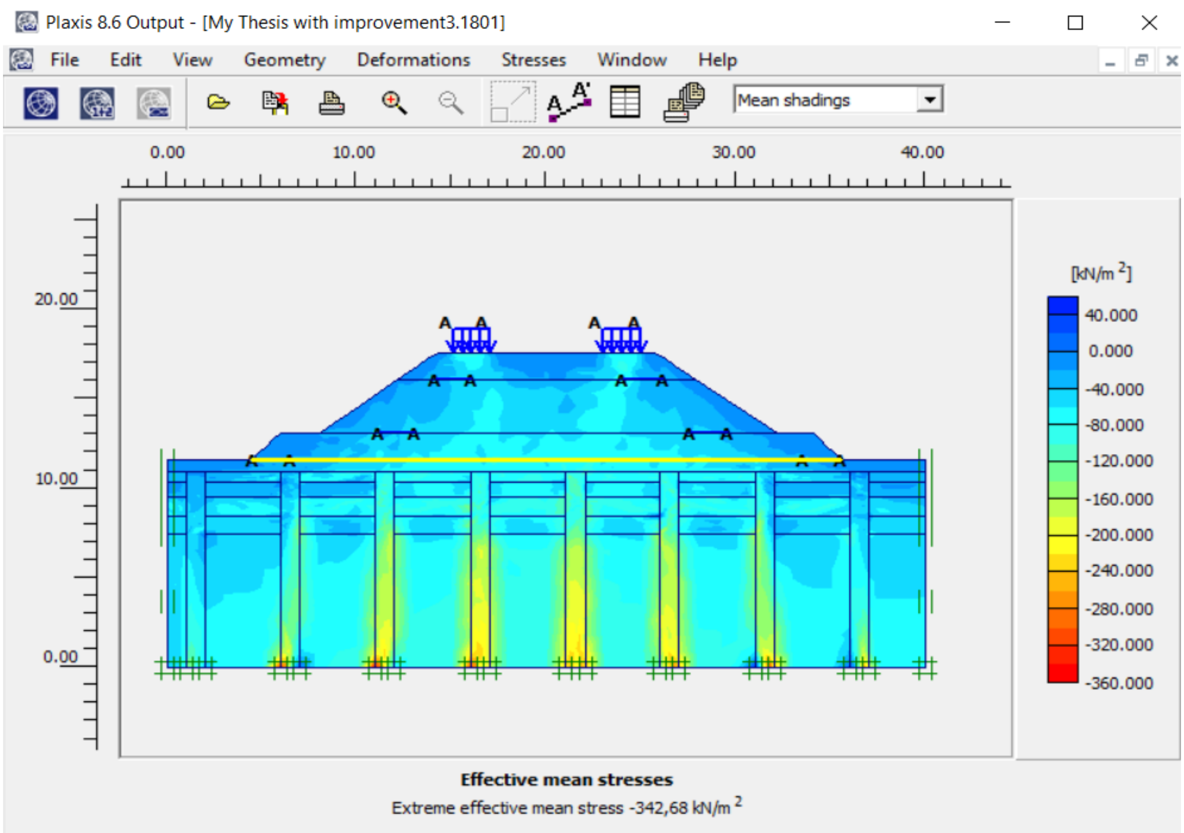


Figure3.33. Effective stress in reinforced soil (Plaxis 2008)

3.6 Cost analysis

In civil engineering practices, the aim always lies in looking for solution to a certain problem, but finding solutions are not usually enough to conclude. We always need to look for the best solution to the problem, this is optimisation. Cost analysis is one of the process to optimise solutions for a problem. In this case, we are going to compare cost of the improvement solution we obtained from our study with the cost of construction of the embankment without foundation soil treatment. But we need first to come out with the cost of our technique.

3.6.1 Quantity evaluation

The solution we propose makes use of mainly three materials, gravel in the class $25/63$,pouzzolana and geotextile of density $400\ g/m^2$. To evaluate the volume of gravel to be used, we start by evaluation from Figure 3.15, the number of columns needed and the volume (cylinder) will be calculated by the formulae stated in chapter 2.

3.6.1.1 Quantity evaluation for reinforcement with columns spaced by 5 m

$N = 480$

We obtain $V = 4\ 147\ m^3$

The surface area of the geotextile used is calculated as the total area of Figure 3.15 as follows

$A = 12\ 000\ m^2$

Similarly, the volume of the pouzzolana(drainage blanket) can be obtained as

$V = 7\ 800\ m^3$

3.6.1.2 Quantity evaluation for reinforcement with columns spaced by 4 m

$N = 800$

We obtain $V = 6\ 912\ m^3$

The quantities of geotextile and pouzzolana needed remain the same irrespective of the spacings

3.6.1.3 Quantity evaluation for reinforcement with columns spaced by 3 m

$N = 1\ 300$

We obtain $V = 11\ 232\ m^3$

3.6.2 Unit prices

The unit prices obtained from the project documents are summarized in Table 3.11

Table3.11. Summary of unit prices (Project documents)

Material	Unit price (FCFA)
Stone column Gravel 25/63 (m^3)	93 000
Soil replacement by Pouzzolana (m^3)	21 000
Geotextile (m^2)	2 600
Embankment material-reddish clay (m^3)	6 500

It should be noted that, the above mentioned unit prices take into account the engines used for the work and the labour wages.

