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UNIVERSITÀ DEGLI STUDI DI PADOVA

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DEPARTMENT OF CIVIL, ARCHITECTURAL AND ENVIRONMENTAL ENGINEERING

STABILITY OF CUT SLOPE IN A POZZOLANIC DEGRADED ROCK MASS ALONG THE BANGANGTE-FOUMBOT-BAMENDJING-GALIM ROADWAY

A thesis submitted in partial fulfilment of the requirements for the degree

Of Master of Engineering (MEng) in Civil Engineering

Curriculum: **Geotechnical Engineering**

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DEDICATION

To

My beloved parents, ***MABAPGUAP Daniel*** *and* ***MAFO Sylvie,***

No words would suffice to express my gratitude for your sacrifices and love, bless you.

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I would like to thank first of all the Lord GOD Almighty for the grace and the favour that he granted me to make these studies and to complete this thesis, may the honour be given to Him.

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LIST OF ABREVIATIONS AND SYMBOLS

ABREVIATIONS

AASHTO	American Association of State Highway and Transportation Officials
ADEME	Agence De l'Environnement et de la Maitrise de l'Energie
ASTM	American Society for Testing and Material
BS	Bishop Simplified
CO_2	Carbonate dioxide
DA	Design Approach
DWG	Drawing File
DXF	Drawing interchange File
EC	Eurocode
EN	European Norm
FEM	Finite Element Method
FHWA	Federal Highway Administrator
FoS	Factor of Safety
JS	Janbu Simplified
LEM	Limit Equilibrium Method
M-P	Morgenstern and Price
PNDP	Programme National de Développement Participatif
SSR	Shear Strength reduction
USCS	Unified Soil Classification System

SYMBOLS

c'	Effective cohesion
c_a	Apparent cohesion
C_c	Coefficient of curvature
C_u	Coefficient of uniformity
DS	Direct shear test
E	Young Modulus
e_0	Void ratio
EA	Axial stiffness
EI	Flexural stiffness
N	Normal force
N'	Effective normal force
R_d	Design force resistance
s	Suction
S_d	Destabilizing force
S_m	Mobilized shear resistance
S_r	Saturation degree
S_{soil}	Soil shear resistance
T	Driving force
Tx	Triaxial test
u_a	Air pore pressure
u_w	Water pore pressure
ν	Poisson ratio
W	Slice weight
β	Slope angle
γ_M	Partial factor of material
γ'	Effective unit weight

γ_G	Partial factor for permanent actions
γ_{sat}	Unit weight of saturated soil
γ_{soil}	Soil unit weight
γ_w	Unit weight of water
θ_w	Volumetric water content
σ	Total stress
σ'	Effective normal stress
τ	Shear resistance
φ	Effective soil friction angle
φ'_d	Design value of internal friction angle
φ'_K	Characteristic value of internal friction angle
φ^b	Slope of the plot of matrix suction versus the shear stress line
ψ	Dilatancy angle

ABSTRACT

The main objective of this work is to propose a technical and economic solution for the stabilization of an excavated slope in a pozzolan deposit over 4.6 km of the Bangangté-Foumbot-Bamendjing-Galim roadway. To achieve this aim, 3 steps are developed. The first step which is the analysis of the 3 slope cases of pozzolan present on the site through numerical analysis methods such as limit equilibrium via the SLOPE/W product of GeoStudio R2 2018 and finite element analysis via PLAXIS 2D. The second step is the analysis of the most unstable slope reinforced by the geocell slope protection, shotcrete and soil nailing and in the third step a comparative study of methods based on technical, economic and environmental criteria. From this methodology, it emerged that although slightly divergent on the safety factor values, the analyses via LEM and FEM unanimously proved the instability of the slope with the highest gradient with a respective safety factor of 0.899 and 0.711 showing a superficial sliding surface that progresses with time towards depth. Subsequently, the analysis with reinforcement showed that all the three remedial measures directly applied to the critical zone have effectively stabilized the pozzolan by 33.4%, 38.71% and 39% for respectively geocell, shotcrete and soil nail. Finally, the comparative study revealed that technically the last two methods, which require extensive use of machinery are no time consuming while Geocell uses less machine but takes a lot of time due to great amount of manpower. Also, in financial terms, the use of Geocell instead of soil nailing and shotcrete reduces the construction costs of respectively 35.65% and 47.94%. This method has also proved to be less polluting than others. Therefore, from the previous benefits of geocell on soil nail and shotcrete, it has been chosen as an efficient method to stabilize the pozzolan cut slope.

Key words: Slope stability - Pozzolan - LEM - FEM - Stabilisation technique

RESUME

L'objectif principal de ce travail est de proposer une solution technique et économique pour la stabilisation d'un talus dans un dépôt de pouzzolane sur 4.6 km du tronçon routier Bangangté-Foumbot-Bamendjing-Galim. Pour y parvenir, 3 étapes sont développées. La 1ère étant d'analyser les 3 cas pente de pouzzolane présents sur le site par 2 méthodes d'analyse numérique que sont les équilibres limite via SLOPE/W de GeoStudio R2 2018 et des éléments finis via PLAXIS 2D. La 2e étape concerne l'analyse de la pente la plus instable renforcée par les géocellules, le béton projeté et le clouage des sols et en 3e étape une étude comparative des précédentes méthodes basée sur les critères technique, économique et environnemental. De cette méthodologie, il ressort que bien que légèrement divergentes sur les valeurs du facteur de sécurité, les analyses via LEM et FEM ont unanimement prouvé l'instabilité du talus ayant le gradient le plus élevé avec un facteur de sécurité respectif de 0.899 et 0.711 pour une surface de glissement superficielle progressant avec le temps vers la profondeur. Par la suite, l'analyse avec renfort a dévoilé que les trois mesures correctives directement appliquées sur la zone critique ont effectivement amélioré la stabilité de la pouzzolane de 33.4%, 38.71% et 39% pour respectivement les géocellules, le béton projeté et le clouage du sol. Enfin, l'étude comparative a révélé que, techniquement, les deux dernières méthodes, qui sollicitent grandement l'utilisation de machines ne nécessitent que peu de temps pour l'exécution, tandis que la méthode avec les géocellules utilise moins de machines et demande beaucoup de temps en raison de la haute intensité en main-d'œuvre nécessaire pour son installation. De même, du point de vue financier, l'utilisation des géocellules au lieu du béton projeté et du clouage au sol réduit les coûts de construction de 47.94% et 35.65% respectivement. Cette méthode est également de très loin, moins polluante que les autres. Par conséquent, à partir des précédents avantages des géocellules sur le clouage du sol et le béton projeté, il a été choisi comme une méthode efficace pour stabiliser le talus de pouzzolane.

Mots clés : Stabilité des talus - Pouzzolane – Equilibre limite – Elément finis - Technique de stabilisation

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GENERAL INTRODUCTION

The development of road network is one of the principal goals for the successful implementation of Cameroon's on-going economic emergence in 2035. The Cameroonian Government has therefore recently embarked on several road projects involving the rehabilitation, reconstruction and upgrading of many existing roads, as well as the construction of new ones. It should be noted that a great majority of these road projects are located in rugged terrains which demand a large number of fills and excavation to be made in order to achieve acceptable grades.

Cutting natural slopes to make way for new road infrastructure alters the geometry of the slopes, making them vulnerable to environmental degradation and therefore increasing their susceptibility to failure. Slope failure of cut slopes along highways and rural roads is the most hazardous for users and government due to the high risks of human lives and the economic losses encountered every year across the globe. There are many factors affecting the stability of cut slope after construction. Among them, the most current is the deterioration of the earth's materials forming the slope over time especially when it is a granular residual material which are easily eroded. Natural pozzolan is one of such material.

Pozzolan is a highly porous material derived from volcanic projections. Its name derives from the town of Pozzuoli (Naples), in southern Italy, where it is widely widespread. It has been used in civil engineering since the 16th-18th century mainly in the industrial field for the production of cements for high-compactness and lightweight concretes. Although the literature presents a wide range of documents concerning this use, it remains rather reserved as to the evaluation of its behaviour in its place of deposition, especially when it is found in a slope, and in an excavated slope.

Being excavated, this type of material is highly susceptible to erosion and degraded easily due to its granular configuration. Many researchers such as Cecconi and Viggiani (2000); Avşar et al. (2015) have analysed the stability of this type of material in slope, but few of them have proposed an efficient and optimal stabilization technique against any instability. It is therefore in this context that the study of the stability of an excavated slope in a pozzolanic degraded rock mass in order to propose an efficient stabilization is of interest.

The creation of an excavation on a pozzolanic soil carried out on the Bangangté-Foumbot-Bamendjing-Galim road section from 48+900 to 51+700 in the West Cameroon region generated an instability of the created slope manifested by the spilling of pozzolan grains on the roadway, thus harming the safety of the users of the said section, raison of what stabilization is required. The main objective of this research work is therefore to propose a technical and economic solution for the stabilization of the pozzolan cut slope. As specific objectives: analyze the slope cases present on the site in order to detect the most unstable one(s) as well as the location of sliding surface, analyze the most unstable slope case reinforced by different remedial methods and Carry out a comparative study to select the optimal method of stabilization.

To achieve this, this document will be structured as follows:

- A literature search divided in 2 main parts: the first part based on soils in general and pozzolanic soils in particular, focusing on their physical, chemical, hydraulic and geotechnical properties, with a view to better understanding their mechanical behaviour; the second part of literature review highlighting the behaviour of granular soils such as pozzolan in excavated slopes focusing on factors that influence their stability, the failure mechanisms and the observe sliding modes, followed by the different methods of analysis of the slopes and concluding with the presentation of the corrective measures to this type of problem;
- The presentation of the methodology of analysis of excavated slopes without and with reinforcement as well as the criteria for the choice of the optimal stabilization method;
- Finally, the implementation of the previous methodology by presenting the results of slope analysis using the LEM method in GeoStudio's SLOPE/W product and the FEM method in the Plaxis 2D software, followed by a comparison of the stabilisation measures according to technical, economic and environmental criteria.

CHAPTER 1 LITTERATURE REVIEW

Introduction

The occurrence of slopes failure, whether natural or man-made, reveals a lack of knowledge necessary for the study and design of the named slopes in order to ensure their stability. This is not without consequences, as the instability generated leads to significant infrastructure damage and loss of human life. Since slopes are made by soils, it will be therefore important to study these soils i.e. their properties, classification, behaviour etc. for the best understanding of factors leading the slope to fail. With the aim to analyse a cut slope made by a pozzolanic soil in order to propose a technical and economic stabilization method; this chapter will focus on a review of the principal concept around soils in general and pozzolanic soils in particular. The concept of cut slope and slope failure mechanism in such soil type will be presented, followed by the approaches to stability analysis of slope and finally the appropriates remedial measures to instability in this type of soil material will close this chapter.

1.1. Soils

1.1.1. Definitions

Soils are natural aggregates with very different grains sizes, that forms the upper part of the earth crust. According to Budhu (2011), soils are simply materials that are derived from the weathering of rocks. Craig (2004) defines soil more explicitly as any uncemented or weakly cemented accumulation of mineral particles formed by the weathering of rocks, with the void space between particles containing water and/or air. Concerning rock, it is an aggregation of minerals into hard mass.

1.1.2 Formation of soils

As said, soils are derived from rocks. There are 3 main types of rocks: igneous rocks which are formed from magma emitted from volcanoes, sedimentary rocks formed from sediments and animal and plant materials that are deposited in water or on land on the earth's surface and then subjected to pressures and heat and metamorphic rocks formed deep within the earth's crust from the transformation of igneous and sedimentary rocks into denser rocks (Budhu, 2011).

Soils are formed by two rocks weathering process. the physical weathering which involves reduction of size without any change in the original composition of the parent rock, is caused by unloading, erosion, freezing, and thawing. The second process is the chemical weathering causing both reductions in size and chemical alteration of the original parent rock. The main agents responsible for chemical weathering are hydration, carbonation, and oxidation (Budhu, 2011).

1.1.3. Properties of soils

1.1.3.1.Physical properties

Physical properties of soils concern the soil structure, texture, colour, consistency, bulk density, pore space and permeability.

- Soil Structure refers to the arrangement and organization of soil particles in the soil, and the tendency of individual soil particles to bind together in aggregates;
- Soil texture refers to the proportion of the soil "separates" (sand, silt, clay) that make up the mineral component of soil;
- Soil colour gives an indication of the various processes going-on in the soil as well as the type of minerals in the soil;
- Soil consistence which is the resistance of a soil to deformation or rupture and is determined by the cohesive and adhesive properties of the soil mass;
- Bulk density which is the mass of soil per unit volume of soil (volume includes both soil and pores).
- Permeability refers to ability of a soil to transmit water and air. (FAO, 1987)

1.1.3.2.Geotechnical properties

Geotechnical properties of soils influence the stability of civil engineering structures. Some of that properties are:

- Particles size: the percentage of different sizes of soil particles coarser than 75 µ is determined by sieve analysis whereas less than 75 µ are determined by hydrometer analysis;

- Atterberg Limits: a gradual decrease in water content of a fine-grained soil slurry causes the soil to pass from liquid to plastic state, from plastic to semi-solid state, and finally to the solid state;
- Shear strength of soil is the result of friction and the interlocking of particles and possibly cementation or bonding at the particle contacts. It is expressed as a cohesion and the internal friction angle;
- Compaction is a process in which by expending compactive energy on soil, the grains are more closely rearranged. It increases the shear strength of soil and reduces its compressibility and permeability. (Roy and Bhalla, 2017)

1.1.4. Typology of soils

In general, soil type is identified by the texture of the soil. Thus, gravel, sand, silt and clay are listed as the main soil types. Sands and gravels are grouped as coarse-grained soils whose mechanical behaviour is affected by the size of the grains and the interaction between them. Clays and silts are fine-grained soils, their behaviour is affected by the type of minerals they contain.

According to Abramson et al. (2002), soil must be recognized in terms of the means of their transportation as well as the manner of deposition. Therefore, different type of soil will include:

- Alluvial deposit are fine sediments that have been eroded from rock and transported by water, and have settled on river and stream beds.
- Glacial deposit are mixed soils consisting of rock debris, sand, silt, clays, and boulders.
- Residual deposit is formed in place by a mechanical and chemical weathering of their parent bedrocks
- Colluvial deposit are soils found at the base of mountains that have been eroded by the combination of water and gravity
- Marine deposit soils are sand, silts, and clays deposited in salt or brackish water.

1.1.5. Classification of soils

Geotechnical engineers classify earth materials, for their properties according to their engineering behaviour relative to foundation support or the use as building material. This classification depends on their grain size distribution. The fraction subdivision of their diameter differs in the various international standard. The two commonly used system are:

- Unified Soil Classification System (**USCS**) standardized in ASTM D2487. In this norm, coarse-grained soils are classified according to their grain sizes and grain size distribution while fine-grained soils are related to their plasticity obtained by Atterberg Limits;
- American Association of State Highway and Transportation Officials (**AASHTO**) system with an original purpose to set material useful for road construction. This norm classifies soils into seven primary groups, named A-1 through A-7, based on their relative expected quality for road embankments, sub-grades, sub-bases, and bases.

After a short presentation of the soils in general, the rest of this review will be devoted to the study soil, namely the pozzolanic soil.

1.2. Pozzolanic soils

1.2.1. Definitions

Pyroclastic rocks are blocks of solidified lava ejected during the eruption of a volcano and are classified as ash, lapilli, bombs and boulders according to their diameter (Day, 1990). ‘‘Pyroclastic soil/rock’’ refers to soil/rock consisting of more than 75% of fragments originating from volcanic activity (Cecconi et al. 2010).

Pozzolan is a natural rock corresponding to scoriaceous volcanic projections, essentially Strombolian and basic (of basaltic composition) (Day, 1990). So called ‘‘soft’’ pyroclastic rocks (Cecconi et al. 2010), it is a loose, low-density material with an alveolar structure, mainly composed of volcanic glass, present in the form of elements of varying size (ashes, lapilli, blocks) RATSARAHASINA (2003).

1.2.2. Origin

The term "pozzolan" came from a U.S. simplification of "pozzolana" which evolved from the location Pozzuoli, in Italy where the Romans found a reactive silica-based material of volcanic origin which they called "pulvis puteolanus" (Day, 1990). Pozzolana belongs to the family of volcanic rocks. These rocks are divided into two groups, lava and projections; Pozzolana is formed by projection of basaltic magma fragments into the atmosphere (bombs, blocks, lapilli, ashes) followed by a sudden cooling during their aerial travel; and a deposit in a zone more or less distant from the place of emission.

1.2.3. Worldwide deposit of pozzolans

Pozzolans are found in almost all regions of the world, precisely in areas where there has been volcanic activity. The type of deposits encountered differs from one place to another.

- In America, the pozzolans deposit are encountered in the west of the Mississippi in US and Bolivia a region around LaPaz. It can be generally described as volcanic ashes, diatomaceous earth (Day, 1990);
- In Europe, the two prominent pozzolans are fine grained volcanic ash found mostly in Italy and Greece (Day, 1990);
- In Asia, some volcanic tuff beds also exist in north east of India;
- In Africa, it has been noted that the natural pozzolans can be found in 6 principal countries: Burundi, Cape Verde, Ethiopia, Rwanda, Tanzania and Cameroon (Day, 1990);
- In Cameroon pozzolana deposits are found mainly in 4 regions: in the Adamaoua Plateau; in the coastal region precisely in Mombo city (Djoungo commune) and Djombe-Penja city. Pozzolan is also founded in the South-West region near Mount Cameroon (Meme division). In the West region, it is found in Foumbot city.

1.2.4. Classification of pozzolanic soils

According to the degree of welding of pyroclasts, the natural pozzolana can be classified into 4 forms (Cecconi et al. 2010).

- Lithoid pyroclastic rocks where the pyroclasts (the clasts composing the pyroclastic rock) are strongly welded together and the deposit behaves as a soft rock which sometimes can be hydrothermally altered;

- Weakly welded and interlocked pyroclastic rock/soil: the pyroclasts are held together even without capillary tension. The behaviour of this soil is intermediate between welded granular soils and soft rocks;
- Granular pyroclastic deposit: the pyroclasts are not linked even if apparent cohesion may be present. With the weakly welded pyroclastic, they are the most usual type of deposit of pozzolan in nature;
- Altered pyroclastic deposit by an alteration process. The behaviour of this material is between fine grained soil and granular soils.

Figure 1.1 is an example of a weakly welded and interlocked pyroclastic soil.

Figure 1.1. Interlocked pyroclastic rock unit (proximal pyroclastic pozzolana deposits) in Tenerife (del Potro and Hürlimann, 2008)

Others classification system of pozzolana were established for determination of the suitable pozzolans for various applications in industry. For example, ASTM C618-92a classify pozzolan according to its quality standard into 3 classes (class N, F, C); each of them determined a precise chemical composition, physical properties and pozzolanic activity with lime. As already said, this is not our interest in this document.

1.2.5. Properties of pozzolanic soil

1.2.5.1. Geological properties

The pozzolan soil are soft pyroclastic weak rock. According to the value of unconfined compressive strength of pozzolana Nera, Cecconi and Viggiani (2001) have described pozzolan as a transitional material between soils and rocks. In the same study, it was said that the degree of fissuring of the pozzolana rock mass is generally quite low, without any detectable families of discontinuities. Del Potro (2008) studying weakly welded and interlocked pyroclastic rock units, concluded that despite having been classified as disintegrated granular units in the past, pozzolan are now classified as rock masses due to the high degree of interlocking between clasts.

1.2.5.2. Physical properties

a. Grain size

Pozzolana soil is mainly an ash flow deposit with grain size varying from fine ash (silt and clay) to lapilli (sand, gravel) with subordinated blocks and bombs. The grain size distribution of pozzolana soil depends strongly on the techniques adopted to separate larger aggregates before sieving (Cecconi and Viggiani, 2001). In general, there are large amount sand particles, medium silt and very few quantities of clay.

b. Colour

It is a material whose colour generally varies from deep black to brick red (see Figure 1.2) with all the colours in the intermediate range. On layers of altered deposits, it is brown in colour. These different colours give qualitative information on the mineralogical composition precisely of the weight percentages of ferrous and ferric oxides of the whole deposit.

Figure 1.2. Black and red colours of pozzolana grains

c. Structure and texture

The pozzolanic grains particles are generally angular to sub rounded as illustrated in Figure 1.3 and have rough to very rough surface texture. They have also a clastic texture diffuse in a scoriaceous matrix containing frequent crystals of leucocite, pyroxene and biotite as well as lithic fragments of lavas. Their morphological structures are vacuolar type and the cavities present at their surface (Figure 1.4) are due to the gases trapped in the lava during the volcanic eruption.