3.6.3 Cost estimate

The cost estimate is evaluated according to the procedure defined in chapter 2.

3.6.3.1 Case with stone columns spacing equal 5 m

Table3.12. Cost estimate of the solution 1 proposed

Material	Unit	Quantity	Unit Price (FCFA	Total Price(FCFA)
Gravel 25/63	m^3	4 147	93 000	385 671 000
Pouzzolana	m^3	7 800	21 000	163 800 000
Geotextile 400	m^2	12 000	2 600	31 200 000
Embankment refill	m^3	677	6 500	4 400 500
Immobilisation of the enterprise due to consolidation time	month	0.67	200 000 000	134 000 000
			TOTAL	7191 500

3.6.3.2 Case with stone columns spacing equal 4 m

Table3.13. Cost estimate of the solution 2 proposed

Material	Unit	Quantity	Unit Price (FCFA	Total Price(FCFA)
Gravel 25/63	m^3	6 912	93 000	642 816 000
Pouzzolana	m^3	7 800	21 000	163 800 000
Geotextile 400	m^2	12 000	2 600	31 200 000
Embankment refill	m^3	628	6 500	4 082 000
Immobilisation of the enterprise due to consolidation time	month	0.83	200 000 000	166 000 000
			TOTAL	1 007 898 000

3.6.3.3 Case with stone columns spacing equal 3 m

Table3.14. Cost estimate of the solution 3 proposed

Material	Unit	Quantity	Unit Price (FCFA	Total Price(FCFA)
Gravel 25/63	m^3	11 232	93 000	1 044 576 000
Pouzzolana	m^3	7 800	21 000	163 800 000
Geotextile 400	m^2	12 000	2 600	31 200 000
Embankment refill	m^3	516	6 500	4 082 000
Immobilisation of the enterprise due to consolidation time	month	0.43	200 000 000	86 000 000
			TOTAL	1 329 658 000

3.6.3.4 Case without reinforcement

Table3.15. Cost estimate of building on initial soil

Material	Unit	Quantity	Unit Price (FCFA	Total Price(FCFA)
Embankment refill	m^3	1 412	6 500	9 178 000
Immobilisation of the enterprise due to consolidation time	month	4.17	200 000 000	834 000 000
			TOTAL	843 178 000

From the previous analysis, the optimal solution is the realisation of stone columns of unit diameter spaced by center to center distances of 5 m. This solution is judged optimal (Table 3.16) because it reduces considerably the time of consolidation, the amount of settlement, assures a good safety factor and relatively low cost compared to the cost of constructing the embankment without treating the foundation soil. The cost of different solutions is summarised in the chart in Figure3.34.

Table3.16. Presentation of factors confirming the efficiency of the solution chosen

	Construction on Unreinforced foundation soil	Construction on Reinforced foundation soil
Cost (FCFA)	843 178 000	719 071 500
Time required for end of consolidation (days)	287	182
Time gain	χ	✓
Money gain	χ	✓