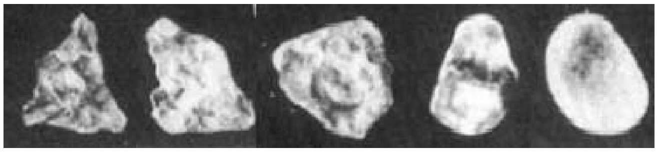


Figure 1.3. Variable pozzolana grain Shape (Cecconi et al. 2010)

Concerning the others properties, the properties of the Phlegraean pozzolana soil (De Vita, 2008), pozzolana Nera (Cecconi et Viggiani, 2001) and pozzolana of Neapolitan volcanic deposits (Pellegrino 1967) show that pozzolana is a material with a relatively low dry unit weight (ranging from 6 to 15 kN/m^3 and high porosity values (0.44 - 0.68) mainly due to the intra and inter granular pores as presents Figure 1.4. Pozzolana has also a great ability to absorb water and odors and a faculty of sound and heat insulation.

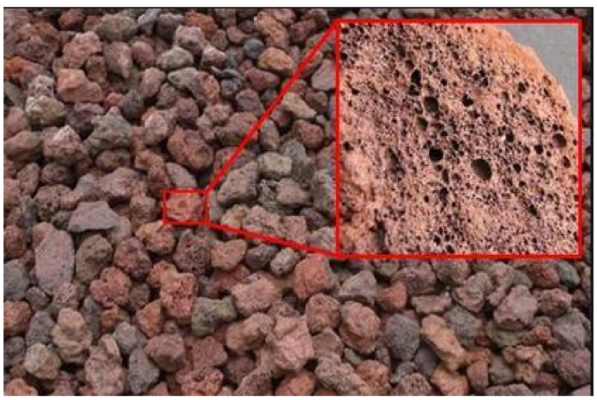


Figure 1.4. Intra granular pores of Thueyts pozzolana grains (Source: Geocaching)

1.2.5.3.Chemical properties

Pozzolan is a material with neutral PH and an average chemical composition of 46.4% silica (SiO_2), 17.5% alumina (Al_2O_3), 9.7% iron oxide (Fe_2O_3). It also contains lime, sodium, potassium and many trace elements. According to ASTM C125-07 norm, this composition allows pozzolan to react chemically with calcium hydroxide ($Ca(OH)_2$) at ordinary temperature in the presence of water and lime to form hydrated calcium silicates, a compound with binding properties-water and odor absorption capacity. This reaction is known as the pozzolanic reaction.

1.2.5.4.Hydraulic properties

The main hydraulic property of pozzolana soils is their ability to retain water by pores they content. This ability is assessed by the soil water retention curve (SWRC) which represent the volumetric water content $\boldsymbol{\theta_w}$ in function of suction **s**.

The evaluation of the SWRC of reconstructed sample of pozzolana Nera of variable voids ratio e_o, with natural sample of the same deposit shows that all the reconstructed sample has the same air entry pressure (a.e.p) = $(ua - uw).e$ meaning that the retention of water does not depends on e_o, but certainly on grain size dimeter (Canttoni et al. 2007). It was also noticed that, the a.e.p. of natural sample was lower than the one of reconstructed and the water content $\boldsymbol{\theta_w}$ of natural sample is greater than the one of reconstructed. This is due to the maximum grain size distribution in natural sample greater than for reconstructed. So, the ability of pozzolana to retain comes partly from the grain size distribution.

1.2.6. Mechanical behaviour of the pozzolanic soils

For the determination of shear strength behaviour of pozzolanic soils, several experimental and analytical studies have been carried out. These studies were based on the effective stress concept and on the Mohr-Coulomb model presented on the sections below.

1.2.6.1.Effective stress concept

In saturated soils, the effective stress (σ’) is the stress generated by the soil skeleton at the inter-particles contacts while pore water pressure (u) is the pressure exert by water in pores. The correlation of effective stress with soil behaviour is the principle known as effective stress principle defined by the Equation (1.1):

$$\sigma' = \sigma - u \qquad (1.1)$$

Among these stresses, it is only the effective stress that controls the shear strength behaviour of soils, and this can be implemented via the Mohr Coulomb failure criterion (Abramson et al. 2002).

1.2.6.2.Mohr-Coulomb failure criterion

It is an idealized representation of soil helping to interpret its shear strength. The principal hypothesis of this criterion is based on the fact that a combination of normal and shear stresses creates a more critical limiting state than would be found if only the major principal stress or maximum shear stress were to be considered individually (Abramson et al. 2002). The Mohr-Coulomb failure envelope is described as:

$$\boldsymbol{\tau = c' + (\sigma - u_w) \tan \varphi'} \quad \text{or} \quad \boldsymbol{\tau = c' + \sigma' \tan \varphi'} \qquad (1.2)$$

where **c'** = cohesion (intercept on the strength axis)

u_w or **u** = pore water pressure

φ' = angle of internal friction related to the slope of Mohr-Coulomb line shown in Figure 1.5

In the Figure 1.5, the Mohr circle A indicates a safe stress state while the circle B is tangential to the Mohr Coulomb failure envelope, meaning that a critical combination of σ and τ has been reached.

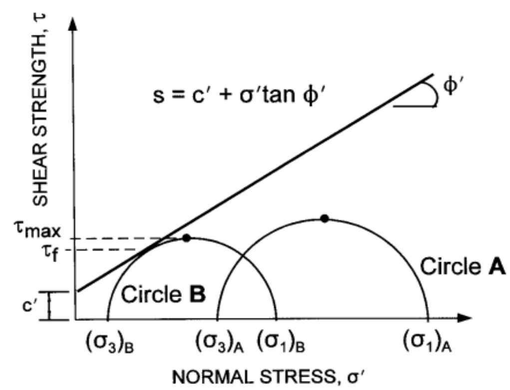


Figure 1.5. Mohr-Coulomb failure envelope for shear strength of soils (Abramson et al. 2002)

1.2.6.3.Mohr-Coulomb failure envelope for unsaturated soils

For unsaturated soils, the above described Mohr-Coulomb failure criterion must to be adjusted. The applied stresses in such conditions will be sustained by the soil skeleton, pore water and air voids (Abramson et al, 2002). In the Figures 1.6 and 1.7 representing the extended Mohr-Coulomb failure envelope of unsaturated soils, the increase in the difference between pore air and water pressures lead to the increase of apparent cohesion (ca). This increase of ca is function of φ^b angle.

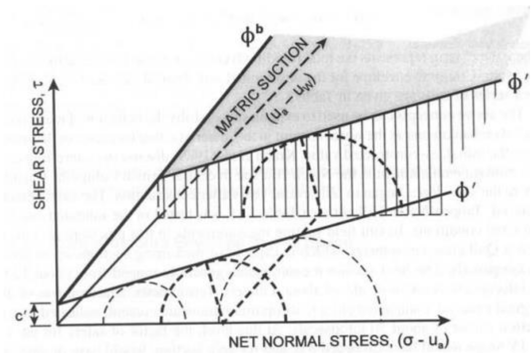


Figure 1.7. Extended Mohr-Coulomb failure envelope for unsaturated soils (Abramson et al. 2002)

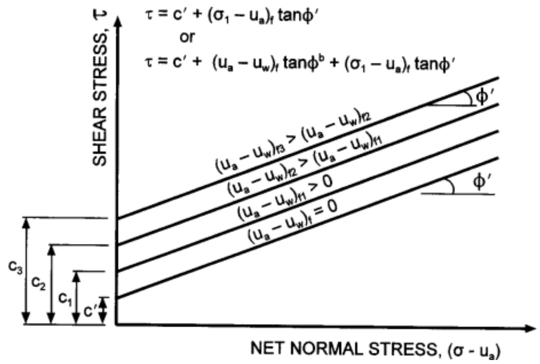


Figure 1.6. Horizontal projection of contour lines of the failure envelope onto the τ versus (σ – ua) (Abramson et al. 2002)

The shear strength of unsaturated soils can be readily accommodated using the total cohesion method. Herein, the modify value of c' is used to represent the effect of negative pore pressure u_w within the slope. Fredlund et al (1978) proposed a relation to express the shear strength of unsaturated soils which is:

$$\boldsymbol{\tau = [c' + (u_a - u_w)\tan\varphi^b] + (\sigma - u_a)\tan\varphi'} \qquad (1.3)$$
$$\boldsymbol{= c^* + (\sigma - u_a)\tan\varphi'}$$

with $\boldsymbol{\varphi_b}$ the slope of the plot of matrix suction versus the shear stress line and **ua** the pore air pressure.

c* represents the total cohesion to be used, which is the sum of effective cohesion **c'** and the apparent cohesion expressed as: $\boldsymbol{c_a = (u_a - u_w)\tan\varphi^b}$

When the soil approach to saturation ($\boldsymbol{u_w \approx u_a}$), Equation 1.3 becomes simply 1.2.

1.2.6.4.General behaviour of granular soils

Granular soils are particles assemblies predominantly sand and gravels which are devoid of inter-particles cohesion, and where the individual particles are independent of each other except for frictional interaction (F. H. Tinoco, 1967). Even if the determination of their shear strength is possible to be made in-situ, laboratory tests are far the most common.

a. Laboratory tests for shear strength determination

Triaxial compression (TX) test is one of the most common laboratory tests used for shear strength determination even total and/or effective strength. Direct shear (DS) test may also be used.

But, because there is no possibility to control or measure the pore water pressure during this test, its shear strength results are usually reported in terms of effective stress.

In unsaturated conditions, some of the useful tests to obtain shear strength are the modified DS test and the Undrained test. They are much more complicated because it needs to control and monitor during the test the pore air pressure (Abramson et al, 2002).

b. Observed behaviour of granular materials

The stress–strain curve shown in Figure1.8 illustrated the behaviour of a sand prepared at different relative densities (loose and dense). At the beginning dense soils denote a progressive hardening until the peak value. After that, the shear resistance decreases up to reach a constant value at large displacement (ultimate or residual or critical strength). The change from maximum to minimum strength is attributed to dilatant behaviour initially, which is then followed by contraction as the material approaches the critical state. Loose soils display a non-brittle behaviour and have to reach large displacement to mobilize the fully resistance: no peak occurs in stress-strain curves, only hardening is observed.

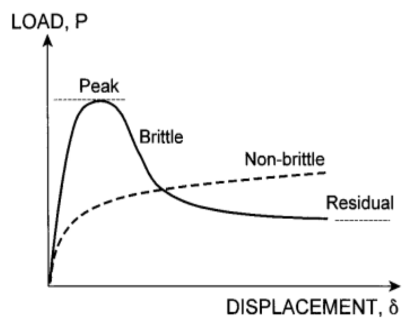


Figure 1.8. Typical stress-strain curve for granular soils (Abramson et al. 2002)

c. Effect of suction on shear strength of pozzolanic soils

As many others types of granular soils, the shear strength that pozzolanic soil exhibits depends roughly on their microstructure and on their hydraulic properties.

Unsaturated soils are characterized by a pore pressure, lower than the atmospheric pressure, called suction (De Vita et al, 2008). Suction affects the mechanical behaviour of pozzolanic soils in two different ways.

- the different air and water pressures modify the soil stress skeleton;
- the air-water interface provides a sort of additional "bonding" to the particle contacts (additional to the natural inter particles bonds) creating a small amount of apparent cohesion. However, such cohesion is for temporary conditions so they are not generally relied upon for slopes design (Abramson et al, 2002). But many researches have been done in such condition to evaluate the effect of suction on shear strength of pozzolan.

In the aim to determine the influence of suction on shear strength, many studies have been carried out on different type of pozzolan soil such as pozzolana Nera, Phlegraean pozzolan soil, pozzolan deposit in a quarry in the Campi Flegrei area near Naples in Italy (Cantoni et al. (2005); Cantoni et al. (2007); De Vita et al. (2008); Stanier and Tarantino (2013)).

A suction controlled Triaxial tests with shearing on unsaturated reconstructed pozzolana Nera allowed the stress-strain behaviour to be investigated. From the results in figure1.9, it can be noticed that, the peak shear strength increases with increasing suction. Suction increase modifies the stress state of the soil, specifically it increases the effective stress by reciprocal attraction among solid soil particle acted by capillary meniscuses (De vita et al, 2008). It follows for the Mohr-Coulomb model, the increase of shear strength. For this reason, the more diffused approach models the effect of the suction as an additional contribution to the shear strength by an apparent cohesion (ca) that depends on the suction itself. A strong dilatant behaviour is observed and dilatancy seems to be mainly induced by the increase of suction (Canttoni et al. 2007). No sensible effects of suction or degree of saturation can be detected on the ultimate shear strength as it presented on Figure1.9.

Stanier and Tarantino (2013) performed an investigation of suction contribution on shear strength behaviour. It was noticed that, when shear strength criterion for non-aggregated soil given by Equation (1.4) is extrapolated at high suctions, the contribution of suction to shear strength $\Delta\tau$ increases indefinitely with suction following Equation (1.4) as Figure 1.10 illustrates. That was not intuitively acceptable because Equation (1.5) fails when pore water is abundantly present in the form of absorbed water as it is occurs at high suction. So, it was assumed a residual suction at which when it is exceeded, the contribution of suction to shear strength becomes constant.

$$\Delta\tau = s \cdot Sr \cdot \tan \phi' \qquad (1.4)$$

$$\tau = (\sigma - u_w S_r) \tan \phi' = (\sigma + sS_r) \tan \phi' \qquad (1.5)$$

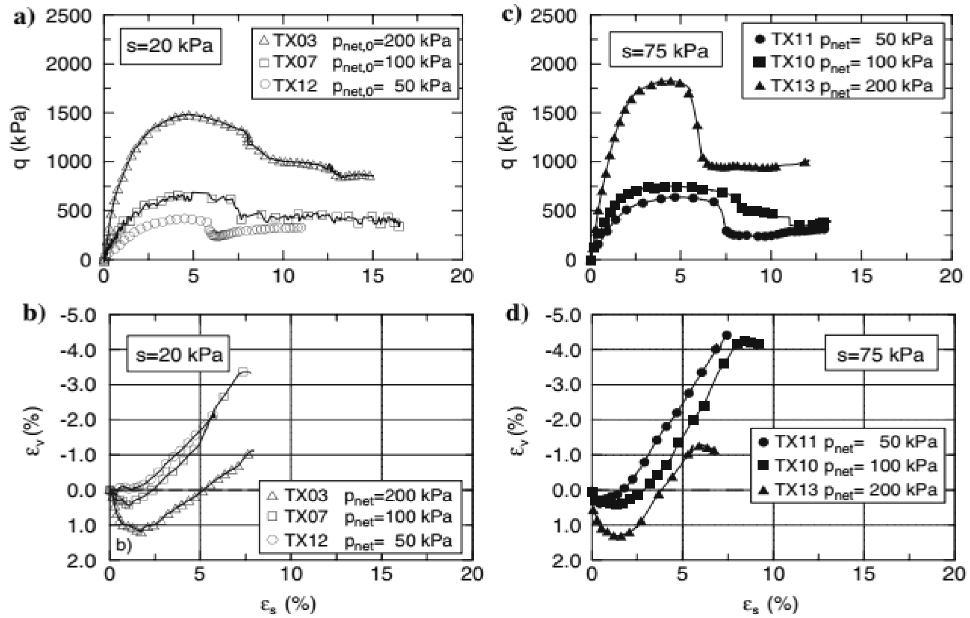


Figure 1.9. Triaxial compression tests on pozzolana Nera at constant suction s (Canttoni et al. 2007)

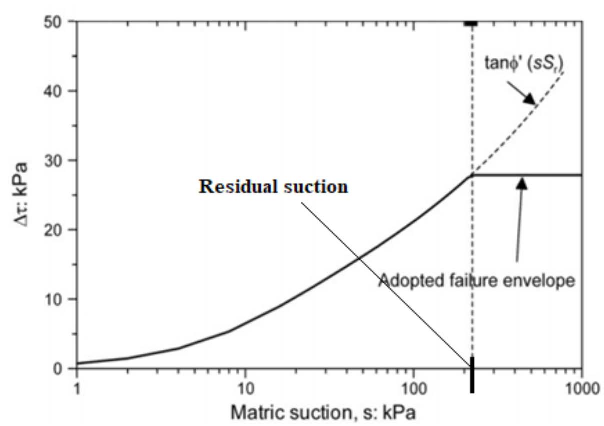


Figure 1.10. Shear strength criterion of pozzolanic soil near Naples Italy (Stanier et Tarantino, 2013)

1.2.7. The use of natural pozzolana in industry

The interest in the use of pozzolan in industry comes from its chemical properties which derive from its composition of silica, alumina and iron oxides. In general, pozzolans have 2 main fields of use.

1.2.7.1.Cement manufacture

Pozzolana have been continuously exploited since Roman and Etruscan times to produce hydraulic mortars and cement

According to the American standard **ASTM C125-07**, natural pozzolans are siliceous or silico-aluminous materials, which do not themselves possess binding properties but which, in finely divided form and in the presence of moisture, react chemically with calcium hydroxide at room temperature to form compounds with binding properties. These reactions promote their use in the manufacture of cements.

Pozzolanic cements, known as PPC (Portland Pozzolanic Cement), are a mixture of pozzolan (natural or artificial), clinker and/or slag. The raw materials used in the manufacture of this cement are taken from their natural state and ground into fine particles. The mixture is then heated to form clinker. This clinker is mixed with pozzolanic materials in the required proportions to obtain Portland pozzolanic cement. One of the advantages of this cement is that it has a very good resistance to sulphate attack and is therefore used in hydraulic structures (Source: Natural Pozzolan Association).

1.2.7.2. Manufacture of lightweight concretes

Due to their high porosity, which gives them a low density, pozzolans are used as aggregates in the manufacture of lightweight concretes that have low mechanical resistance but excellent thermal and sound insulation qualities. In addition to these main areas of application, others use of pozzolans are also to be noted. In road infrastructure it is used for the subgrade of light embankments. It is also used for the construction of sports grounds as athletics track, tennis court etc. and also in agriculture as drainage layer.

After presenting soils in general and granular soil as pozzolan in particular, the next sections will focus on an important topic in geotechnical engineering which is the stability of cut slope in such granular soils.

1.3. Cut slope

1.3.1. Definition

A slope formed by excavating overlying material to connect the original ground surface with a lower ground surface created by the excavation is called a cut slope (Highway Runoff, WSDOT). The objectives of cutting a rock or soil deposit is in general to create space for the road template and driving surface as presented in Figure 1.11.

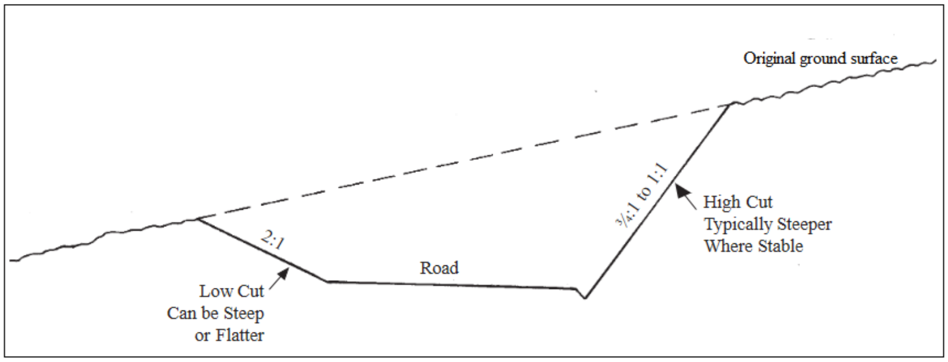


Figure 1.11. Schematic view of cut slope

1.3.2. Design requirements

The aim when designing a cut slope is to determine a height and inclination that is economical and corresponding to the type of soil in place in order to remain stable for a reasonable life span. Ideally, vertical cut slopes should not be used unless the cut is in rock or very well cemented soil. Long-term stable cut slopes in most soils and geographic areas are typically made with about a 1H:1V or ¾H:1V. Others common stable slope ratios for varying soil/rock conditions are presented in Table 1.1.

Table 1.1. Cut slope ratio for varying soil/rock condition (Source: fs.fed.us)

Soil/Rock type	Slope ratio (H: V)
Most rock	1:4 to 1:2
Very well cemented soils	1:4 to 1:2
Most in place soils	3:4 to 1:1
Very fractured rock	1:1 to 3:2
Loose coarse granular soils	3:2
heavy clay soils	2:1 to 3:1

To achieve a well design stable cut slope, specifics data (geological data, soil strength, ground water conditions) must be collected and analyse before beginning excavations.

During excavation (short term condition), the free-water surface will drop slowly to a stable zone (below the new cut surface) and the factor of safety starts to decrease gradually and rapidly especially in sandy cut slope. From the end of excavation to long-term, a rising of water table is observed which reduces the shear strength of the slope and consequently diminution of the safety factor occurs. So, the riskier condition is at long-term as presents the Figure 1.12, where the negative pore pressure, due to lateral decompression of slope, dissipates and generating a reduction of shear strength. This low FoS at long term explains why cut slope generally can fail many years later without much warning.

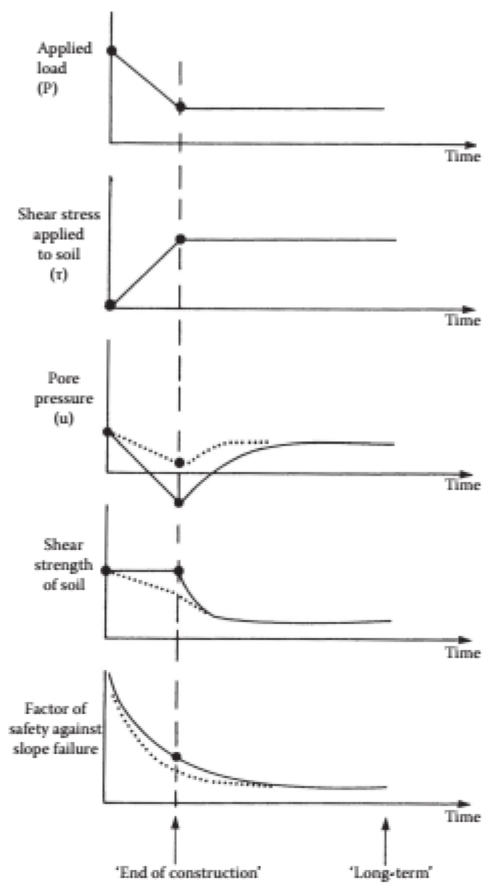


Figure 1.12. Stability condition for a cut slope in a clayey soil (Clayton et al., 2014)

1.4. Slope failure

1.4.1. Overview

So called land slide, slope failure is a movement of rock, earth or debris down a sloped section of land. In cut slope, this phenomenon typically occurs where a slope is over-steep or where cuts in natural soils encounter ground water or zones of weak material. The consequences of cut slope failure after construction as presented in Figure 1.13 can range from direct costs such as removing the failed material; to a wide variety of indirect costs such as damage to vehicles and injury to passengers. (Wyllie and Mah, 2005).

Figure 1.13. Cut slope failure in Bukit Lanjan, Malaysia December 2003 (source: AGU, Advancing earth and space science, 2010)

1.4.2. Factors leading to slope instability in granular soils

Slope stability in granular soils can be significantly impacted by the slope geometry (slope angle and height), material strength, the nature of grains and deposit, geo-hydrological condition (water effect and earthquake). All these factors can be classified into internal and external factors.

1.4.2.1. Internal factors

These factors are those relating to the physical characteristics of the grains and their arrangement and interaction within the deposit.

a. Shape

When particles have the shape and freedom permitting to roll, it will tend to detach easily to reach a more stable position, making the slope unstable.

b. Angularity

When particle presents a low degree of angularity, the interlocking with others particles is reduced and the freedom to roll increases, creating instability.

c. Surface texture

It provides a friction effect between the particles, which generates slip resistance and thus promotes slope stability.

d. Gradation

Well graded materials are by nature more stable on slopes. Such materials are characterized by a coefficient of curvature Cc [1-3] and a coefficient of uniformity Cu > 6 for sand and Cu >4 for gravel. In addition, stable arrangement results when larger particles are in contact with one another and smaller particles occupy the voids among larger.

e. Particles orientation

An alignment of flat or elongated particles parallel to the slope generally tend the grains to slip more easily, making the slope unstable. In contrary, when grain alignment is perpendicular to the slope, the deposit can remain stable even for a great value of slope inclination (see Figure 1.14).

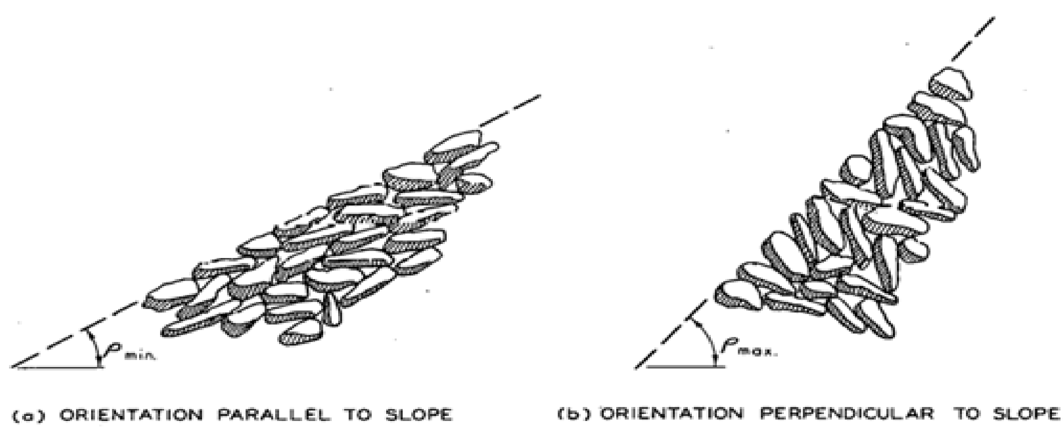


Figure 1.14. Effect of particles orientation on stability of slope (Maclver, 1967)

f. Slope angle effect

Slopes at different angles (with same particles orientation) present different grain arrangements and this has a huge impact on stability. Considering the mass as a systematic stacking of spheres as shown in Figure 1.15, it is found that if a single particle at toe was disturbed on a less steep slope, the rest of the surface particles would not be affected and the slope would remain stable, but the disturbance of any particle on the steeper slope would result in the displacement of the entire surface layer, causing a shallow failure.

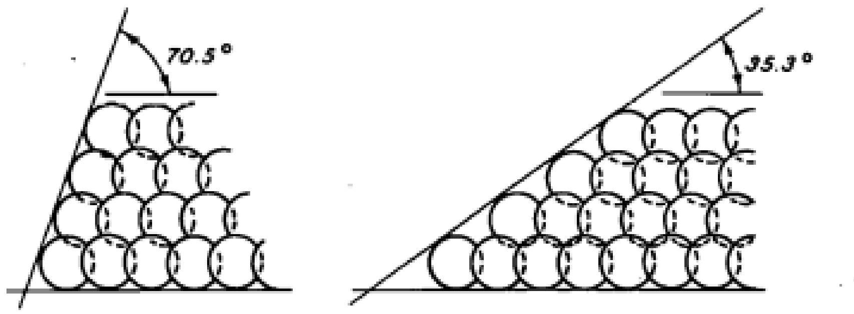


Figure 1.15. Different slope angle with correspondent particles arrangement (Maclver, 1967)

g. Water content effect

The effect of a small amount of water in a cohesionless material is to give the material an apparent cohesion by forming menisci at inter-particle contacts (see Figure 1.16), increasing the pressure of these contacts by surface tension and therefore enhance stability. This effect is eliminated as the slope is completely submerged and also as it is completely dry.

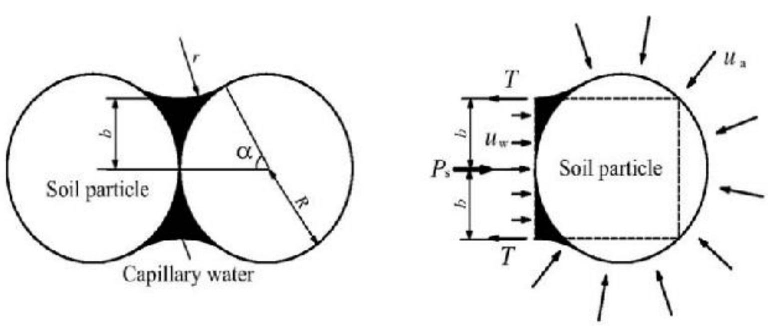


Figure 1.16. Capillary menisci between two spherical particles (Chen and Liu, 2010)

1.4.2.2. External factors

a. Hydrologic effect

Intense rainfall can generate extreme infiltration on granular soils because of their high degree of permeability. This long period of rainfall saturates the soils and promotes erosion at shallow depths as presents the Figure 1.17. The rise of the water level in the slope implies the increase of the degree of saturation which will cause the reduction of the apparent cohesion and then decrease the soil strength leading to failure.

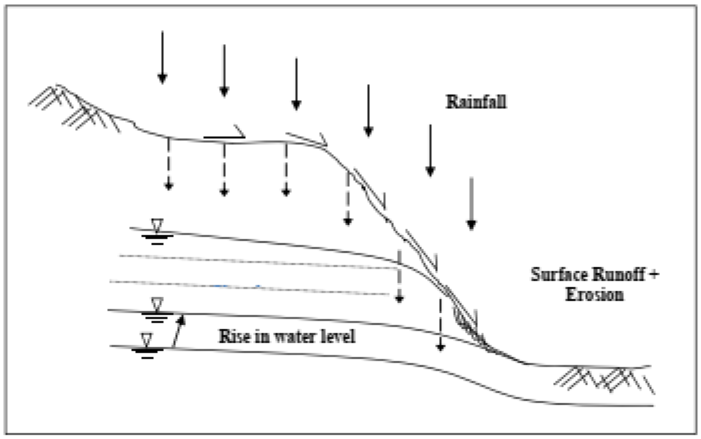


Figure 1.17. Effect of rainfall on high permeable slope (Niroumand et al.2012)

b. Instability induced by earthquake

Especially encountered in saturated granular soils, earthquake vibrations or vibrating machines can lead to the liquefaction of such soils. Due to vibration, the inter-granular contact breaks (σ' =0) and water fills the created spaces as shown in Figure 1.18 which leads to a loss of shear resistance of the slope (τ' = 0) and therefore the slope becomes unstable. The soil changes from the state of solid-like to fluid-like materials, and the observed soil movement is a debris flow.

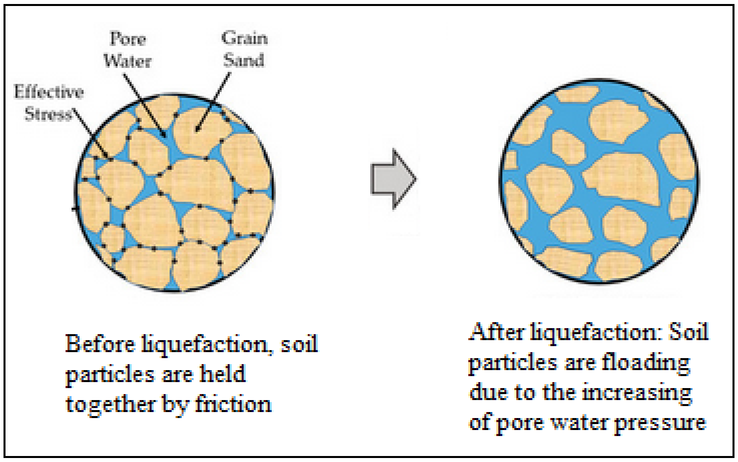


Figure 1.18. Liquefaction mechanism: soil particles floating due to the increment of pore water pressure (Araujo and Ledezma, 2020)

1.4.3. Observed movement during failure

In granular soil, single particles and aggregate mass movement can be observed when slope fails. Single particles can either slide or roll depending on the equilibrium of forces and moment around the centre of gravity. According to aggregate mass, among the type movement based on Varnes (1978) classification, the ones which can correspond to non-cohesive soils are:

- **Flow** characterized by downward and outward movement under saturated condition in which material moves as a viscous fluid as presents the Figure 1.19.

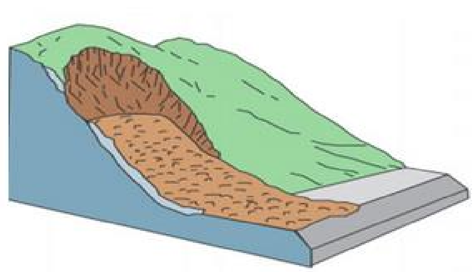


Figure 1.19. Debris flow

- **Slide** which is a movement toward the slope toe of a mass of soil or weak residual rock, which occurs along a surface or in a shear band where shear strains are localized. This movement may be translational, rotational or combination of both.

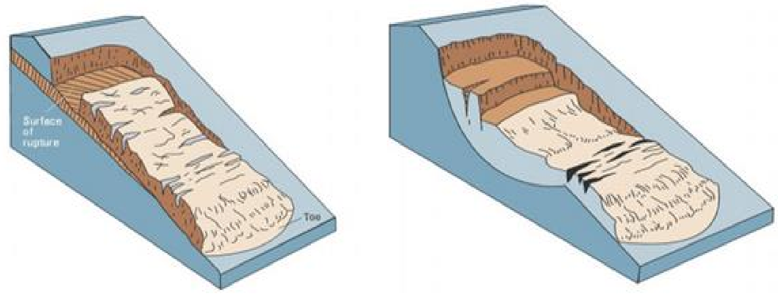


Figure 1.20. Translational and rotational slide movement

1.5. Mechanical approaches to slope stability analysis

Slope stability analysis is a static or dynamic, analytical or numerical method to access safe and economic design and evaluate the stability of any type of slope. For this aim, the concept of safety factor is widely used.

1.5.1. Safety factor concept

To assess if a slope is stable or not, the concept of safety factor (**FoS)** was established. The most widely used definition of safety factor for a slope is the ratio of the maximum shear strength of the soil to shear stress required for equilibrium.

$$\mathbf{F_0S = \frac{\tau_{max}}{\tau}} \tag{1.6}$$

Many others definition exist:

- Based on the forces equilibrium: $$\mathbf{FoS} = \frac{\textit{Resisting forces}}{\textit{Driving forces}} \tag{1.7}$$
- Based on moment equilibrium $$\mathbf{FoS} = \frac{\textit{Resisting moment}}{\textit{Overturning moment}} \tag{1.8}$$

Theoretically, the slope is stable when FoS > 1, unstable when FoS < 1 and in critical condition when FoS = 1. Because there are many factors that must be included when calculating the factor of safety and they were not or were just assumed, the computed values of FoS in practice are not precise, due to uncertainty of variables. Therefore, the factor of safety should be larger to be on the safe side (Duncan and Wright, 2005). Duncan and Wright (2005) recommend the value of FoS listed in Table 1.2 to verify the stability of slope.

Table 1.2. Recommended minimum values of factor of safety (Duncan and Wright, 2005)

Cost and consequences of slope failure	Uncertainty of analysis conditions	
	Small	Large
Cost of repair comparable to incremental cost to more conservatively designed slope	1.25	1.5
Cost of repair much greater than incremental cost to construct more conservatively designed slope	1.5	2.0 or greater

Further requirements based on the assessment of the risk of loss of life and economic loss (Table 1.3) have been adopted in Hong Kong, and these values have been found to be satisfactory.

Table 1.3. Recommended safety factor (Cheng and Lau, 2014)

Risk of economic losses	Risk of human losses		
	Negligible	Average	High
Negligible	1.1	1.2	1.4
Average	1.2	1.3	1.4
High	1.4	1.4	1.5

1.5.2. Methods of stability analysis

The appropriate method for a stability analysis depends strongly on the type of failure identified. Some of these methods are based on large assumptions but require few parameters, while others, with small assumptions on the sliding surface, recommend many soil parameters.

1.5.2.1. Limit equilibrium method (LEM)

LEM is the most popular approach in slope stability analysis. This method is well known to be a statically indeterminate problem, and assumptions on the interslice shear forces are required to render the problem statically determinate (Cheng and Lau, 2014). Amongst the methods based on LEM, there are:

a. Infinite slope analysis

The infinite slope analysis is the easiest method based on LEM and is characterized by a single rigid block of soil, sliding on an infinite planar slip surface. It reflects the behaviour of a slope that extends for a relatively long distance. A typical slice in dry condition is represented in Figure 1.21 where T and N are respectively driving force and the normal force expressed as N = Wcosβ and W the weight of the considered slice. From Equation (1.7) we obtain

$$\text{FoS} = \frac{\boldsymbol{S}}{\boldsymbol{T}} = \frac{\boldsymbol{N}\tan\boldsymbol{\varphi}}{\boldsymbol{W}\sin\boldsymbol{\beta}} \qquad (1.9)$$

Where **S** is the available frictional strength along failure plane (resisting force)

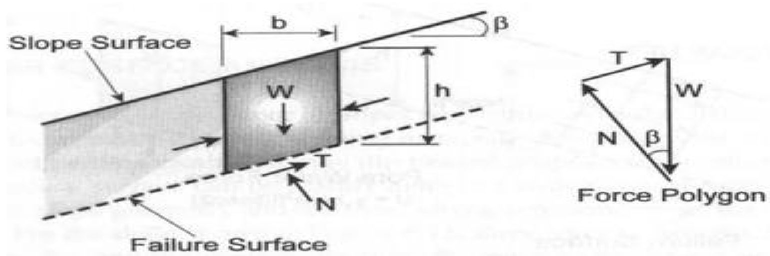


Figure 1.21. Infinite slope failure in dry sand (Abramson et al. 2002)

b. Method of slice

In this method, the soil mass above a circular slip surface is divided into a number of vertical slices, and the limit equilibrium of each slice is performed to obtain the safety factor. The greater the number of slices, the more precise the safety factor calculation. This division of the moving mass into vertical slices has allowed the development of a very large number of methods such as Simplified Bishop's method, the Morgenstern-Price method, Janbu's method, the ordinary method of slices (OMS) etc. They generally differ only in the assumptions needed to make the equations statically determinate (Kliche, 1999).

i. Bishop simplified method (1955)

The simplified Bishop method assumes zero interslice shear forces and satisfies vertical force equilibrium for each slice and overall moment equilibrium about the center of a circular trial surface.

Using the Figure 1.22 and considering the vertical forces equilibrium to the slice the following relation is obtained.

$W_i+\Delta X_i=N_i cos\alpha_i + Ub_i cos\alpha_i + (N_i tan\phi' + C'\Delta l_i) sin\alpha_i/FoS$

$\rightarrow N_i= [W_i+\Delta X_i- Ub_i cos\alpha_i-C'\Delta l_i \ sin\alpha_i/FoS]/(cos\alpha_i + sin\alpha i \ tan\phi'/FoS)$

assuming that $m= cos\alpha i + sin\alpha i \ tan\phi'/FoS$

$\rightarrow FoS= \Sigma[C'\Delta l_i + (W_i+\Delta X_i- Ub_i cos\alpha_i-C'\Delta l_i \ sin\alpha_i/FS)tan\phi'/m]/(\Sigma W_i sin\alpha_i)$.

Neglecting the interslice forces, the safety factor using bishop formula is:

$$\mathbf{FoS} = \frac{\Sigma[C'\Delta li + (Wi - Ubicos\ \alpha i\ C'\Delta li\ sin\alpha i/FoS)tan\ \varphi'\ /m\ )]}{\Sigma Wisin\ \alpha i} \quad \mathbf{(1.10)}$$

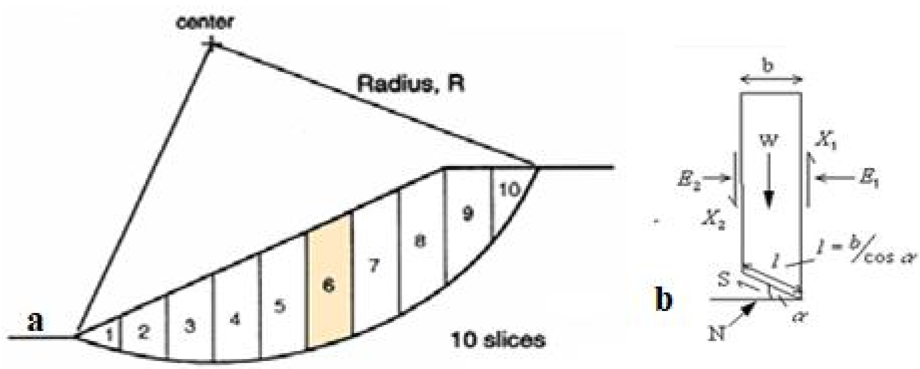


Figure 1. 22. Division of potential sliding mass into slices; b) forces acting on a slice

ii. Janbu simplified (1956)

The method considers the inter-slices forces as horizontal forces, and also assumes any failure surface (non-circular). Horizontal and vertical forces equilibrium are verified by neglecting the moment equilibrium which results in the safety factor of the Equation (1.11)

$$\mathbf{FoS} = \frac{\Sigma[c'l+(N-ul)tan\varphi']cos\alpha}{\Sigma W tan\alpha} \quad \mathbf{(1.11)}$$

iii. Morgenstern-Price method (1965)

The M-P method is based on the following assumptions:

- The method considers non circular failure surface;
- Inter-slices forces are parallel
- Normal forces act at the center of the base of each slice.

This method verifies the balance of horizontal and vertical forces as well as the balance of moments at a single point. It is known to be very accurate and applicable to all geometries and soil types.