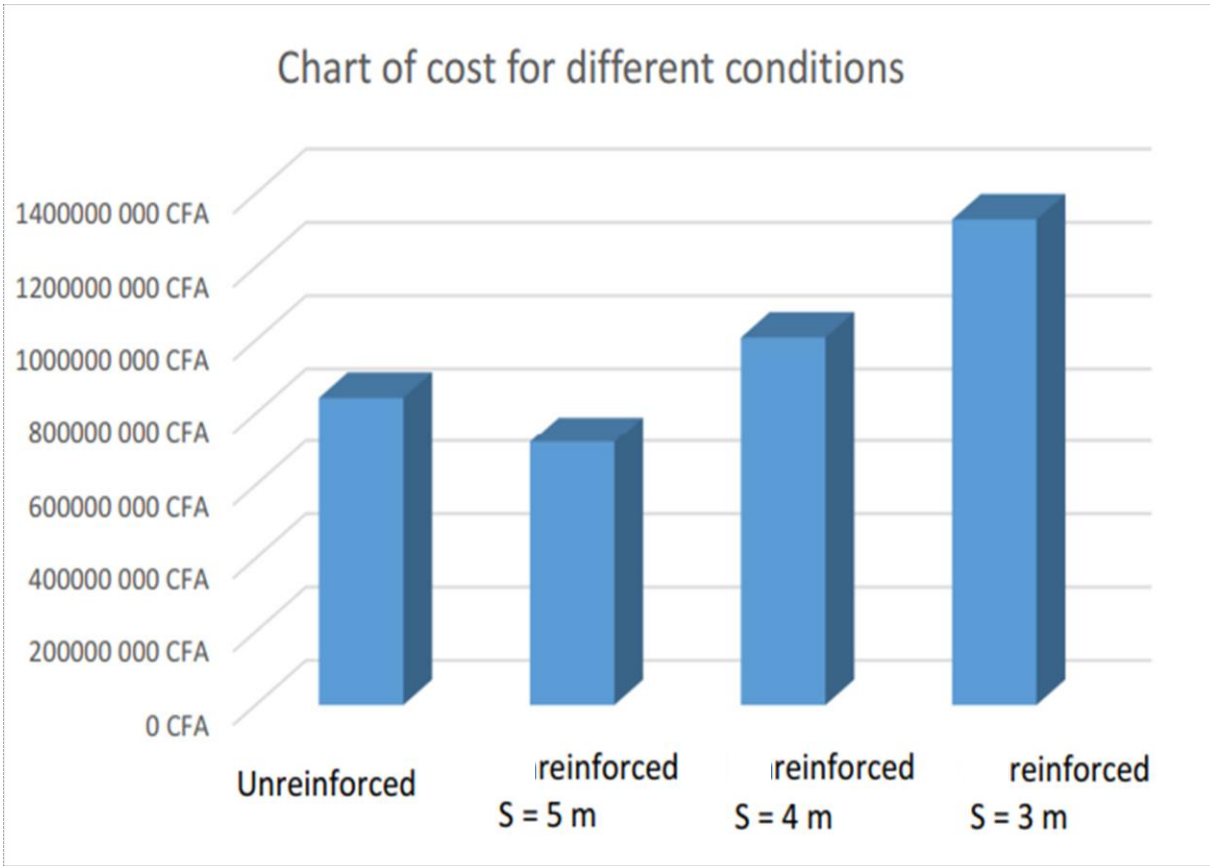


Figure3.34. Chart for the cost for different conditions

Conclusion

This chapter aimed at presenting and analysing the results obtained from our study. It can be observed that, the insitu soil nature and characteristics could not effectively allow the construction of the embankment due to the fact that it would allow settlements over a long period of time. The remedy to this problem was the realisation of stone columns coupled to a geotextile at the base of the embankment. The results of this treatment applied to our case study allines with the predictions in the literature review (Chapter1). The stone columns plaid a very great role in reducing considerably the settlement and consolidation end time thereby assuring the stability of our embankment in a brief time. Practically, the fact that our embankment attains stability relatively quickly with stone columns treatment is of great advantage because it permits the project's time frame to be respected.

GENERAL CONCLUSION PERSPECTIVES

The present study had as title "**Stability of a high embankment on soft soil along the BANGANGTE-FOUMBOT-BAMENDJING-GALIM road**". The main objective was to guarantee the stability of the embankment within considerable timeframe by designing an efficient improvement technique.

To attain this objective, the work was based on three main parts that are; the state of art on soft soils, embankments, the overview on stability and ground improvement techniques, methodology of work and the presentation of results of the analysis and interpretations.

The approach adopted was to go from the analysis of the model without ground improvement to that of the model with ground improvement. It was seen that, in the case where the embankment is built on the soil without improvement, the embankment attained stability 287 days after the beginning of the construction. When the embankment is built on the improved soil, the embankment attained stability 182 days, 187 days and 175 days after the beginning of the construction respectively for stone columns spacings of 5 m, 4 m and 3 m. This gap of time loss (between construction with and without improvement), of not less than 3.5 months represents about 1.2 times the cost of the improvement solution. At the end of this work, it can be concluded that the results obtained from ground improvement with stone columns spaced by 5 m are satisfactory since it reduces by half the total settlement of the embankment and reduces by 3.5 months the consolidation end time of the embankment thereby limiting the risks of exceeding the project's timeframe. It also encourages the use of local materials.

As perspective, further analysis can be the monitoring of the stone column supported embankment and interpretation of recorded values of settlement in time with specify methods (settlement rods for example) to confirm the results obtained from this numerical analysis.