1.5.2.2. Finite element method (FEM)

Although LEM provide useful preliminary analysis and are mostly adequate for simple slope failure mechanisms, since they only consider forces or moments, not slope movements, they are unable to fully capture complex slope failure mechanisms (Sari, 2019). This is how the finite element method came into being to overcome this limitation. The finite element method represents a powerful alternative approach for slope stability analysis which is accurate, versatile and requires fewer a priori assumptions, especially regarding the failure mechanism. Slope failure in the FEM occurs naturally through the zones in which the shear strength of soil is insufficient to resist the shear stress (Griffiths and Lane, 1999).

FEM uses the SSR (Shear Strength Reduction) method by which the soil shear strength is reduced to bring a slope to verge of failure (Duncan and Wright, 1996). In this technique, the Mohr-Coulomb material shear strength is reduced by a factor F (or safety factor) until failure occurs. The definition is expressed as:

$$\frac{\tau}{F} = \frac{c'}{F} + \frac{\tan\phi'}{F} \qquad \textbf{(1.12)}$$

Replacing $\mathbf{c^* = \frac{C'}{F}}$ and $\boldsymbol{\phi^* = \arctan\left(\frac{\tan\phi'}{F}\right)}$ we obtain

$$\frac{\tau}{F} = c^* + \tan\phi^* \qquad \textbf{(1.13)}$$

With **c*** and **φ*** the reduced Mohr coulomb parameters.

When modelling with this method, the soil continuum is divided as presented in Figure 1.23 into discrete and simple geometrical units called finite elements and are interconnected at their nodes at predefined boundaries of continuum model. Each element is subjected to the action of neighbouring elements. The material properties are defined according to perfect plastic or elastoplastic model, and then the differential equations governing a particular phenomenon are solved in order to obtain stress and displacement fields compatible with these equations.

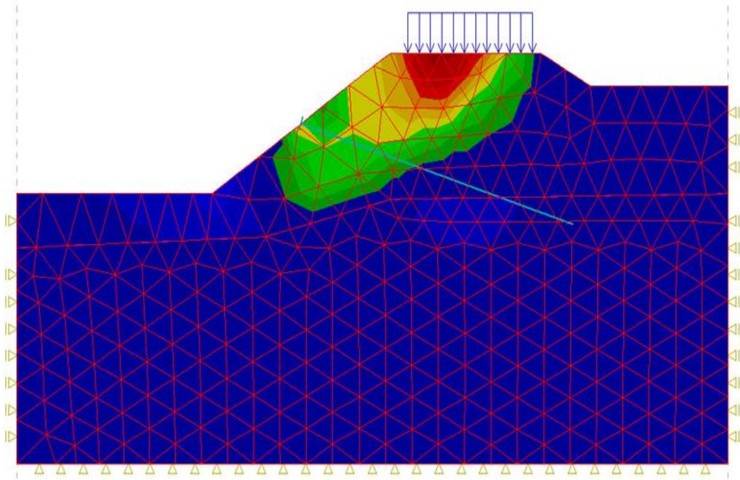


Figure 1.23. Finite element mesh (source: Wikipedia)

1.6. Slope stabilization techniques

Many researches have been realized in the aim to develop a suitable stabilization technique of granular soils on slope; this in order to enhance the safety factor obtained after analysis. Among them there are:

1.6.1. Modification of slope profile

When constructing a cut slope, if the angle created by excavation is greater than the angle of repose of the material forming the slope, it would be important to modify the slope angle and may be its height in order to get a more stable situation. Modification of slope profile by excavation will reduce the weight soil which is a destabilising action to the stability.

There are many types of modification that can be done on a slope: the top of the slope can be excavated reducing its height, bench can be created and slope angle can be reduced by flattening as see in Figure 1.24.

One of the constraints of this technique could be the presence of endangered structures at the ridge if the excavation is closer to it.

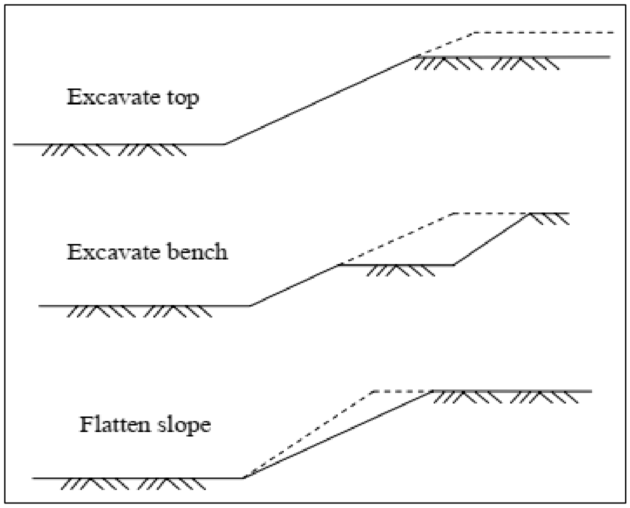


Figure 1.24. Schematic diagram of a slope stability improvement by excavation (Wanstreet, 2007)

1.6.2. Retaining walls

Retaining walls are the oldest known and most widely used slope stabilisation techniques in the world. They are generally used for small landslides, for toe protection against erosion and also for retaining an excavation. There exist many types such as mass gravity, Cantilever, Gabion (see Figure 1.25), Mechanically stabilized Earth (MSE) wall, Crib walls (see Figure 1.26), embedded wall, etc.

Figure 1.25. cut slope stabilization through gabion walls (Source: Geotech Rijeka)

Figure 1.26. Crib wall (Source: CTS BARE)

The main objective when using a retaining wall to stabilise a slope is to increase the strength of soil and decrease the forces which cause instability in order to increase the safety factor. The factors that influence the use of one type of wall over another concern the knowledge of the technique, the constraints linked to the site (type of materials, environmental conditions, the space reserved for the structure, etc.) and the budget allocated to the construction.

The materials used for the construction of these walls differ from one wall to another. Some are essentially made of cast-in-place concrete (mass gravity wall), others are made of reinforced concrete (semi gravity, cantilever), a crushed rock and steel mesh (the gabion wall), geotextiles and earth fill (MSE).

Since failures in granular material generally begin at slope toe, where one particle detaches from this slope and leading the others particle on surficial layer. The construction of retaining wall at the toe of a slope may mitigate and /or prevent small size or secondary landslips that often occur along the toe portions.

1.6.3. Biotechnical slope stabilization method

Slopes created by excavation are highly susceptible to surface erosion due to wind and water run-off, and therefore making the slope unstable by carrying away the surface particles. Specialized methods have been developed for establishing vegetation on slopes and these are called **s**oil biotechnology or soil bioengineering systems. These construction methods used mainly unrooted cuttings, which are taken from live plants and installed in the ground by various means and in various configurations (Morgan and Rickson, 2005). There are many types of soil bioengineering systems. Most of these systems serve the dual purpose of reducing surface erosion and reinforcing the soil.

1.6.3.1. Vegetated geogrid

Also called soil confinement system, vegetated geogrid is a slope stabilisation technique which divides the top layer of slopes into many small areas or cells (see Figure 1.27) reducing the downward migration of individual particles caused by gravity and hydraulic traction. The system consists of geocells (geosynthetic), anchoring material, backfill material and vegetation. The method presents 2 final purposes:

- The geogrid provides immediate stability by confining surface soil and allow seed to germinate quickly;
- When it will grow, vegetation will contribute to the long-term protection against surface erosion and shallow sliding as their roots bind the soil and provide cohesion improving stability.

Besides providing slope stability, an attractive feature of the soil confinement system is preserving the natural look of the slope surface. (Gofar and Kassim, 2008).

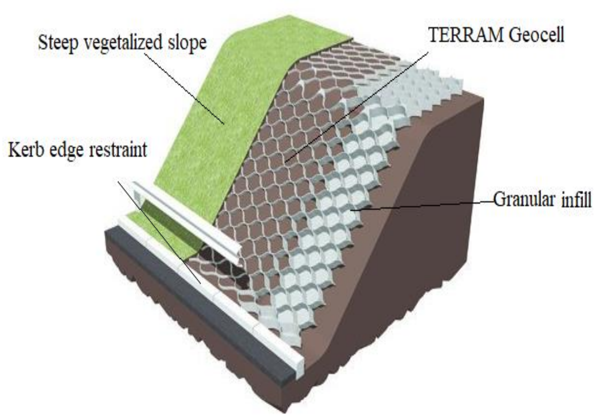


Figure 1.27. Soil confinement system for erosion control

The French standard NF EN ISO 10318 defines a geosynthetic as a product of which at least one of the components is based on synthetic or natural polymers and which is presented in the form of webs, strips or three-dimensional structures. They are used to provide one or more of the following functions: separation, reinforcement, filtration, drainage or liquid barrier.

They are grouped in 4 main families:

- **Geotextiles:** permeable fabrics which, when used in association with soil, have the ability to separate, filter, reinforce, protect, or drain.
- Other Geosynthetics and similar (geogrids, geonets, geocells)
  - **Geogrids:** used to reinforce retaining walls, as well as subbases or subsoils below roads or structures. Generally made by polymer materials, such as polyester, polyvinyl alcohol, polyethylene or polypropylene
  - **Geonets:** consisting of integrally connected parallel sets of ribs overlying similar sets at various angles for in-plane drainage of liquids or gases.
  - **Geocells** are widely used in construction for erosion control, soil stabilization on flat ground and steep slopes, channel protection, and structural reinforcement for load support and earth retention
- **In Geo-membrans,** the best features of different materials are combined in such a way that specific applications are addressed in the optimal manner and at minimum cost. They are flexible, polymeric sheets that have very low hydraulic conductivity (typically less than 10-11 cm/sec) and, consequently, are used as liquid or vapor barriers
- **Geo-composites and geosynthetic clay liners**

The Figure 1.28 presents the different type of geosynthetic states above.

Figure 1.28. Different types of geosynthetics

1.6.3.2. Earthen Brush layer method

Brush layering is a biotechnical method for slope stabilization which consists of inserting live, cut branches or brush between successive lifts or layers of compacted soil (Gray and Sotir, 1995) as shown in Figure 1.29.

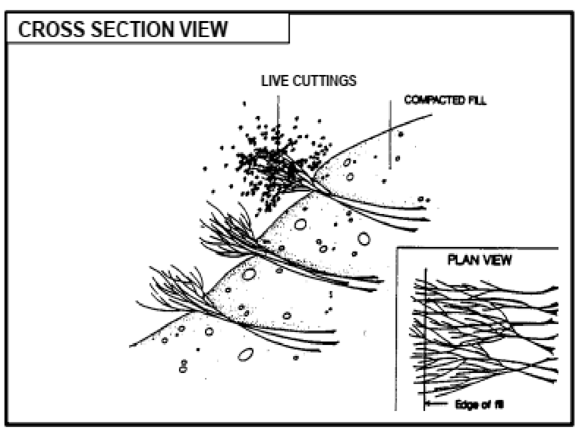


Figure 1.29. Schematic illustration of fill slope stabilization using live brush layers (Gray and Sotir, 1995)

This form a strong barrier, preventing the development of rills, and trap material moving down the slope. In the long term, a small terrace will. According to Gray and Sotir (1995), the method alone will suffice to stabilize a slope where the main problem is surficial erosion or shallow face sliding.

1.6.4. Soil nailing technique

Soil nailing is a ground improvement technique that, in recent years, has been widely used all over the world to support excavation, slopes and retaining walls. Granular soils are among the suitable soils for this technique, but they must be dense and naturally cemented sands and gravel above the water table, not loose materials. This technique which the main element are nails, facing, bar-facing connection or plates as presented in Figure 1.30 is very useful for cohesionless soils because length and spacing of nails are engineered to address all potential modes of failure (planar/circular, deeper/shallower), and its facing acts as a surface protection to avoid particles movement.

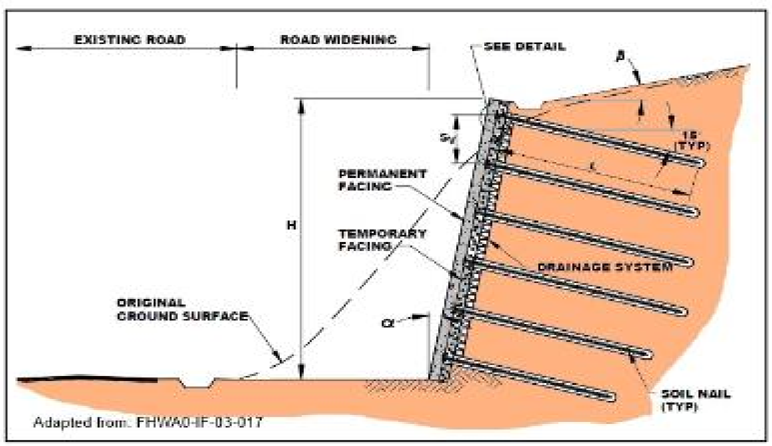


Figure 1.30. Soil nailing wall on a hill side steep cut slope of road section (FHWA)

1.6.5. Shotcrete for slope stabilization

Shotcrete is a concrete which is conveyed through a pressurized hose to a nozzle at a high velocity onto the receiving surface to form a structural or non-structural component of buildings as presents the Figure 1.31, and it is a process of simultaneous compaction, condensation and hardening of concrete (Qiao and Zhou, 2017).

The application of shotcrete for slope stabilization is a common solution to help prevent an excavated (cut) slope from eroding, for either in temporary or permanent slopes. It’s relative ease of constructability and cost effectiveness (Buesing corp, 2021).

Figure 1.31. Pumping shotcrete

The main component is a concrete made of Portland cement, aggregates and water. Sometimes, other ingredients are added to improve the mechanical properties, workability, and pumpability of shotcrete. They include silica fume, admixtures, accelerators, fibers etc. (Qiao and Zhou, 2017). According to the individual site requirements (slope, excavation …) the initial strength development is defined as three different J-classes according to EN 14487-1. In a J2 shotcrete, typically applied in overhead applications, the sprayed concrete reaches a very early strength up to 1.0 – 1.5 MPa to be achieved within the first one to two hours (Lindlar et al. 2020).

Apply a shotcrete onto slope surface can be done by 2 mix methods: the dry mix process where a premixed cement and aggregate is pumped through the hose and water is added to this mixture at the nozzle. while in the wet-mix process, all constituents (cement, aggregate and water) are mixed and then pumped to the nozzle.

Using either dry or wet process, to spray the concrete at high pressure required performant pumping equipment. But it is also important to note that, the final quality is strongly affected by experience of nozzleman.

According to the type of material forming the slope, shotcrete can either be designed with or without a steel mesh or fabric. The goal when using a steel wire mesh is to keep the soil stable when spaying the concrete. The design thickness of the shotcrete may vary per project depending on the project purpose, temporary or permanent design life, soil type, and owner/agency. Comparing it with slope vegetation, it far more cost-saving, and there is no need to worry about the falling off of plantings with its.

Conclusion

The main objective of this chapter was to make a review on the pozzolanic soil and to highlight the main features governing cut slope stability in such type of soils with final aim to propose a technical and economic method of stabilization. As a result, pozzolanic soil, which coming from volcanic projections, has characteristics and behaviour similar to any other granular soil with or without cohesion according to the type of deposit encountered. This lead the slope stability concept to be reviewed for granular soil. Thus, the main types of slopes, especially excavated slopes, were presented, underlining the conditions for a good design and especially the critical conditions, i.e. those leading to a low safety factor. Thereafter, internal parameters such as shape, texture, orientation of particles as well as external parameters like rainfall and earthquakes were presented as greatly influencing the stability of this type of soil. Being essentially superficial, slope failure is manifested by a sliding or rolling of particles and/or a flow or sliding of mass soil. Subsequently, analysis methods of limit equilibrium and finite element were developed and this was followed by a presentation of retaining walls, bioengineering method, soil nailing and shotcrete method as effective stabilization measure to superficial instabilities.

CHAPTER 2 METHODOLOGY

Introduction

As seen in the previous chapter, analysing the stability of slopes requires knowledge of the type of material in place (its properties), the geometry of the slope, the modes of failure and possible causes, and the use of well-established analysis methods in order to verify the stability and to propose solutions in case the slope 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 analyse a cut slope of pozzolana on the Bangangté - Foumbot - Bamendjing - Galim road section in order to confirm their instability and to propose a technical and economic solution for stabilization. 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 slope. The approach for the determination of the minimal safety factor for comparison will then be presented, followed by the presentation of the numerical analysis software. The possible remedial solutions to be implemented in the numerical model will be specified and finally, the formulation of comparison criteria for the choice of the optimal solution will close this chapter.

2.1. Site recognition

The general recognition of the site requires a documentary research with the aim of defining the physical characteristics of the site (location, climate, relief, geology and hydrology) as well as the social and economic characteristics (demography, agriculture).

2.2. Site visit

After recognition of the location and the physical characteristics of the project site through documentary research, a site visit will be made to get a better idea of the state of the place. The observation that will be carried out will allow to understand the problem of slope instability that generates the detachment of the pozzolana grains and their distribution on the roadway.

2.3. Data collection

The collection of geotechnical and geometric characteristics of selected pozzolana cut slopes on a road section from **48+900 to 51+700** will enable a two-dimensional modelling and analysis using numerical methods later described in this chapter.

2.3.1. Geotechnical data

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 and mechanical characteristics of the material forming the slopes.

2.3.2. Geometric data

The geometric data were also given by the same company. These data present the cross-sections of the typical profiles of the excavated area. The inclination angles of three cases of pozzolana cut slopes to study and the width of the berm are also specified.

2.4. Determination of the minimum safety factor for verification

In Cameroon, the reference standards and guidelines used for geotechnical problems analysis and verification are based on the Eurocodes (**EC7**) and French annexes in operation. This norm does not provide any inequality to be satisfied to check the overall stability; nor any calculation model given. However, to verify if the slopes of this study are stable, a value of safety factor to reach can be obtained using the partial factors of actions, resistances and materials strength of the design approach 3 (DA3) which is the most critical design approach for slope stability. Table 2.1 presents the partial factors in this design approach.

Table 2.1. Partial factors in Design Approach 3 (Eurocode 7)

Actions	Symbol	Value
Permanent unfavourable	$\gamma_{G,\, unf}$	1.00
Permanent favourable	$\gamma_{G,fav}$	1.00
Variable unfavourable	$\gamma_{Q,\, unf}$	1.30
Variable favourable	$\gamma_{Q,fav}$	0.00
Soil parameters	**Symbol**	**Value**
Internal friction angle	γ_{φ}	1.25
effective cohesion	$\gamma_{c'}$	1.25
Undrained shear strength	γ_{cu}	1.4
Weight density	γ_{γ}	1.00

Therefore, in order to obtain this minimum safety factor, the infinite slope failure illustrated in Figure 2.1 was assumed. It was chosen for its simplicity and because the failure mechanism of pozzolanic soils (which is granular material) is in general a shallow slope failure which is often parallel to the slope surface.

Figure 2.1 represent an infinite inclined slope with a sliding block of thickness **DC = b** and with a slip surface (EF) parallel to ground at depth **z**.

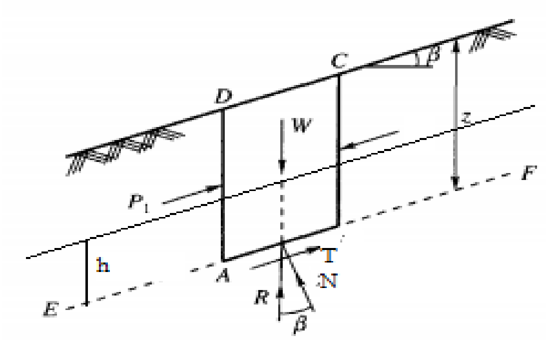


Figure 2.1. Infinite slope in cohesionless soil

In order to be in safe condition, the design destabilizing action $\mathbf{S_d}$ (weight component causing sliding) should be less than the design resistance force $\mathbf{R_d}$.

$$S_d < R_d \qquad (2.1)$$

Considering the equilibrium of forces **W, T, N;** it can be obtained: $\mathbf{N = W\cos\beta}$ and $\mathbf{T = W\sin\beta}$

The weight of the block of soil is expressed as $\mathbf{W = \gamma_{soil}\, z\, b\cos\beta}$

Replacing the expression of W in the normal force N and tangential force T we obtain:

$\mathbf{N = \gamma_{soil}\, z\, b\cos^2\beta}$ and $\mathbf{T = \gamma_{soil}\, z\, b\cos\beta\sin\beta}$

The destabilized action is expressed as: $\mathbf{S_d = \gamma_G\, T = \gamma_G\, W\sin\beta = \gamma_G\,\gamma_{soil}\, z\, b\cos\beta\sin\beta}$

While the design resistance is given by $\mathbf{R_d = \gamma_G\, N\tan\varphi'_d = \gamma_G\,\gamma_{soil}\, z\, b\cos^2\beta\,(\tan\varphi'_k)/\gamma_M}$

In DA3, $\mathbf{\gamma_M = 1.25}$ (in drained condition), substituting in Equation (2.1):

$$\gamma_G\gamma_{soil}\, z\, b\cos\beta\sin\beta < \gamma_G\,\gamma_{soil}\, z\, b\cos^2\beta\,(\tan\varphi'_k)/1.25$$

It comes from the previous expression: $\frac{\tan\varphi'k}{\tan\beta} > \mathbf{1.25}$, With FoS = $\frac{\tan\varphi'k}{\tan\beta}$.

===> FoS > 1.25

In conclusion, the factor of safety to reach when analysing the pozzolanic slope cases by means for numerical tools must be at least equal to **1.25** in order to satisfy the Eurocodes requirement of stability.

2.5. Numerical analysis of slope stability

2.5.1. Choice and justification of stability analysis methods

For the purpose of this work, both LEM and FEM numerical analyses have been undertaken with the respective commercial software Slope/w product of GeoStudio and Plaxis 2D.

We first choose the limit equilibrium of slices because it is probably the simplest and most used method for slope stability analysis. The developing of numerical versions allows, through iterative procedures, to evaluate the stability of the slopes, with circular and non-circular slip surfaces enabling to perform a large number of calculations in a very short time. Moreover, they have the advantage of providing a graphical output, which shows the geometry of all analysed sliding surfaces and their corresponding FoS.

However, the LEM methods consider forces acting on one or more discrete points along the slip plane assuming that failure occurs instantaneously and that the available shear force is mobilised along the entire slip plane at the same time. As an alternative, we use FEM to overcome these limitations because during deformation and failure it can provide the type and magnitude of displacements in the slope and also provide FoS, which may be different from those obtained with the LEM method since no specific failure surface is defined. These calculation programs are presented as following.

2.5.2. Presentation of GeoStudio 2018 R2

2.5.2.1. Generalities

GeoStudio is an integrated geotechnical software used to solve various soil problems such as landslides, settlement, consolidation, water infiltration in the body of a dam and other geotechnical problems. As it is presented in Figure 2.2. the software comprises a suite of product which are **SLOPE/W, SEEP/W, SIGMA/W, QUAKE/W, CTRAN/W, TEMP/W, AIR/W and**

VADOSE/W. The availability of such quantity of product allows a rigorous analytical capability to diverse geoengineering and earth science problems.

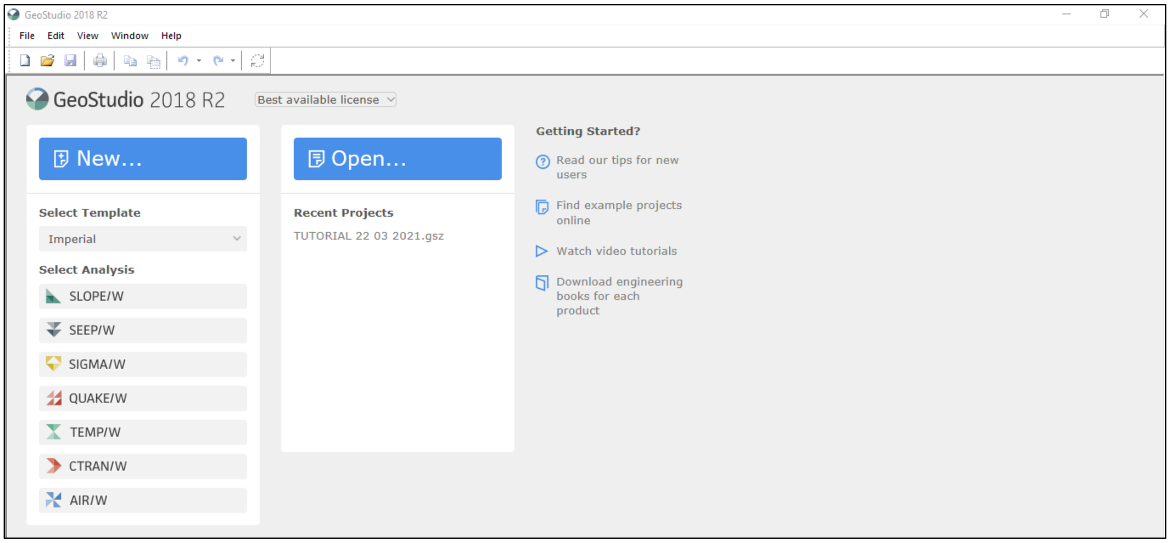


Figure 2.2. Principal interface of GeoStudio 2018 R2

Depending on the type of analysis that is going to be done in this work, the product to be used for analysing the pozzolanic slopes is the SLOPE/W product.

2.5.2.2. SLOPE/W Product Analysis

SLOPE/W was the first geotechnical software product of GeoStudio available commercially since 1977 for analysing slope stability. Developed by Professor Fredlund, it was not specifically designed for earth-reinforced retaining walls, but can be used to analyse the stability of a wedge of soil that has been reinforced with a structural component such as a pre-stressed anchor, a soil nail, geo-fabric or some other material. When integrated to the others suite of GeoStudio products, it opens the door to several types of analyses to a much wider and more complex spectrum of problems including the use of finite element computed pore-water pressures and stresses in a stability analysis.

2.5.2.3. Design methodology in SLOPE/W

The design methodology for slope analysis in SLOPE/W that will be executed is order in terms of four components which are:

- Geometry for the description of the stratigraphy and shapes of potential slip surfaces;

- Soil strength parameters used to describe the soil (material) strength;
- Pore-water pressure to define the pore-water pressure conditions inside the model;
- Reinforcement which included fabric, nails, anchors, piles, walls and so forth.

a. Geometry

Define a geometry is the first step for the analysis. To this aim, the following procedure was adopted.

i. Region drawing

Since the geometry was not complicated, regions were directly drawn in the software instead of being imported from a DXF or DWG file. Having only one material type, regions are connected one to another by points to form a continuum.

ii. Slice discretization

SLOPE/W discretizes the soil mass with slices of varying widths in order to ensure that there is only one soil type at the bottom of each slice. Even if the number of desired slices can be specified by the user, it was adopted for the study the default number of slices which is 30.

iii. Ground surface line

In SLOPE/W, a ground surface line is a necessary feature for controlling what happens at the actual ground surface. Its definition will be done so that, all the trial slip surfaces must enter and exit along this.

iv. Slip surface shape

The analyses were performed for the circular failure model. For this model, there were several methods for defining the shape and position of trial slip surfaces in SLOPE/W, but the entry and exit method was selected.

b. Material strength

In the SLOPE/W, there are many different ways to describe the strength of geotechnical materials either soil or rocks but the most common of them, which is going to be used in this work is the Mohr-Coulomb ‘s model. The Equation (2.2) derived from this model represents a straight line on a shear strength versus normal stress plot.

$$\tau = c + \sigma_n \tan\phi \, . \qquad (2.2)$$

c. Pore pressure

It is important to consider the pore pressure condition in slope analysis because the most realistic position of the critical slip surface is obtained when the effective strength parameters are used in the analysis. Since the water table was not specified in the given data, it would not be considered in our analysis.

d. Safety factor calculation method

There are many methods to calculate the factor of safety using LEM. The one used in this study is the Morgenstern-Price (M-P) methods.

e. Verification and computation

When all the above described steps were performed (geometry, material strength, pore pressure…), SLOPE/W runs several checks to verify the input data using the verify/optimize data command in the Tools menu. If there are no errors, the software computes the factor of safety using the method of slice selected.

2.5.3. Numerical analysis based on FEM

2.5.3.1. Presentation of Plaxis 2D

Plaxis is a finite element package intended for the two-dimensional analysis of deformation and stability in geotechnical engineering projects (Plaxis manual). Developed in 1987, the program consists of four sub-programs: Input, Calculation, Output and Curves.

a. Input program

It is in this sub-program where:

- 2D geometry is defined by the use of points, lines and clusters;
- Boundaries conditions are activated by the vertical boundaries allowing to a model a free movement (up-down) and the horizontal boundary considered to be fixed.
- Material properties containing data sets for soil and interfaces, plates, geogrids and anchors and material model such as Mohr-Coulomb model, the linear-elastic model are defined;
- the meshes are generated inside the studied element by discretizing into finite element containing nodes and stress or gauss points;

- the initial conditions consisting of two different modes which are the generation of initial water pressures and the specification of the initial geometry configuration and the generation of the initial effective stress.

b. Calculation program

This program defines the type of calculation (plastic calculation, consolidation analysis, Phi-c reduction) to be performed and the type of loading or construction stages to be activated during the calculation (staged construction, incremental multiplier and total multiplier) that must be specified after choosing the calculation type.

c. Output program

This program is useful to view all the analysis results generated by the input data and calculations. The main output quantities are the displacements at the nodes and the stresses at the stress points. When the program involves structural elements, the structural forces are calculated in these elements (Plaxis manual). These results concern displacements, strains and stresses, pore water pressure change etc.

d. Curves program

This program is used to generate the stress-strain diagrams, load-displacement curves time-displacement and stress or strain paths of pre-selected point in the geometry (Plaxis manual).

2.5.3.2. Design and analysis procedure in Plaxis 2D

a. Mesh generation

After drawing the geometry in the input window, the 15-node triangular elements were used to generate the mesh and then the default initial conditions of stresses were used in all the 3 slope cases model.

b. Material model

The Mohr Coulomb model was used here as material model. This material model involves six parameters, namely Young's modulus **E**, Poisson's ratio **ν**, the dilatancy angle **ψ** the cohesion **c**, the unit weight **γ**, and the friction angle **φ**.

- The dilatancy angle, **ψ** which controls an amount of plastic volumetric strain developed during plastic shearing and is assumed constant during plastic yielding. Clays (regardless of over-consolidated layers) are characterized by a very low amount of dilation **($\psi \approx 0$),** For non-cohesive soils (sand, gravel) with the angle of internal friction **$\varphi > 30°$** the value of dilation angle can be estimated as **$\psi = \varphi - 30°$** (Bartlett, 2011).

- The Young's modulus and the Poisson's ratio are the soil elastic parameters. The Young's modulus, E is defined either by the initial modulus of elasticity **E_0** or by the secant modulus at 50% of the compressive strength as shown the Figure 2.3 while the Poisson's ratio **ν** is evaluated through the at rest earth pressure coefficient **$K_o = \nu\ (1- \nu)$** for gravity loading with values between **0.3 and 0.4**.

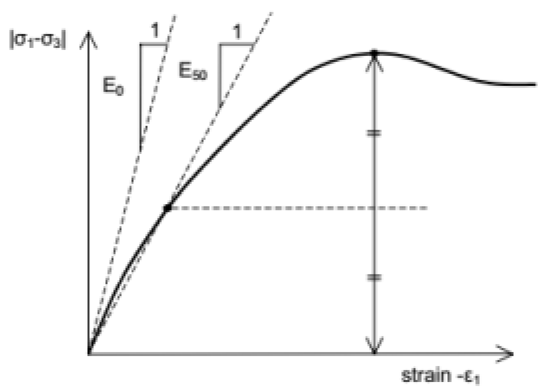


Figure 2.3. Young Modulus definition

Although, these parameters have a significant influence on the computed deformations prior to failure, they have very little influence on the predictions of safety factor of slopes (Griffiths and Lane, 1999). In fact, the most important parameters in the finite element analysis of slope stability are angle of friction **φ '**, cohesion **c**, and unit weight **γ**. Since no laboratory tests were performed to obtain the soil elastic parameters for our case study**,** the values of **E** and ***ν*** to be used will be obtain by means of a literature review.

Therefore, the values of elastic parameters of Table 2.2. given by the manual of estimating soils properties for foundations design will be used.

Table 2.2. Typical values of soils elastic parameters (Kulhawy and Mayne, 1990)

Typical ranges of Young modulus (E)			**Poisson ratio (v)**
Soils	**Consistency**	**Values (MPa)**	**Values (-)**
Clay	Soft	05 - 25	0.2 - 0.4
	Medium	15 - 40	
	Stiff	40 - 100	
Sand	Loose	10 - 25	0.1 - 0.3
	Medium	30 - 50	
	Dense	50 - 81	0.3 - 0.4

c. Analysis type

The stability analysis of the cut slopes of this study is conducted by the Phi-c reduction calculation type with the incremental multiplier loading input. In this approach, the tan φ and c characteristics of the soil are gradually reduced until failure occurs. The total coefficient called reduction coefficient ΣM_{sf} is used to define the value of the soil characteristics at a given stage of the analysis.

$$\sum M_{sf} = \frac{\tan \phi_{given}}{\tan \phi_{reduced}} = \frac{c_{given}}{c_{reduced}} \qquad \textbf{(2.3)}$$

Where the characteristics marked "given" refer to the initial values of the material properties and the ones marked "reduced" refer to the reduced values used during the analysis. Once the failure mechanism is reached, the factor of safety is given by:

$$\textbf{FoS} = \frac{available\ strength}{Strength\ at\ failure} = \Sigma M_{sf}\ \textbf{value at failure} \qquad \textbf{(2.4)}$$

2.6. Stability analysis of reinforced slopes

With the final objective of proposing the optimal stabilization measure in the case the result shows an unsafe situation, the reinforced slope was modelled and analysed using the above presented numerical analysis tools. The main type of reinforcement that can be modelled in this software includes anchors, nails, piles, plates and geosynthetics. There is also a user defined option in which the user can feel free to model a type of reinforcement that is particular and specific to its problem. The slope case of this study was stabilized using 3 remedial measures namely Shotcrete, Soil nailing technique and Geocell slope protection. The following section presents how they were designed and modelled.

2.6.1. Shotcrete

Recalling that shotcrete is the technique in which concrete is sprayed at high pressure on an inclined surface (or not) to form a homogeneous, flexible and durable cover protecting slopes against surface erosion (Figure 2.4). Stabilization is based on the principle of penetration of its fine components into existing surface voids and cavities of loose rocks/soils. The standards defining the requirement of the production and execution of this technique are given in the EN 14487.

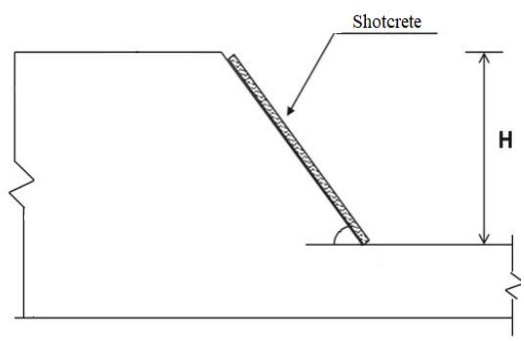


Figure 2.4. Schematic view of shotcrete on slope

The main element of shotcrete that needs to be modelled is the concrete. The properties of concrete depend highly on the mix design of material (aggregate grading, water cement ratio, admixtures…). Qiao and Zhou (2017) established some best practices for using shotcrete for slope stabilization especially the mechanical parameters to obtain a performant shotcrete such as unit weight, compressive strength, modulus of elasticity, flexural strength, shrinkage, etc. In this study, we will focus on some of those parameters to design the shotcrete to stabilize the pozzolan cut slope.

The modelling was done in SLOPE/W by simply input the reinforced concrete unit weight in the high strength material model. This material model does not allow slip surfaces to cross the structure. Therefore, all valid slip surfaces will be the ones passing above and below the formed shotcrete mattress.

In Plaxis, there is a possibility to include more specific concrete characteristics allowing to realistically model this technique. The modelling was done by means of plate element. This option allows to enter the young modulus obtained from the compressive strength which is not similar to the one of conventional concrete. According to Zhang (2014), the compressive strength of shotcrete at 28 days ranging from **27.5** to **68.9 MPa** have been commonly reported in field construction. The compressive strength for this study was taken in this range and was used to compute the concrete elastic modulus using the expression given by the EC2:

$$\mathbf{Ec = 22\left(\frac{f_{Ck}+8}{10}\right)^{0.3}} \qquad \textbf{(2.5)}$$

Obtaining the value of E_c, we therefore compute the axial stiffness **EA = Ec*d*1ml** and the Flexural stiffness **EI =** $\frac{E_c*d^3*1ml}{12}$ with **d** the thickness of sprayed concrete which according to the FHWA ranges from **50mm** to **600mm.**

In addition to the factor of safety calculation, the deformation of the slope has been also determined through Plaxis 2D code in order to check whether the horizontal and total displacements have been reduced by this technique.

2.6.2. Soil nailing

In the aim of using the soil nailing method to stabilize the unstable slope, a detail design procedure and analysis must be clearly established. The following sections presents how they were done.

2.6.2.1. Design procedure

In order to optimize the factor of safety using the soil nailing technique to stabilize the pozzolan cut slope, we will follow a design procedure which include a series of charts developed by FHWA (2015). This design chart aids to provide preliminary nail length and maximum design tensile forces of the nails. The parameters that need to be selected before are nails inclination **i**, FoS of overall stability, drill hole diameter $\mathbf{D_{dh}}$, the wall height **H**, angle of back slope **β**, face batter **α**, vertical and horizontal spacing between bars $\mathbf{S_v}$**,** $\mathbf{S_h}$ as presented in Figure 2.5.

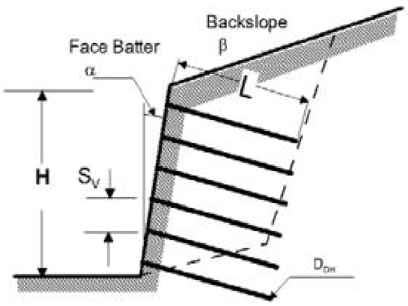


Figure 2.5. Soil nailing scheme (Mohamed, 2010)

After selecting the previous parameters and the soil properties, we calculate the normalized bond resistance μ expressed as

$$\mu = \frac{q_u \cdot D_{DH}}{FoS \cdot \gamma_S \cdot S_H \cdot S_v} \qquad \textbf{(2.6)}$$

where $\mathbf{q_u}$ is the ultimate bond strength obtained using the table 2.3. With this value, we obtain the normalized soil length **L/H** and the normalized max tensile force $\mathbf{t_{max}}$ using the chart presented in Figure 2.6.

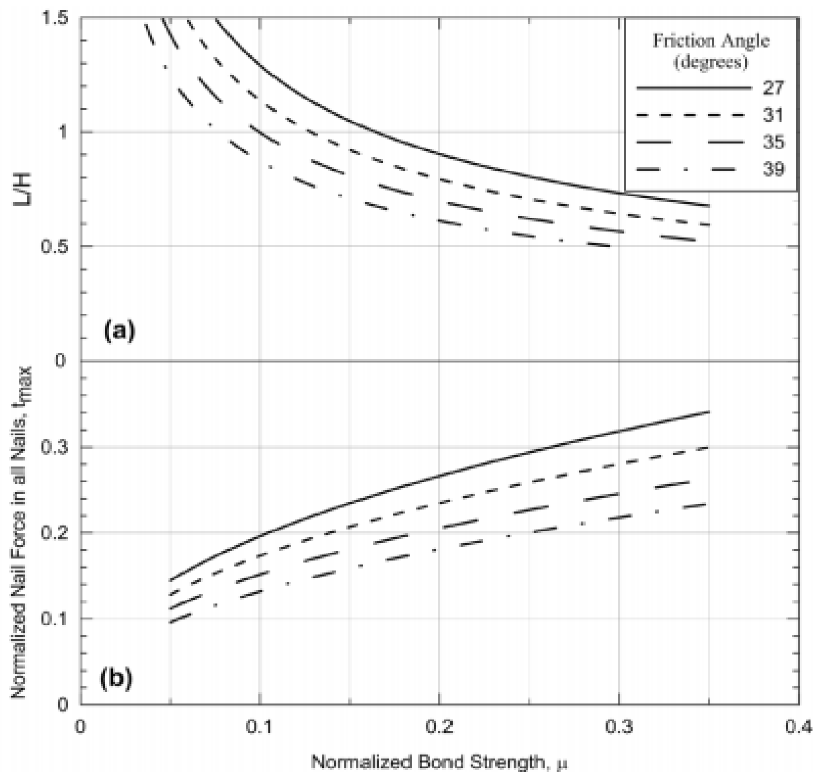


Figure 2.6. Design chart of soil nailing (FHWA, 2015)

These charts were developed for a range of α, β, μ values and the effective internal angle φ'. Chart (a) provides the necessary normalized nail length, L/H, required to achieve a global safety factor of $FoS._G$ = 1.35 while Chart (b) provides the normalized maximum design nail force, $\mathbf{t_{max}}$, as a function of normalized bond strength μ. After obtaining $\mathbf{t_{max}}$, we calculate the soil nail maximum tensile force by using the following expression:

$$\mathbf{T_{max} = t_{max} . \gamma_S . S_V . S_H} \qquad \mathbf{(2.7)}$$

Using the data in Table 2.3, the value of nail pull-out resistance can be obtained applying the following expression:

$$\mathbf{Q_u = \pi . D_c . q_u} \qquad \mathbf{(2.8)}$$

Table 2.3. Estimated bond strength of soil Nails in cohesionless soils (Elias and Juran, 1991)

Material	construction Method	Soil/Rock type	Ultimate bond strength, q_u (kPa)
Cohesionless Soils	Rotary drilled	Sand/gravel	100 - 180
		Silty sand	100 - 150
		Silty	60 - 75
		Piedmont residual	40 - 120
		fine colluvium	75 - 150
	Driven casing	Sand/gravel	
		Low overburden	190 - 240
		high overburden	280 - 430
		Dense moraine	380 - 480
		Colluvium	100 - 180
	Augured	Silty sand fill	20 - 40
		Silty fine sand	55 - 90
		silty clayey sand	60 - 140
	Jet Grouted	Sand	380
		Sand/gravel	700

2.6.2.2. Modelling and analysis

There are three mains options included in SLOPE/W when analysing slope reinforced by nails. The one adopted for this problem is the "Factor of Safety Dependent" Yes option with the distributed force option selected and the anchorage option sets to 'Yes'.