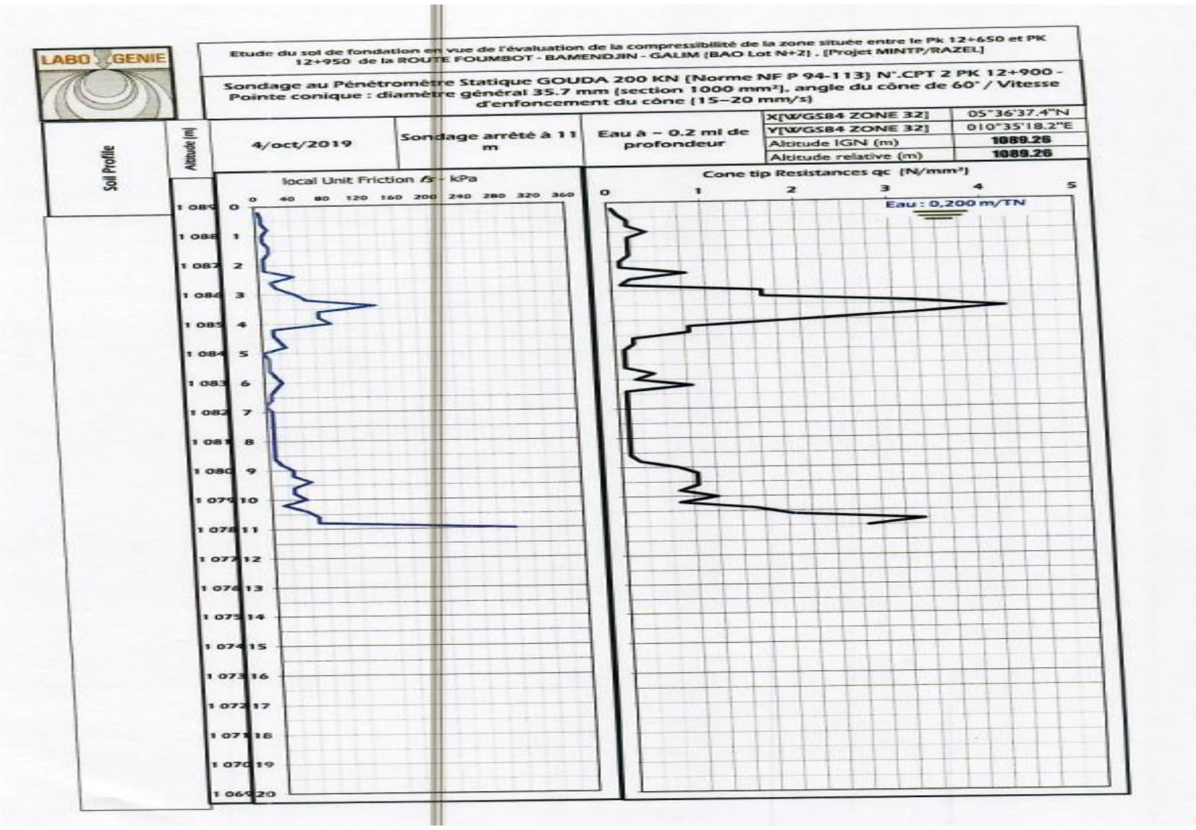
APPENDIX

Appendix 1; Compressibility tests on initial soil samples

Modèles géotechniques au droit des profils étudiés

Projet	remblai (m)	km	formation	z (m)	e0	Cc	Cs	Cu (kPa)	σp' mesuré (kPa)	σp' recalé (kPa)	Cv_Labo m²/s	Cv_retenu m²/s	Couche comp (m)	Temps cons mois	Degré consol
Etude du sol de fondation en vue de l'évaluation de la compressibilité de la zone située entre le Pk 12+650 et PK 12+950 de la ROUTE FOUMBOT - BAMENDJIN - GALIM (BAO Lot N+2) . [Projet MINTP/RAZEL]	2.440	12.934	Argile [Vase (PEI_12)]	3.00 - 3.50 m	1.74060319	0.39392594	0.231837419	25.30	21.95	72.29	0.00100567	0.0100567	0.20 à 6.20	**6.5 mois**	99%
	2.440	12.934	Argile [Jaunâtre (PEI_11)]	2.00 - 2.70 m	1.41757537	0.31065794	0.119587901	26.50	44.80	75.71	0.00241484	0.0241484	0.20 à 6.20	**2.5 mois**	99%
	2.280	12.900	Argile [Noirâtre (PEI_13)	2.00 - 3.00 m	1.37623123	0.28895599	0.120112612	19.00	39.82	54.29	0.00163361	0.0163361	4.20 à 10.40	**3.5 mois**	99%
	2.280	12.900	Limon Argileux sableux [Jaunâtre (PEI_14)	1.50 - 2.30 m	1.23615189	0.22090911	0.090638465	19.80	29.69	56.57	0.00419508	0.0419508	4.20 à 10.40	**1.5 mois**	99%
	2.100	12.825	Sable [Noirâtre (PEI_15)	1.60 - 2.40 m	0.95383517	0.19752481	0.063940981	33.10	48.32	94.57	0.00330561	0.0330561	0.20 à 02.80	**0.5 mois**	99%
	2.100	12.825	Sable [pouzzolanique (PEI_16)	2.00 - 2.80 m	0.93467599	0.20882721	0.104743684	0.00	32.34	0.00	0.00299111	0.0299111	0.20 à 02.80	**0.5 mois**	99%
	2.560	12.750	Argile [Jaunâtre (PEI_17)	1.00 - 1.80 m	Essais de compressibilité en cours d'exécution			12.90	0.00	36.86	Essais de compressibilité en cours d'exécution				
	2.560	12.750	Argile [Jaunâtre (PEI_18)	1.20 - 2.00 m				15.70	0.00	44.86					
	5.120	12.700	Limon Argileux sableux [noirâtre (PEI_19)]	2.00 - 2.80 m				24.50	0.00	70.00					
	5.120	12.700	Sable [noirâtre (PEI_20)] [Altitude :+1087	1.50 - 2.00 m				12.10	0.00	34.57					