In Plaxis 2D the nails and facing were modelled as Plate elements. The most important input material parameters for plate elements are the flexural rigidity (EI) and axial stiffness (EA). They can be determined using the following expressions:

$$\mathbf{EI} = \frac{E_{eq}}{s_h}\left(\frac{\pi D_{DH}^{2}}{4}\right) \qquad \mathbf{(2.9)}$$

$$\mathbf{EA} = \frac{E_{eq}}{s_h}\left(\frac{\pi D_{DH}^{4}}{64}\right) \qquad \mathbf{(2.10)}$$

With D_{DH} the drilling hole diameter, S_h the nail horizontal spacing and E_{eq} the equivalent elastic modulus of grouted soil nail expressed as:

$$\mathbf{E_{eq}} = \mathbf{E_n}\left(\frac{\mathrm{A_n}}{\mathrm{A}}\right) + \mathbf{E_g}\left(\frac{\mathrm{A_g}}{\mathrm{A}}\right) \qquad \mathbf{(2.11)}$$

With $\mathbf{E_g}$ the modulus of elasticity of grout material,

$\mathbf{E_n}$ the modulus of elasticity of nail,

$\mathbf{A = 0.25\pi\, D^2_{DH}}$ is the total cross-sectional area of grouted soil nail;

$\mathbf{A_n = 0.25\pi\, d^2}$ is the cross-sectional area of reinforcement bar (d = bar diameter);

$\mathbf{A_g = A - A_n}$ is the cross-sectional area of grout cover.

2.6.3. Geocell slope protection

2.6.3.1. Basic mechanism of geocell

Geocell provide stability to the top soil layer through confinement of the fill material and protecting the material below from being moved by wind or water. The soil reinforcement effect is mainly manifested in the lateral restraint of the cell to the soil (see Figure 2.7.) including the frictional effect of the cell sheet on the filled soil and the tightening effect of the three-dimensional space structure formed by the cell tension on the filled soil (Zhao and Yin, 2018).

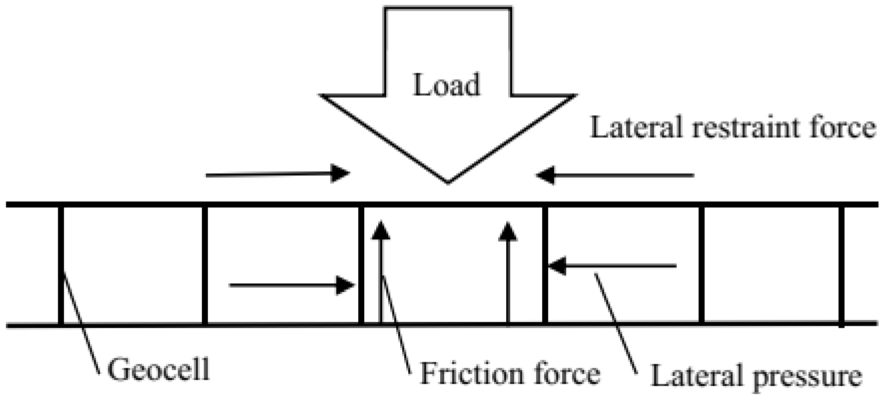


Figure 2.7. Lateral confinement of geocell to its internal soil (Zhao and Yin, 2018)

The main components are geocell, anchors to fix the geocell into the soil and the earth filling material. After construction, vegetation can easily growth increasing the stability of the system by their roots.

2.6.3.2. Design procedure

a. In LEM

For designing the geocell mattress with an infill soil above the pozzolanic soil, geocell layer is treated as a layer of soil with cohesive strength greater than the encased soil and angle of internal friction same as the encased soil. Madhavi Latha, (2011) explained this concept saying that geocells provide all-round a confinement to the soil due to the membrane stresses in the walls of geocells and because of which apparent cohesion is developed in the soil.

The total cohesion of the composite material infill soil + geocell is given by $\mathbf{c_t = c + c_g}$

The additional cohesive strength due to geocell layer can be obtained as:

$$\mathbf{c_g = \frac{\Delta\sigma_3}{2} \cdot \sqrt{k_P}} \qquad \textbf{(2.12)}$$

With $\mathbf{K_P}$ the passive infill soil thrust on the geocell, and $\boldsymbol{\Delta\sigma_3}$ the additional confining pressure due to the membrane stresses (geocell) expressed by Equation 2.13 and derived by drawing Mohr circles for the unreinforced and reinforced soil samples shown in Figure 2.8.

$$\boldsymbol{\Delta\sigma_3 = \frac{2M}{D_0}\left[\frac{1-\sqrt{1-\varepsilon_a}}{1-\varepsilon_a}\right]} \qquad \textbf{(2.13)}$$

With:

ε_a = axial strain at failure

M = modulus of the membrane

D_o = initial diameter of geocell.

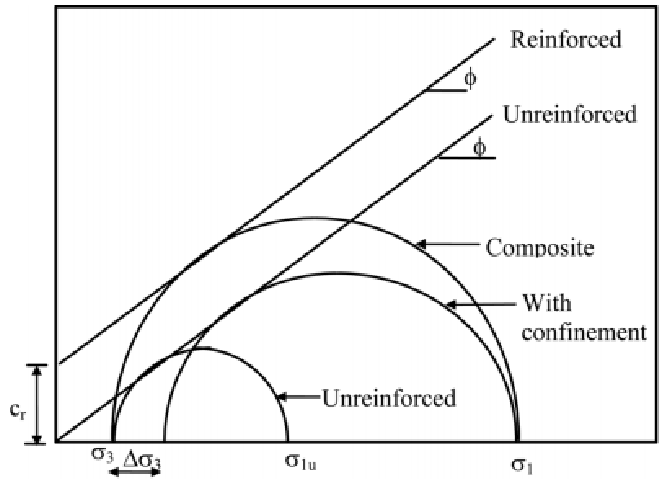


Figure 2.8. Mohr circles for calculating the strength improvement due to geocell reinforcement (G. Madhavi Latha, 2011)

b. In FEM

According to Madhavi Latha (2011), the composite model of geocell + infill soil can be also modelled with geogrids with appropriate tensile strength. It is therefore this model that will be used in finite element in our slope case.

The correspondent young modulus of the composite **E_g** is expressed in terms of the secant modulus of the geocell material and the Young's modulus parameter of unreinforced sand **E** as:

$$\mathbf{E_g} = \boldsymbol{P_a[E + 200.M^{0.16}]\left(\frac{\sigma_3}{P_a}\right)^{0.7}} \qquad \textbf{(2.14)}$$

With **P_a** the atmospheric pressure taken as 101.325 kPa.

This relation was proposed by Madhavi Latha (2000) and Rajagopal et al. (2001) based on triaxial compression tests on geocell encased sand.

The layer of geogrid is maintained onto the slope surface using steel bar of 8 mm minimum diameter.

2.7. Comparative study of adopted stabilization methods

In order to make a choice of the optimal method for the stabilization of the unstable banks of Pozzolana, a comparative study of the different methods listed in the previous section will be carried out, based on 3 main aspects.

2.7.1. Technical point of view

From a technical point of view, the comparison of different solutions will be made according to several criteria, such as the resistance of the structure, their durability, the execution time and the construction procedure of each of them.

The structure resistance is based on the evaluation of the safety factors improved of each method on the cut slope while durability will allow the evaluation of the life span of each structure. The execution time of each structure will consider the time required for each stage of construction; the implementation will take into account the construction procedures, the quality of used materials and the human resources.

2.7.2. Financial point of view

The comparison criterion here is the cost of each technology. The objective is to establish the construction costs of the various selected works precisely of the costs of the tasks and materials of each method. It will be evaluated according to the public works contract of the project for shotcrete and geocell slope protection and according to the cost established by FHWA for soil nailing technique.

2.7.3. Environmental point of view

The two previous aspects are enough to make the choice of the optimal method according to the main objective set for this work. However, it is very important but also essential that the method be respectable for the environment. Thus, the aim here is to ensure that the solution adopted is friendly to the environment by evaluating the amount of CO_2 based (on data given by ADEME 2007) that each method discharges into the environment and choosing the lowest one.

Conclusion

At the end of this chapter, it was question of developing a working methodology in order to solve, using a technical and economic solution, the problem of instability of pozzolana cut slopes along the Bangangté - Foumbot - Bamendjing - Galim road section, precisely in the area of 48+900 to 51+700 of this project. As selected stages, it was highlighted that a general recognition of the site, followed by a visit, will be essential in order to understand the extent of the problem. Using the partial factors of Eurocode 7 design approach 3 (EC7, DA 3) in the infinite slope failure theory, the safety factor 1.25 was obtained as the minimum value to be reached in the different numerical analyses for the slope to be characterised as stable. Subsequently, the different steps of modelling and analysis without and with reinforcement that are to be used by means of the SLOPE/W product of the GeoStudio software and Plaxis 2D were described in detail. Finally, selection criteria based on the technical, economic and environmental aspects will allow the choice of the optimal stabilization method among those presented in this chapter, which are Shotcrete, soil nailing and a Geocell slope protection

CHAPTER 3 RESULTS OF SLOPE STABILITY ANALYSIS IN A POZZOLAN DEPOSIT

Introduction

The road construction project entitles "*Route de desenclavement du Bassin Agricole de l'Ouest lot 2*" aims to improve the movement of agricultural products within and outside the Cameroon West region. The construction of this road includes the realisation of several works such as excavations on soil deposits generating cut slopes. The cut slopes on which this study is based are those made in a pozzolan deposit located from 48+900 to 51+700 in the town of Foumbot. The main objective of this chapter is the display of data obtained from the implementation of methodology. This would be followed by a processing of data to obtained results. To this end, the project site will be presented at the outset by giving its general physical characteristics (location, climate, relief, hydrology) and its socio-economic characteristics m;(demography, agriculture, etc.). Then, the description of the site through the presentation of the observation results will complete the study. This is followed by the presentation of the project in terms of geotechnical, geometric and topographical aspects, as well as the characteristics of the materials used. The numerical analysis of 3 slope cases without and with reinforcements such as Geocell, shotcrete and soil nailing will then be carried out using two commercial software packages namely SLOPE/W based on LEM and PLAXIS 2D based on FEM. Finally, a comparative study of the three stabilisation measures based on technical, financial and environmental criteria will complete this work.

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 characteristics (geographical location, climate, relief, geology...) and the social and economic characteristics (population, agriculture).

3.1.1. Physical Characteristics

This section concerns the geographical position of the road construction project as well as the physical features characterizing the area.

3.1.1.1. Geographical location of the project

The project entitled "*Routes de desenclavement du Bassin Agricole de l'Ouest* " is a social project designed to facilitate the evacuation of agricultural products. Situated in the western region of Cameroon precisely in the eastern zone, lot 2 of this road project links over 107 km the towns of Bangangté-Foumbot-Bamendjing-Galim, this subdivided into 2 sections as shown in the 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.

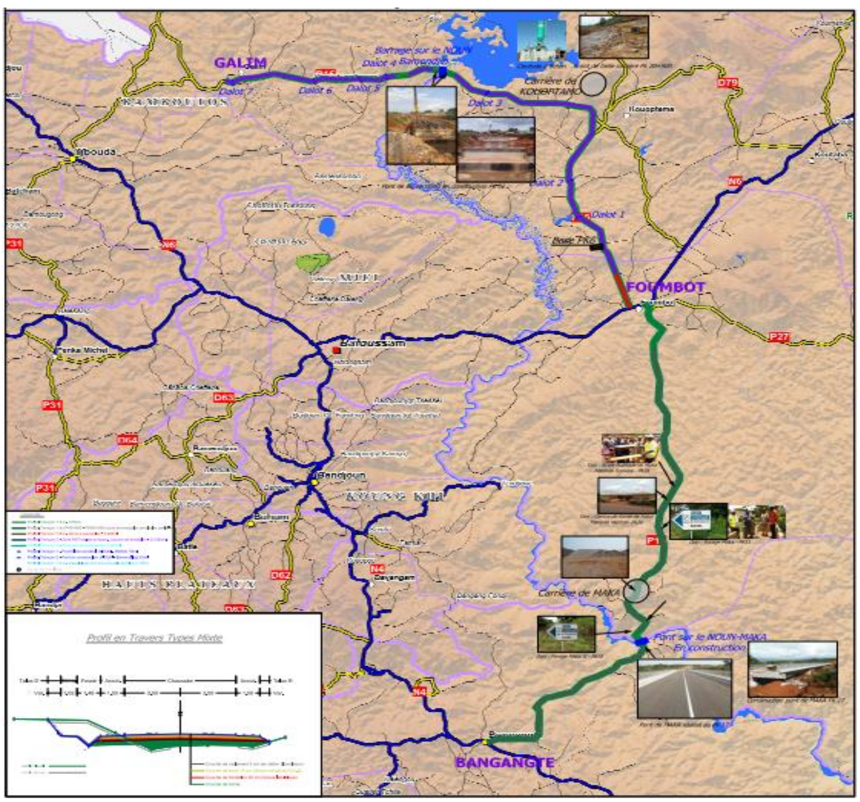


Figure 3.1. Localization of the study area (Project document)

3.1.1.2. Climate

The climate of the west region 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 2500 and 5000 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

Globally, the Western region of Cameroon presents a mountainous relief 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 relief 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 studied 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 relief 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

3.1.2.1. Population

According to the last census of the population of Cameroon in 2005, the western region had about 1720047 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 76486 inhabitants with 38891 women and 37595 men. With a growth rate of 2.6%, this would give a population of 90406 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 presence of soils rich 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.

3.2. Site description

The project site visit began with a presentation of the life base located at KP 6 of section 2 of the project. The area affected by the problem of pozzolana slope instability is located in section 1

(Foumbot-Bangangté road section) of the project from **48+900 to 51+700**, i.e. over a length of **4.6 km**.

Two types of pozzolana deposit were encountered. The black pozzolana which is the most widespread, and also a red deposit of pozzolana. In some places, the pozzolana appears as a purely frictional sand deposit, precisely the black one. However, other deposits seem to be weakly welded rock masses. This concerns mainly red deposits or those made up of a mixture of the 2 colours as shown in Figure 3.2.

Figure 3.2. Cut slopes in: a) Black pozzolan deposit, b) Red pozzolan deposit

The Figure 3.3.a) is a visual evidence of the problem of slope instability manifested by the spreading of pozzolana grains on the pavement while Figure 3.3. b) shows a superficial slope failure in the black pozzolan deposit.

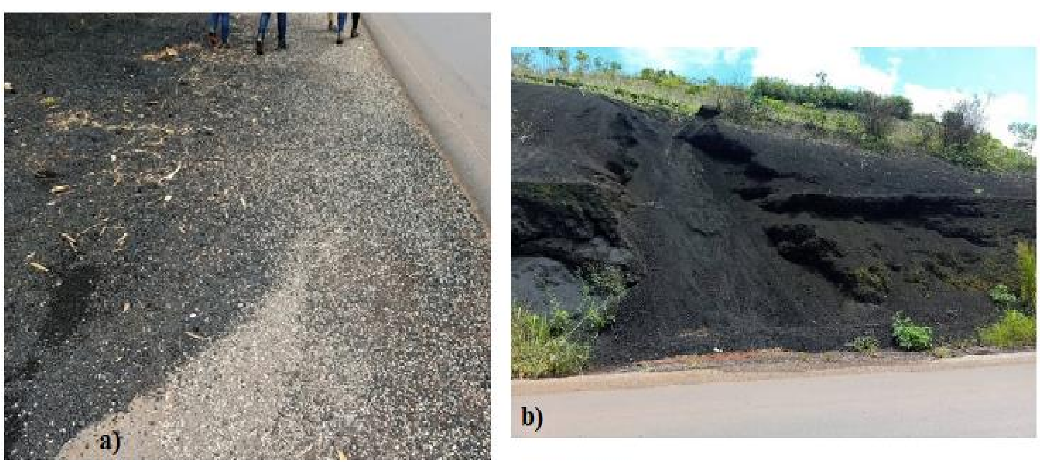


Figure 3.3. Instability manifestations in the black pozzolan deposits

This instability endangers several structures present in the area such as some electric poles and houses. Some of these houses that are too close to the road right-of-way will have to be compensated by the Cameroonian State and subsequently destroyed. Others far enough away from the right-of-way will not be destroyed. It will therefore be necessary to ensure the safety of the latter.

Before any approach to solving the problem of pozzolanic slope instability, it is important to determine the possible cause(s) of the problem. By excavation, the pozzolan deposits have been denuded, leaving their constituent particles vulnerable to environmental actions. As presented in section 3.1.1.2. the commune of Foumbot has a high rainfall value (about 2,500 to 5,000 mm of rain per year according to IRAD (2013)). During the rains, the pozzolan grains are transported at the toe of slopes, thus spreading over the pavements as shown in Figure 3.3.a. In addition, the action of the wind would contribute to the loss of inter-particle contact, causing them to detach from the slopes.

We can therefore say that the instability observed on pozzolan slopes is probably due to the erosion of its surface layer by weathering agents such as rain and wind that occurs in the area. It would therefore be important to cover the surface of this slope with an appropriate technique in order to eradicate this problem.

This visit also provided an opportunity to learn about the local use of the pozzolana. Local residents use it for the manufacture of cinder blocks.

Figure 3.4. The local use of pozzolan for houses construction

It was also noted the presence of the company CIMENCAM in this zone, for the exploitation of a pozzolana quarry to supply the NOMAYOS industrial clinker grinding plan in Yaoundé.

3.3. Presentation of collected data

Data collected from the company in charge of construction work are two types: the geotechnical data and geometric data.

3.3.1. Geotechnical data

The various soil samples for laboratory analysis were taken at point **49+600 m** of the road project. The characteristics that emerged are as follows.

3.3.1.1. Soil stratigraphy in the cutting zone

The most common stratigraphy encountered in the area is the one consisting of 2 layers of material which are

- A deposit of black pozzolan
- A yellowish clay

The stratigraphy where sampling for laboratory analysis was performed (at point **49+600 m**) is presented in the following Figure 3.5:

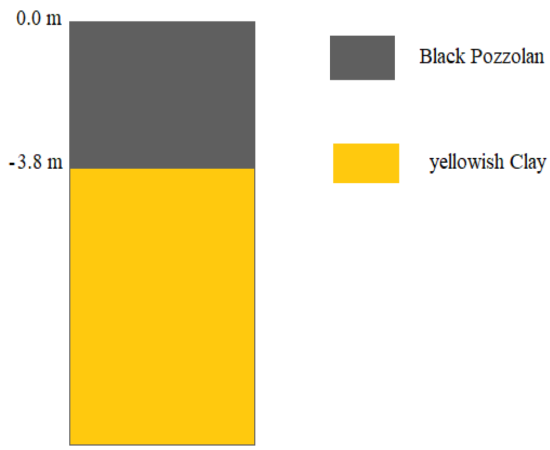


Figure 3.5. Soil stratigraphy of the sampling zone (project document in appendix)

The type of pozzolana studied here is the black one which is purely frictional. The choice was made on the material with the lower geotechnical characteristics (the reddish pozzolana has higher characteristics than the black one). Also, given that in some places the two types are mixed, the choice has been adopted in order to have a homogeneous slope.

3.3.1.2. Geotechnical parameters of each soil layer

This part concerns the geotechnical parameters necessary for the modelling and analysis of the different slope cases. These parameters are obtained through two main laboratory tests which are the modified proctor test in accordance with standard NF 94 – 093 for the determination of **dry unit weight** and the **optimal water content**; and the direct shear test (DST) used to obtain the soil **effective cohesion** and the **internal friction angle**. The parameters obtained are recorded in the Table 3.1. and the sheet of the different tests in the appendix.

Table 3.1. Geotechnical parameters of the studied place

Materials	Parameters			
	c' [kN/m²]	φ [°]	γ_{dry} [kN/m³]	γ_{sat} [kN/m³]
Black pozzolana	0	35°	12.7	15.1
Yellowish clay	30	25°	15.5	18.79

The sieve analysis of pozzolan in Foumbot city gives sand contents higher than 70% with 15 to 25% of silt and a very weak clay content (0 to 10%) (IRS CAM). By these data, pozzolan represents a well graded granular material.

In addition, according to the result of DST shown in appendix, Pozzolan present a hardening behaviour (the ultimate shear strength is equal to the maximum) making it a loose granular material. Since pozzolan is a well graded loose granular soil, the values of Young modulus useful for analysis in Plaxis 2D was chosen for the corresponding material in Table 2.2.

These elastics parameters for both soil layers are assign in the Table 3.2.

Table 3.2. Soils elastic parameters of the project

Materials	Young modulus E (kN/m²)	Poisson ratio *ν* (-)
Pozzolan	13000	0.3
Clay	10000	0.35

3.3.1.3. Another site constraint

Beyond the site properties listed above, there are also some others which are important for the modelling and analysis.

- **Water level**: not specified, so it will be assumed no water table in the model; in fact, pozzolan is a highly drained material;
- **Earth Thrust**: not given; assuming Pa = 0
- **Surcharge:** No surcharge (permanent and variable) at the crest of the slope since we are exclusively in cutting zone and we does not consider the presence of houses Q= 0

3.3.2. Geometric data of the project

The geometric data of the project concerns the slope angle, berm height and width of 3 types of slope cases present on the site as shown in Figures 3.7 to 3.9. These slopes differ essentially in the inclination of the pozzolanic part and with the height of the berms as presented in the Table 3.3.

Table 3.3. Geometric parameters of project

Slope cases	Pozzolanic part		Clayey part		Berm Height	Berm width
CASE 1	2H/3V	56°.31	2H/3V	56°.31	5 m	2.5 m
CASE 2	3H/2V	33°.69	2H/3V	56°.31	4 m	
CASE 3	5H/2V	21°.80	2H/3V	56°.31	4 m	

These slopes are described by indicating the ratio between the depth or horizontal part (H) and the height or vertical part (V) of the slope. For example, the case of slope 1 2H/3V is a slope that corresponds to 2m depth and 3m height as presented in Figure 3.6.

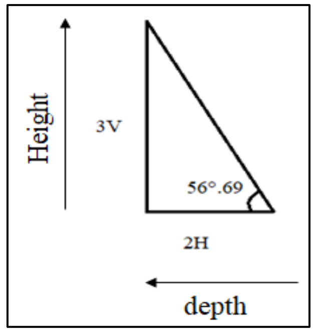


Figure 3.6. H/V ratio

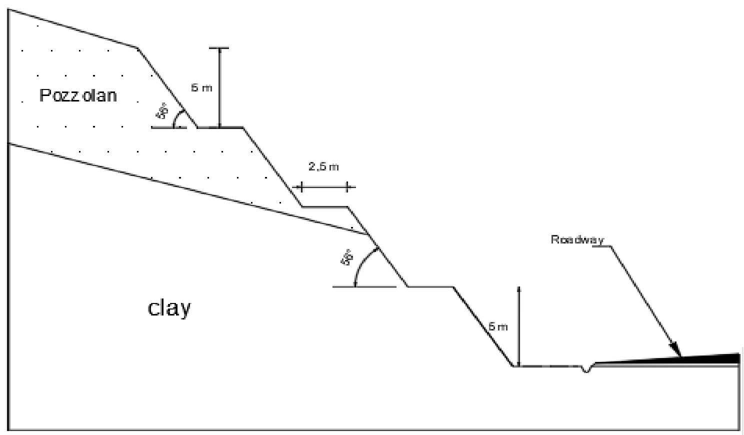


Figure 3.7. Cut slope Case 1 2H/3V (Redrawn from project document in appendix)

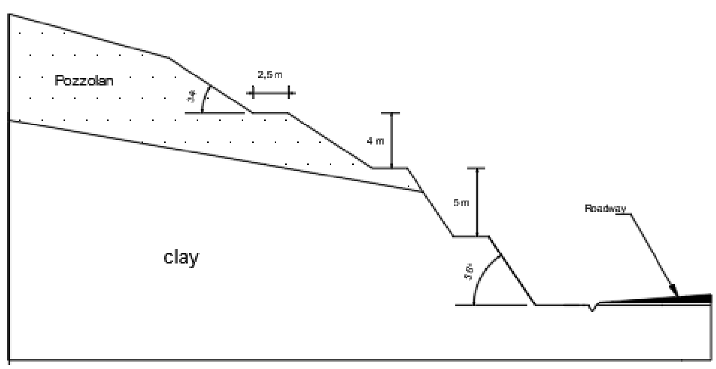


Figure 3.8. Cut slope Case 2 3H/2V (Redrawn from project document in appendix)

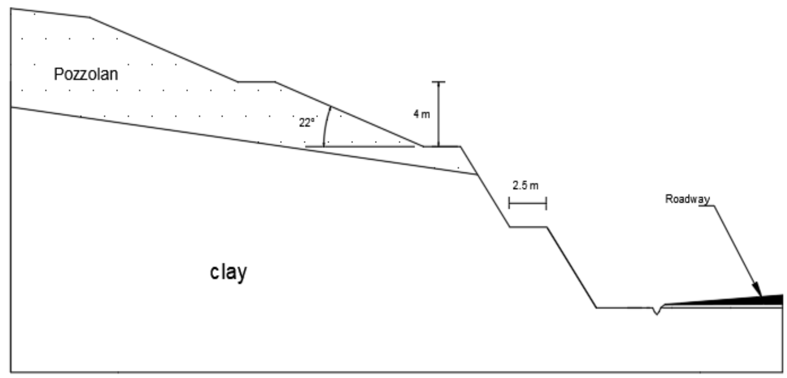


Figure 3.9. Slope Case 3 5H/2V

3.4. Numerical analysis of stability of cut slope

For all the 3 cases of slopes of this study, the numerical analysis is performed using the Slope/W of GeoStudio 2018 R2 and Plaxis 2D v8.6 code, this in order to compare the obtained results and to obtain the best critical factor of safety. These analyses were performed for the long-term condition since pozzolan is a highly drained sandy material.

3.4.1. Stability analysis without reinforcement

3.4.1.1. Case 1: 2H/3V

a. Analysis with LEM

The analysis in GeoStudio software is done for local and global stability using the Morgenstern and Price method. These local and global stability are performed by varying the location of the entry and exist slip surfaces range.

i. Global analysis

The overall stability analysis of slope case 2H/3V was performed by putting the entry and exist slip surface at the highest and the lowest altitude of the slope. The result shown in Figure 3.10 presents a deep slip surface with a factor of safety 1.395 greater than 1.25. Therefore globally, the slope case 2H/3V is stable.

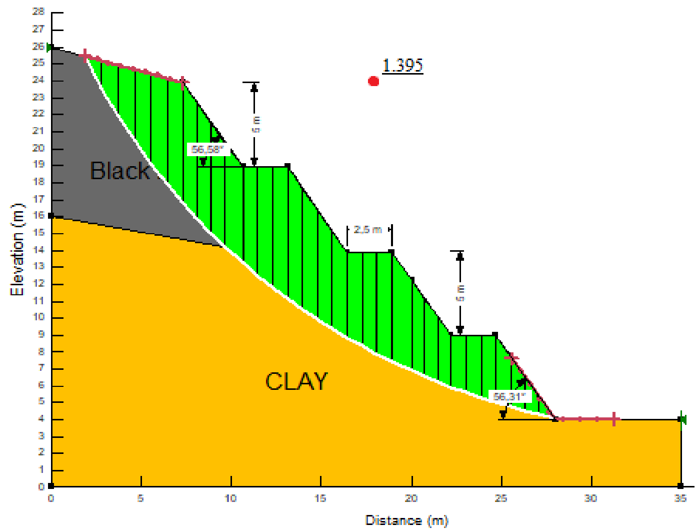


Figure 3.10. Global Stability of slope case 1 2H/3V (GeoStudio R2 2018)

ii. Analysis of the pozzolanic part

Regarding the local analysis of the slope, the result presented in Figure 3.11. shows a shallow slip surface (parameter presented in Table 3.4.) with a safety factor of **0.899 < 1.25**. Therefore, the pozzolanic part is unstable.

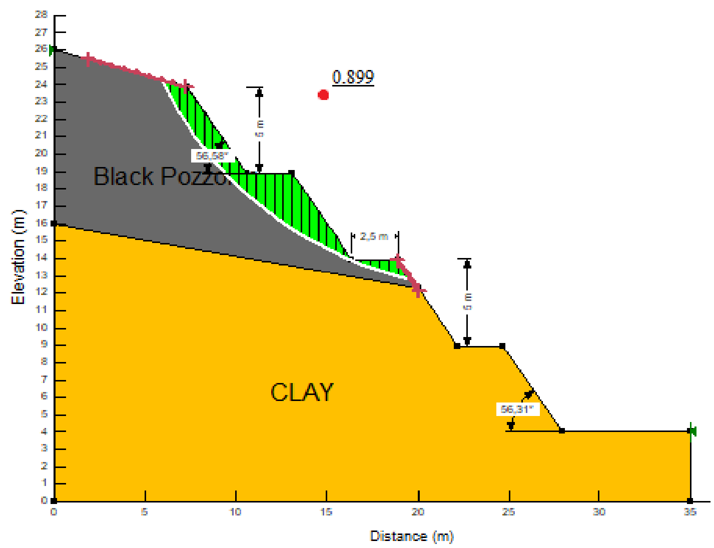


Figure 3.11. Stability analysis of the pozzolan part for slope case 1 2H/3V (GeoStudio R2 2018)

Table 3.4. Critical sliding surface information (GeoStudio R2 2018)

Slip surface	Factor of safety	Radius	Resisting Moment	Activating Moment
353	0.899	20.16547 m	765.36097 kN·m	851.76373 kN·m

b. Analysis with FEM

PLAXIS computes FoS by the c-φ reduction procedure, a detailed description of which can be found in Chapter 2. In this software, it is not necessary to predefined the position of sliding surface. Therefore, we obtain directly the critical slip surface concerning the most unstable zone. The analysis gives a safety factor of 0.711 which is also lower than 1.25 but more critical than that obtained via Slope/w. The total incremental displacements that illustrate the position of the slip surface and the associated safety factor are given in Figure 3.12. This figure shows a shallow critical slip surface located at the crest of the whole slope.

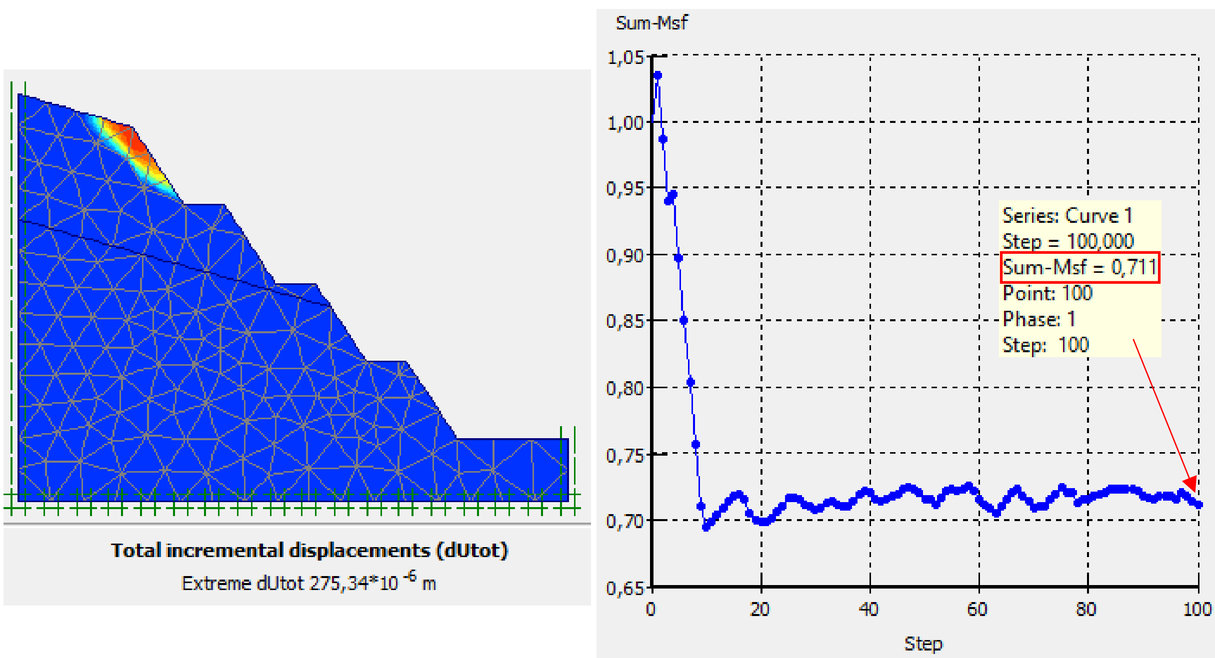


Figure 3.12. Stability analysis of slope case 1 2H/3V (Plaxis 8.6)

3.4.1.2. Case 2: 3H/2V

a. Analysis with LEM

Presented in Figure 3.13, the analysis of pozzolan part of slope case 2H/3V gives a safety factor of FoS = 1.236 which is obviously greater than 1 but does not satisfy the recommendations of the Eurocode 7(**1.236 < 1.25**). This slope case is therefore unstable.

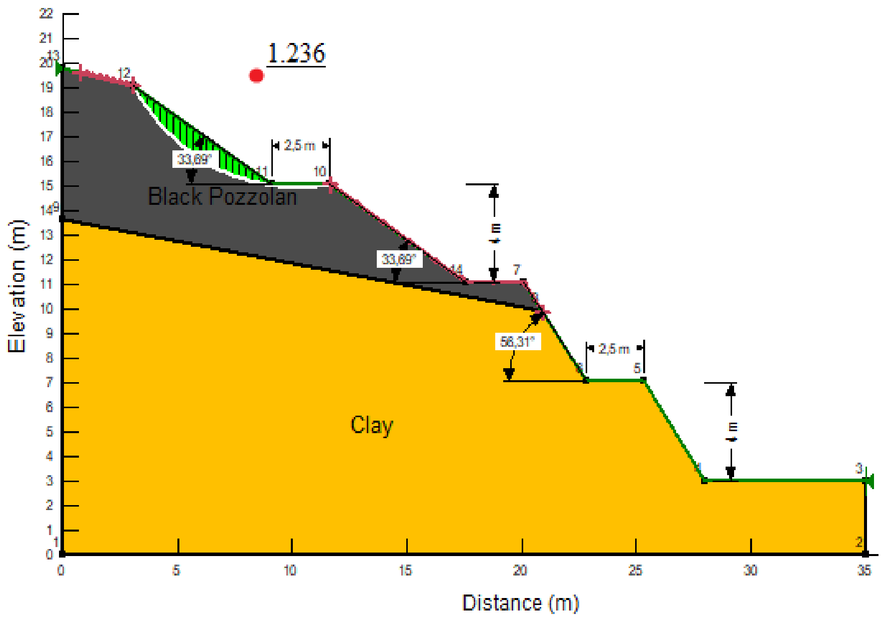


Figure 3.13. Local stability of slope case 2 3H/2V (GeoStudio R2 2018)

b. Analysis with FEM

When analysing with FEM, it can be seen that the area of maximum displacement is now located at the crest of the 2nd berm (see Figure 3.14). The Plaxis code gives also a critical factor of safety but slightly greater than the one in GeoStudio FoS = 1.246 as presented in Figure 3.15.

Since the safety factor of this slope case for both methods is less than 1.25, the pozzolanic part is qualified as unstable.

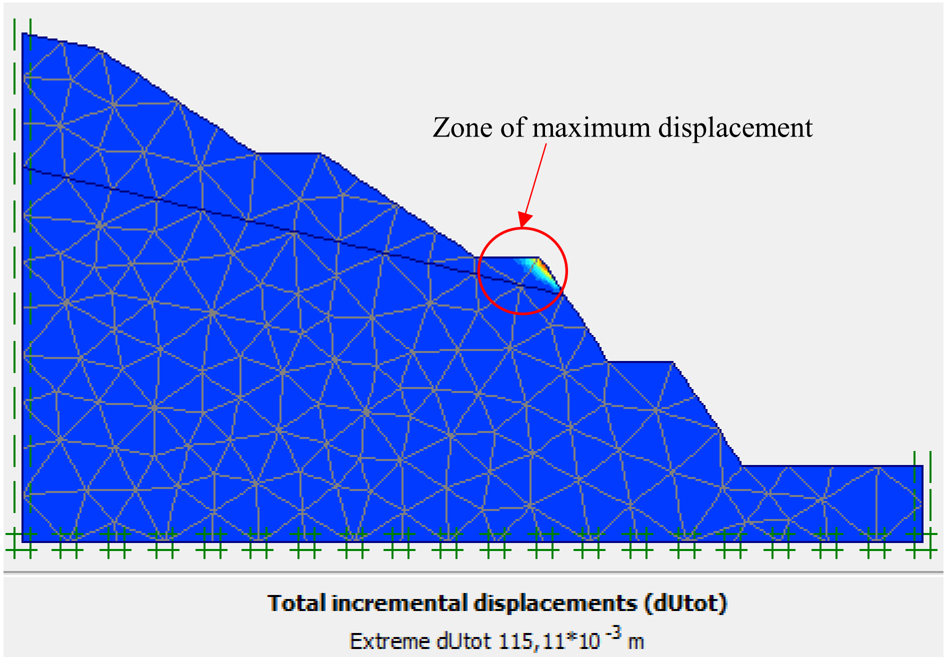


Figure 3.14. Stability analysis of slope case 2 3H/2V (Plaxis 8.6)

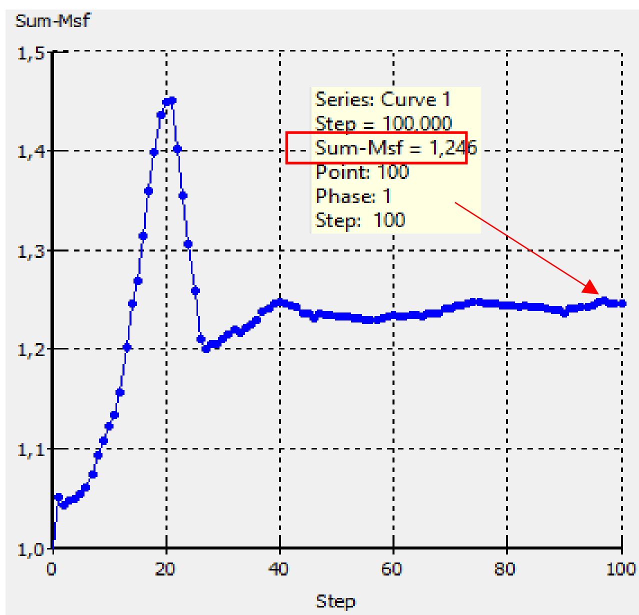


Figure 3.15. Msf vs step curve of slope case 2 3H/2V (Plaxis 8.6)

3.4.1.3. Case 3: 5H/2V

This slope case presents a safety factor greater than 1.25 (FoS = 1.913) using LEM and a value of 1.523 using FEM via the Plaxis code as shown respectively in Figures 3.16 and 3.17. Therefore, it can be qualified as stable.