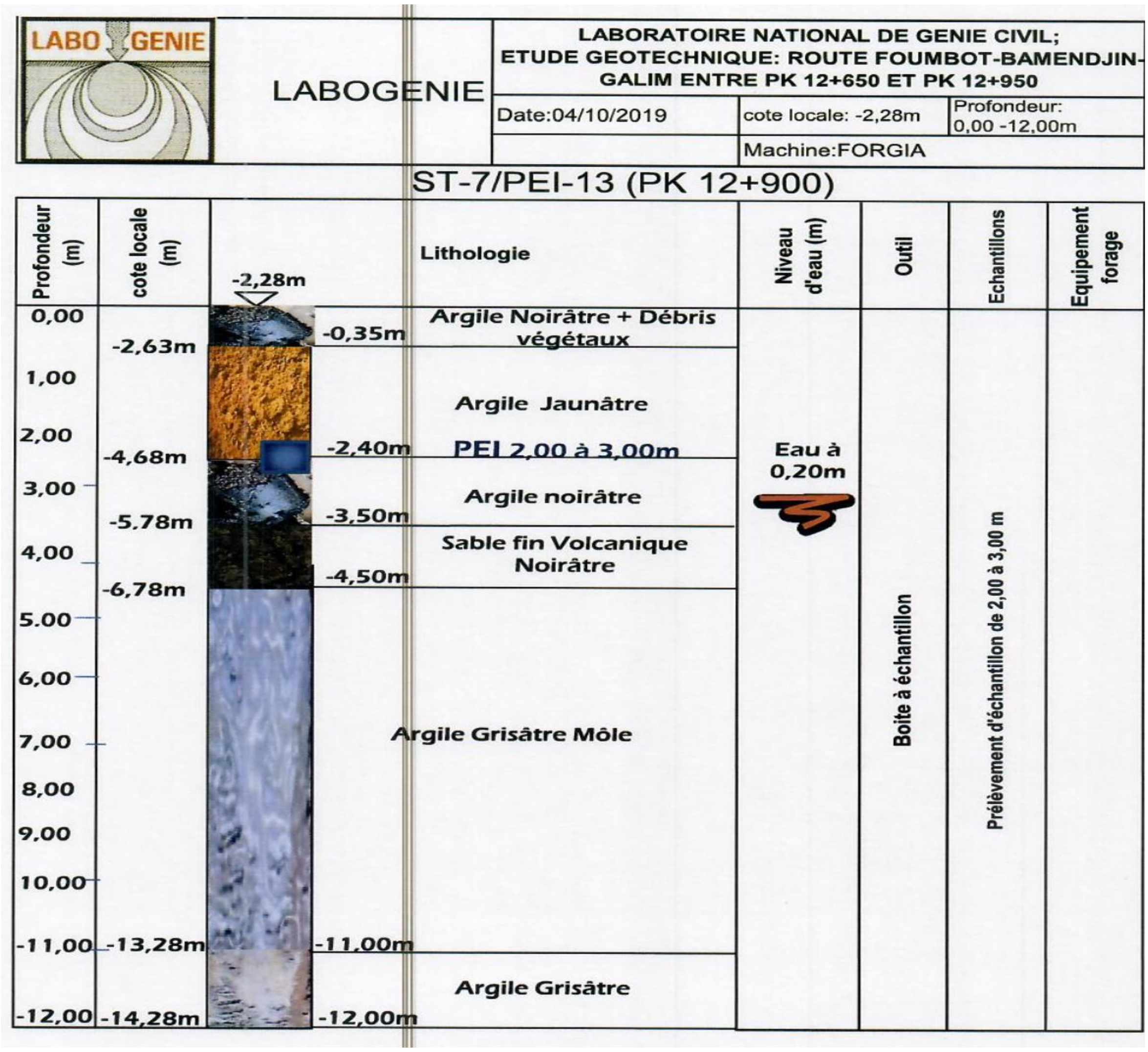
Appendix 2; CPT test graphs



Appendix 3; CPT test results

CPT 2 PK 12+900/ /	Cotes des tranches homogènes, m		Frottement unitaire local	Résistances en pointe	Friction ratio				Etude du sol de fondation en vue de l'évaluation de la compressibilité de la zone située entre le Pk 12+650 et PK 12+950 de la ROUTE FOUMBOT - BAMENDJIN - GALIM (BAO Lot N+2) , [Projet MINTP/RAZEL]		
PROF./TN, m	Sup.	Inf.	fs moy (kPa)	qc (Mpa)	$R_f = fs/qt$ [%]	SBTn [(Robertson et al.]	Cu, Max[8.32 N_{60}:(qc-σ_{v0})/Nk] [kPa]	N_{60}	Nature	Module élasticité E (Mpa)	Rapport Em/qc
3.00	1089.26	1086.26	10.41	0.31	0.048%	2	21.14	1	Argile molle	1.392	4.5
3.40	1 086.26	1085.86	71.44	2.40	0.329%	4	169.00	5	Argile molle	10.800	4.5
4.20	1 085.86	1085.06	43.38	2.55	0.200%	5		5	sable Peu consistant	5.100	2
4.60	1 085.06	1084.66	12.76	0.50	0.059%	3	33.78	1	Argile molle	2.250	4.5
9.00	1 084.66	1080.26	8.35	0.18	0.039%	2	9.85	0	Argile molle	0.818	4.5
10.40	1 080.26	1078.86	24.06	0.86	0.111%	3	55.86	2	Argile molle	3.857	4.5
10.80	1 078.86	1078.46	40.83	2.50	0.189%	4	170.97	6	Argile Peu consistant	11.250	4.5
11.00	1 078.46	1078.26	214.33	2.60	0.993%	3	169.81	7	Argile Peu consistant	11.700	4.5

Appendix 4; Stratigraphy of the soil from CPT



Appendix 5; Oedometer test results

LABO GENIE

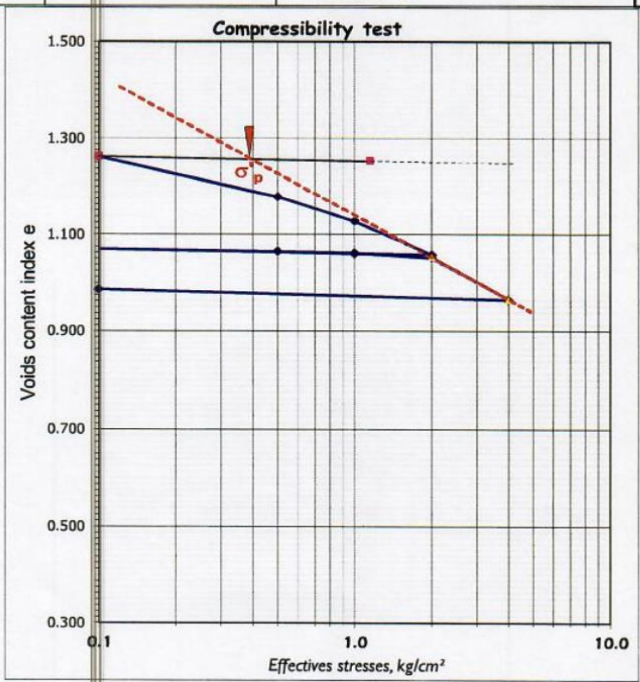
CONSOLIDATION ASTM D2435

Essai œdométrique - XP P94-090-1 / Consolidation ASTM D2435

	23/10/2019	Project	Zone marécageuge du Pk 12+900 (Tronçon Foumbot - Bamendjin)	
Boring, Sample No	6			
Depth	2.00 - 3.00 m	Mould	Oedometer	
Soil description	Argile [Noirâtre (PEI_13)	S, cm²	38.5	E_{oedo} [MPa]
Final weight of water, Pef	35.1	h_p	0.924	4.7
Spécific, gravity γS	2.249	Initial void ratio e_0	1.376	a_v (1/Mpa)
Dry material, Ps	80	Ho, mm	21.96	0.43492
Final moisture content, Wf	43.9%	Initial moisture content, Wi		46.8%

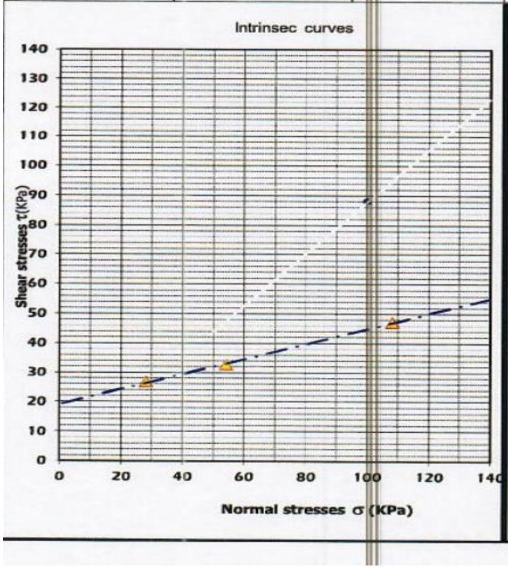
Dates	Pressure, kg/cm²	Settlement H(cm)	Ho-H	e
12-oct.-19	0.100	0.106	2.091	1.262
13-oct.-19	0.300	0.144	2.052	1.205
13-oct.-19	0.500	0.183	2.013	1.178
14-oct.-19	1.000	0.230	1.966	1.127
15-oct.-19	2.000	0.294	1.902	1.058
16-oct.-19	1.000	0.292	1.905	1.061
17-oct.-19	0.500	0.288	1.908	1.064
18-oct.-19	0.100	0.284	1.913	1.069
19-oct.-19	0.500	0.288	1.908	1.064
18-oct.-19	1.000	0.291	1.906	1.062
19-oct.-19	2.000	0.300	1.897	1.052
20-oct.-19	4.000	0.380	1.816	0.965
21-oct.-19	0.100	0.360	1.836	0.987
22-oct.-19				

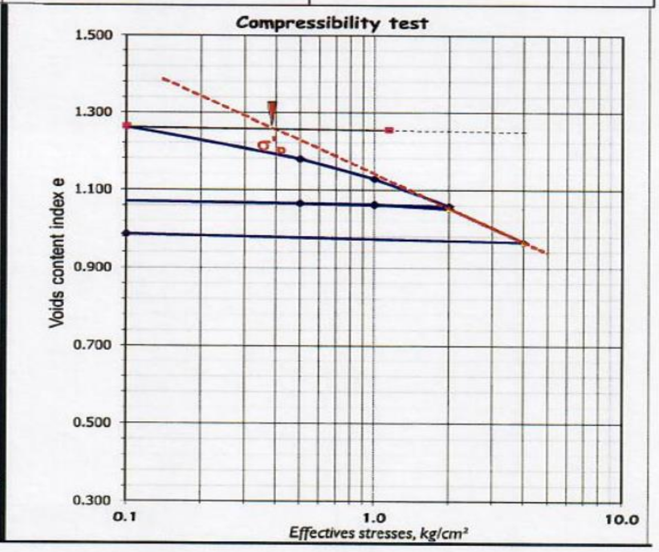
Compression Index Cc	**0.289**	Swelling Index **Cs**	**0.12011**	Perméability Kv (cm/s)
Preconsolidation pressure σ_p, Kg/cm²	**0.398**	Consolidation coefficient **Cv, cm²/s**	**1.6336E-03**	**2.68E-06**



Appendix 6; Direct shear test results

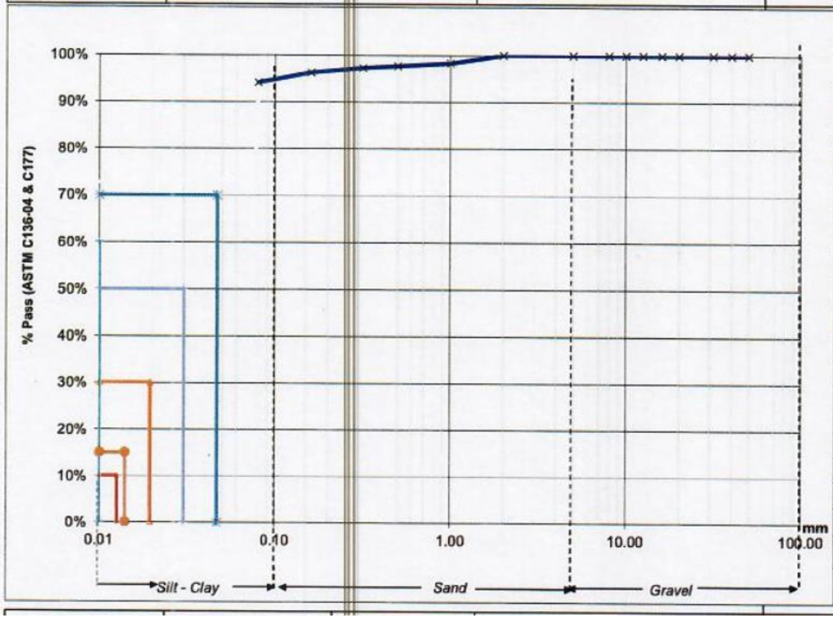
Direct shear test	Undrained shear strength, Cu	19.0 kPa	Compression Index Cc	0.28896	Swelling Index Cs	0.1201
Final Moisture content =71.35%	Internal friction angle φu	14.4 °	Preconsolidation pressure σ'p, Kg/cm²	0.39821	Initial void ratio e0	1.376





Appendix 7; Sieve analysis test results

LABO GENIE	Zone marécageuge du Pk 12+900 (Tronçon Foumbot - Bamendjin)		Argile [Noirâtre (PEI_13) [Altitude ;+1087.m] Sédiment très bien classé	depth recovery / TGS	2.00 - 3.00 m
	Gravel (%)=	0.00%	Sand (%)= 5.90%	Clay (%)=	94.10%
	Bulk density γ_h	1.625			
	Dry density γ_d	1.107	Liquid Limit 55.8	Specific gravity	Classification
23-oct.-19	Moisture content W_{nat}	46.8%	Plasticity Index 25.46	2.249 T/m3	A-7-5(17) [SM_Silty Sand]



Sieves (mm)	% passing
0.08	94.1%
0.107	94.3%
0.1	94.8%
0.16	96.2%
0.315	97.3%
0.425	97.6%
1	98.4%
2	100.0%
5	100.0%
8	100.0%
10	100.0%
16	100.0%
20	100.0%
31.5	100.0%
40	100.0%
50	100%

Médiane : 0.030 mm
Moyenne : 0.036 mm
Coef. d'Asymétrie : 0.299
Coefficient d'Acuité : 1.121
S0 Indice de tri Trask : 1.737

Appendix 8; Atterberg limits tests results

LABO GENIE

ATTERBERG LIMITS ASTM D4318-00

Project	Zone marécageuge du Pk 12+900 (Tronçon Foumbot - Bamendjin)		
Dossier No			
Sample No		**Depth recovery**	2.00 - 3.00 m
Description of the sample	Argile [Noirâtre (PEI_13)		
Date of test	16-oct.-19		

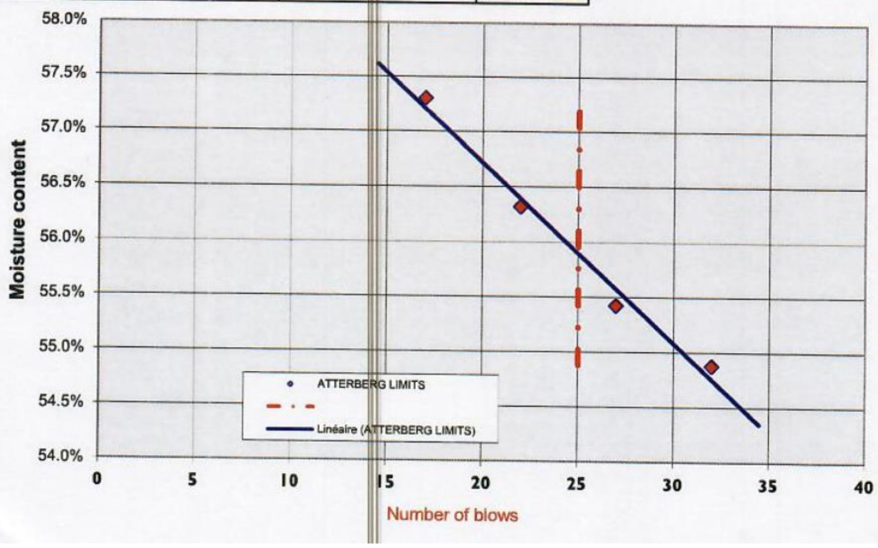
LIQUID TEST

Number of blows	17	22	27	32	
Tare No.	3	U	Z	I	
Total weight of bulk specimen	27.40	26.90	26.20	26.00	
Total weight of dry specimen	22.30	22.00	21.60	21.50	
Weight of tare	13.40	13.30	13.30	13.30	
Weight of water	5.100	4.900	4.600	4.500	
Real weight of dry specimen	8.900	8.700	8.300	8.200	
Moisture content	57.30%	56.32%	55.42%	54.878%	

PLASTICITY LIMIT TEST

Tare No.	N	T 8
Total weight of bulk specimen	32.80	30.50
Total weight of dry specimen	32.10	29.80
Weight of tare	29.80	27.50
Real weight of dry specimen	2.30	2.30
Weight of water	0.70	0.70
Moisture content	30.4%	30.4%

Liquid Limit	55.9%
Plastic Limit	30.4%
Plasticity Index	25.5%



Appendix 9; Summary of order of magnitude of major soft soil properties.

Properties	Peats	Organic soils	Vases	Soft clays
Water content (ω,%)	200 - 1000	100 - 200	60 - 150	30 – 100
Void ratio (e)	3 - 10	2 - 3	1.5 - 3	1.2 – 2
Porosity (n)	0.75 – 0.9	0.7 – 0.8	0.6 – 0.75	0.55 – 0.7
Compressibility $C_c/(1+e_0)$	0.4 - 0.8	0.2 – 0.35	0.25 – 0.4	0.15 – 0.3
Shrinkage index Coe	0.02C_c	0.03 - 0.05C_c		
Coefficient of permeability k(ms^{-1})	10^{-6} - 10^{-9}			10^{-6} - 10^{-11}
Coefficient of consolidation $C_v(m^2s^{-1})$	10^{-6} - 10^{-8}			10^{-6} - 10^{-9}
Undrained shear strength $C_u(kPa)$	10 – 50			
Rate of variation of C_u C_u: $\lambda C_u = \Delta C_u/\Delta\sigma'$	0.5	0.2 – 0.3		
Dry specific weight $\gamma_d(tm^{-3})$	0.1 – 0.5	0.5 - 1	0.7 – 1.5	1 – 1.6
Bulk specific weight $\gamma_{sat}(tm^{-3})$	1.4 - 2	2 – 2.6	2.4 – 2.7	2.6 – 2.7

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