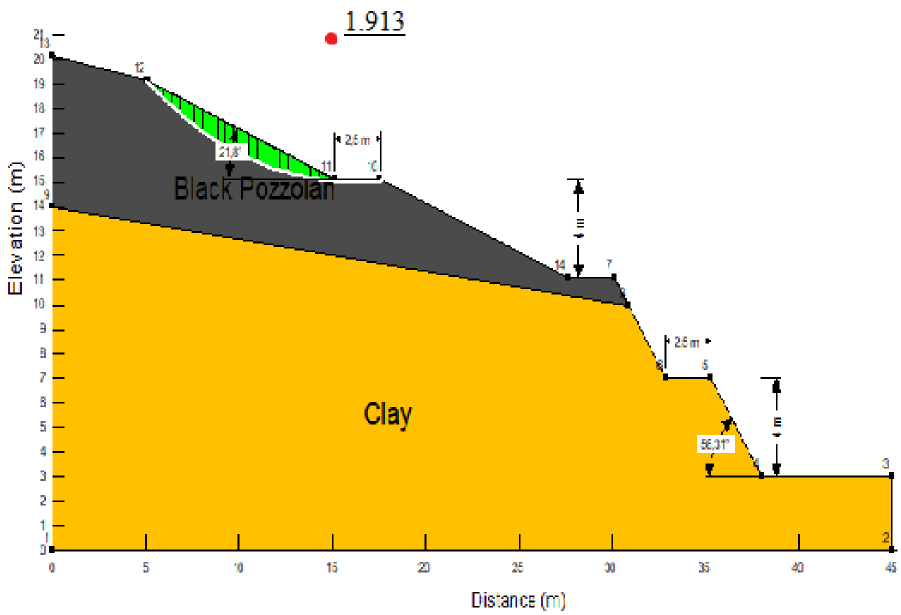


Figure 3.16. Local stability analysis of slope case 3 5H/2V (GeoStudio R2 2018)

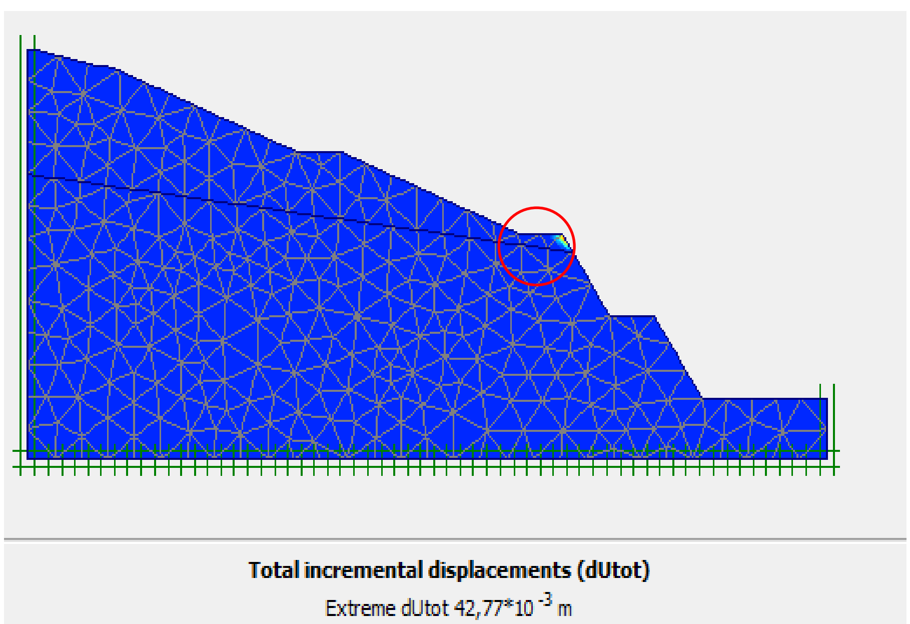


Figure 3.17. Stability analysis of slope case 3 5H/2V (Plaxis 8.6)

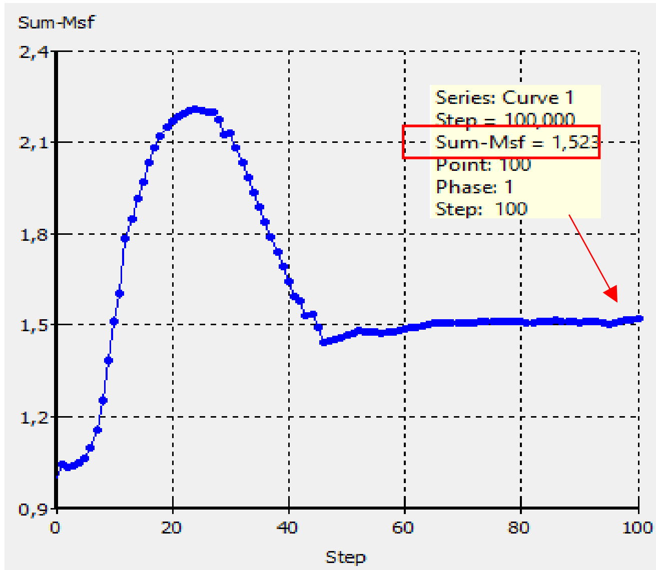


Figure 3.18. Msf vs step curve of slope case 3 5H/2V (Plaxis 8.6)

3.4.1.4. Conclusion on the different analysis

After analysis, it would be important to evaluate the accuracy of FoS obtained by one method compared to the other and the effect of the excavation angle on this safety factor.

a. Comparison of the obtained Safety factors

From the reported results, some similarities of the factors of safety can be appreciated. However, it comes that LEM presents FoS values both above (case 1 and 3) and below (case 2) the values obtained using the FEM (see Table 3.5). Considering the values above, it seems that LEM overestimated from about 20% the factor of safety as compared to the Finite element method, this can be due to the large amount of assumptions done in this method.

Table 3.5. Summary of stability analysis results

SLOPE CASES	Angles (°)	Factor of safety	
		GeoStudio (Local analysis)	Plaxis 2D
Case1: 2H/3V	56.31	0.899	0.711
Case2: 3H/2V	33.69	1.236	1.246
Case3: 5H/2V	21.8	1.913	1.523

The use of different analysis methods also allowed to better specify the failure zone (the most unstable slope zone) in order to define the adequate stabilisation method since the use of a remedial method depends highly on the position of the sliding surface as well as its depth. Therefore, in the case of the most unstable slope, i.e. the 2H/3V slope, the failure zone is located in the highest part of the slope (first berm). Any effective stabilisation method will focus on this area.

From the analyses carried out in this two software, it appears that the case of slope 3 is unanimously stable while the case of slope 1 is unanimously unstable. It is then the last slope case that will be analysed with different stabilization measures in order to improve the safety factor.

b. Effect of slope angle on factor of safety

From the same Table 3.5. we can also see that as the slope angle decreases (case 1 to case 3), the calculated safety factor increases and becomes equal to 1.25 for a certain angle value. This angle so called angle of repose is the angle at which the excavation on this pozzolan deposit should be done to acquire stability of the cut. This angle which is about **33°** is obtained for a FoS of 1.25 via the Figure 3.19. Exceeding this limit, it will only be obvious that the unstable slope will seek to reach its more stable grain arrangement.

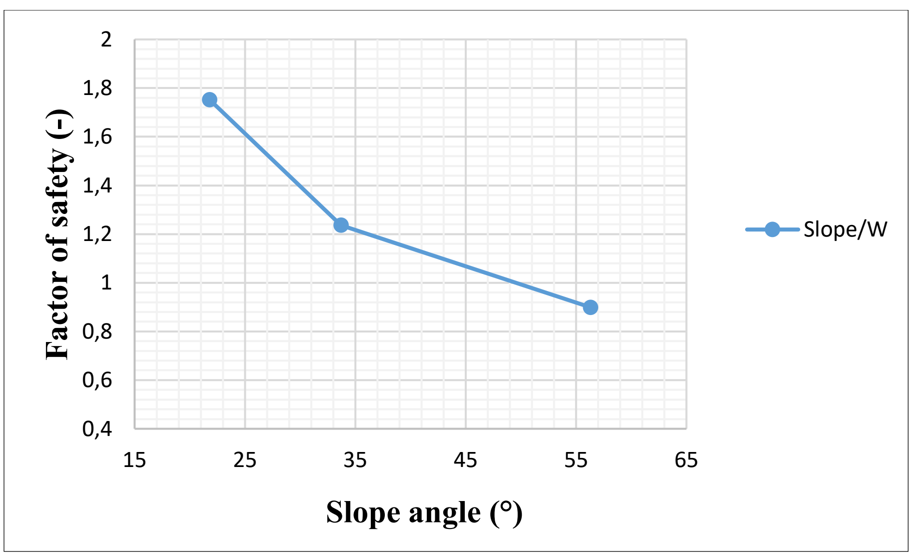


Figure 3.19. Effect of berm inclination on FoS

The effect of the angle of excavation on the factor of safety led Maclver (1967) to state the relation between the angle of repose **ρ** and the angle of deposition **δ** (or excavation). This relation is a better measure of the factor of safety of a granular cohesionless slope than the relation of either the angle of repose or angle of deposition to the angle of internal friction (which is a classical relation of limit equilibrium) and he establishes the following relation:

$$\mathbf{F_0S = \frac{\tan \rho}{\tan \delta}}$$

Stabilize the slopes 2H/3V and 3H/2V by excavating up to the angle of repose would be then a satisfactory option. However, it will be necessary to consider some important parameters:

- a slope bared by excavation is subjected to environmental actions such as rain, wind making it more unstable even though it is at its angle of repose
- the presence of houses at the top of the slope as shown in the Figure 3.20 was not included in the public utility domain (*DUP: Domaine d'utilité publique*) of the project. A new DUP would have to be drawn up and new expropriation procedures would have to be carried out, which would be extremely costly and time-consuming as the last procedure took 2 to 3 years.

Figure 3.20. House at top of slope at KP 49 of project

In conclusion, the option of making additional excavations or reducing the slope is rather limited in this context. It will be therefore necessary to stabilize this slope by reinforcement measures.

3.4.2. Stability analysis with reinforcement

Optimal stabilization selection requires identifying the technology that best fits the design criteria. As already said, slope analysis with reinforcement will be done only for slope case 2H/3V which has been shown as being the most unstable in both Slope/w and Plaxis 2D. This will be done by means of 3 different remedial measures.

3.4.2.1. Stabilization by Geocell slope protection

a. Result of design procedure

The Figure 3.21 presents how the geocell slope protection technique is implemented on the site.

Figure 3.21. Soil confinement system through Geocell (TERRAM Geocell)

According to TERAM Geocell, geocell for slope stabilization can be used on slope up to 1:1 (45°). Above this value, the sliding forces will lead the geocell and in fill soil down to the slope. Therefore, since the slope case 2H/3V has greater gradient than 1:1 **(56°.31 > 45°),** an excavation was firstly done. This gives a safety factor of 1.186 with LEM and 1.163 with FEM which is unstable.

For modelling, we choose a geocell with height h = 200 mm, cell diameter of D_o = 200 mm, a modulus of M = 320 kN/m and an axial strain at failure ε_a = 2.5%. We obtain:

- A confining pressure due to geocell $\boldsymbol{\Delta\sigma_3 = \frac{2M}{D_0}\left[\frac{1-\sqrt{1-\varepsilon_a}}{1-\varepsilon_a}\right]}$ = 41.3 kPa
- The additional cohesive strength due to geocell layer is $\boldsymbol{c_g = \frac{\Delta\sigma_3}{2} \cdot \sqrt{k_p}}$ = 39.65 kPa

  With the passive earth thrust of infill soil on geocell, $\boldsymbol{k_p = \frac{1+\sin\emptyset}{1-\sin\emptyset}}$ = 3.69

- The total cohesion of composite geocell + infill is **c = c' + cg** = 0 + 39.65 = 39.65 kPa

In plaxis 2D, the elastic modulus used to model geocell was obtained as:

$E_g = 101.325\ [13000 + 200 * 320^{0.16}]\ (41.3/101.325)^{0.7} = 734.301*10^3$ kPa = 734.3 MPa

The axial stiffness $E_gA = E_g * h * 1ml = 734.3 * 0.2 * 1 = 1.4686*10^5$ kN/m

The steel bars of 10 mm of diameter and one meter long spaced by 2 m used to fix the geocell into the soil below were modelled by plate element.

All the parameters used to model the geocell slope protection in slope/w and Plaxis 2D are presented in Table 3.6.

Table 3.6. Modelling parameter of Geocell slope protection

Parameters	Values
Slope/w	
Material model	Mohr Coulomb
Infill soil parameters	c = 0kPa; φ = 35°; γ = 19 kN/m^3
Geocell heigth	h = 200 mm
Composite Geocell + infill soil	c = 39.65kPa; φ = 35°; γ = 19 kN/m^3
Plaxis 2D	
Material model	Plate element
Axial stiffness of geogrid EA,	$1.4686*10^5$ kN/m
Axial stiffness of steel bar, EA	$2* 10^6$ kN/m
Flexural stiffness of steel bar, EI	16.667 kNm²/m

b. Result analysis

i. LEM results

The result of analysis using LEM presented in Figure 3.22. gives a factor of safety 1.350 greater than 1.25: Geocell has therefore stabilized the pozzolan cut slope.

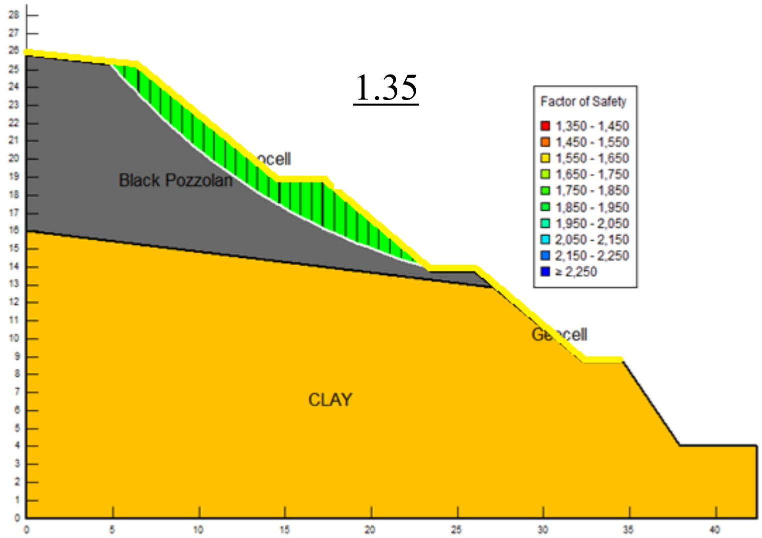


Figure 3.22. result analysis of slope reinforced by geocell (GeoStudio R2 2018)

ii. FEM results

Concerning the analysis with Plaxis 2D, the following construction stages have been applied:

- Phase 1: phi/c reduction calculation of the cutting slope;
- Phase 2: Plastic analysis calculation including Geocell+ infill soil;
- Phase 3: phi/c reduction calculation for the whole model.

After analysis, we obtain the Msf vs step surve of Figure 3.23 showing a safety factor of 1.310 which is greater than 1.25. The slope case 2H/3V is now in stable state.

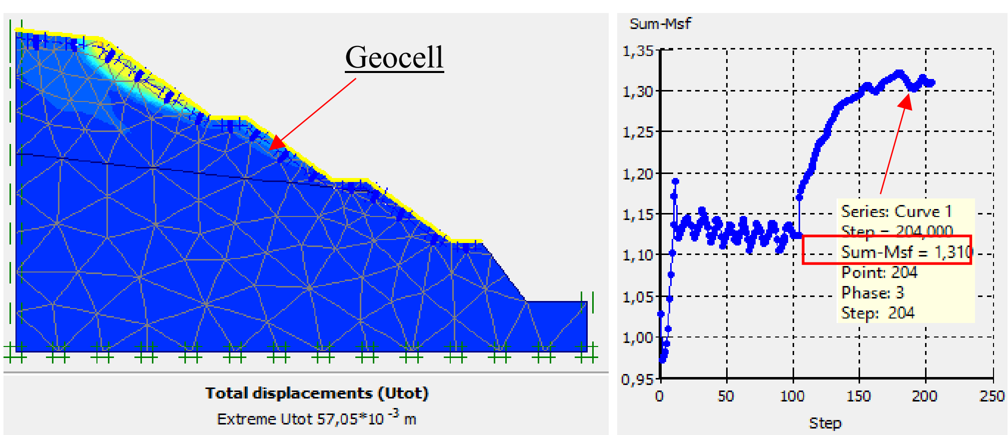


Figure 3.23. Plaxis result analysis of slope reinforced by Geocell (PLAXIS 2D)

The improvement of displacement was also evaluated by this method. Slope surface reinforced by geocell has reduced the horizontal displacement from **0.616 m** to **0.0365 m** as presented in the Figure 3.24.

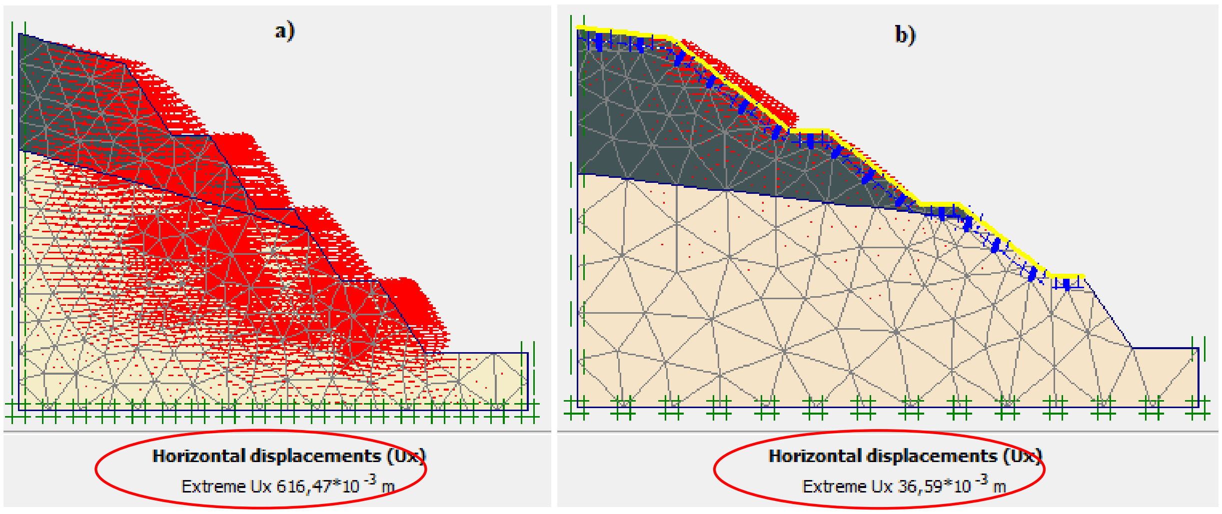


Figure 3.24. Horizontal displacement without (a) and with (b) Geocell stabilization (Plaxis 2D 8.6)

3.4.2.2. Stabilization using Shotcrete technique

a. Material properties

Reinforced concrete instead of unreinforced concrete was used to stabilize the unstable pozzolan slope in order to improve the tensile capacity. For stabilization of rock/soil highly degraded, EN 14487-1 recommends a shotcrete class which can reach a compressive strength of 100MPa at 28 days.

A unit weight of 25 kN/m^3, thickness d = 200mm, a compressive strength f_c = 45MPa are used for this study and we obtain an elastic modulus of concrete E_c = 36.3GPa, an axial stiffness EA= 7.26 *10^6 kN/m and flexural stiffness EI = 24200 kN.m^2/m for the modelling.

b. Results analysis

i. LEM results

The result analysis in GeoStudio presented in Figure 3.25. shows an increase of factor of safety from **0.899** to **1.467** which is now greater than 1.25.

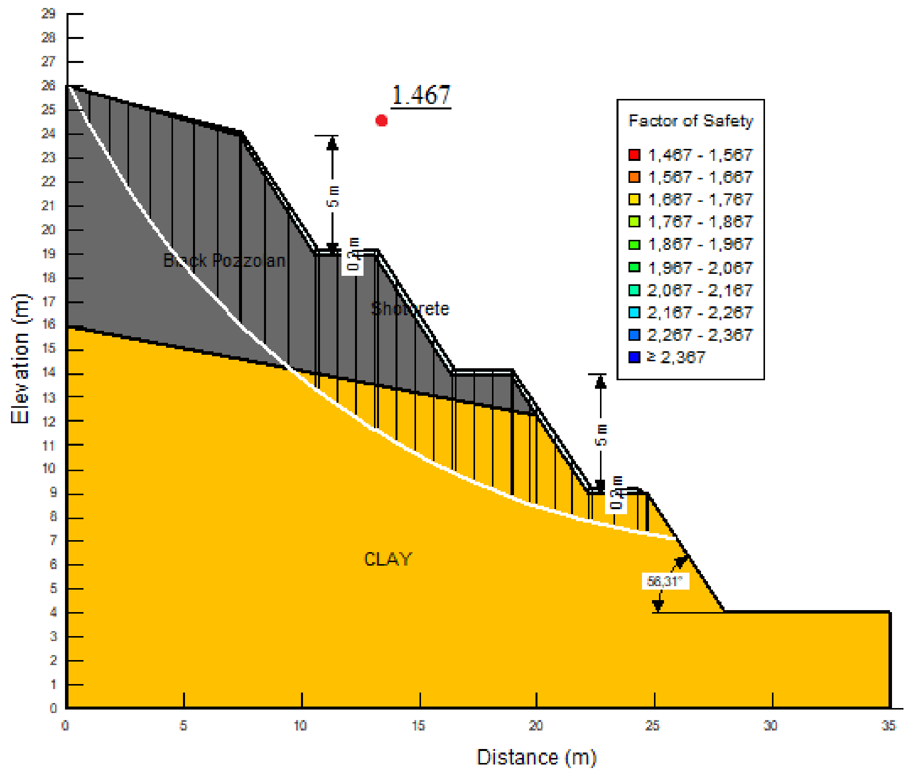


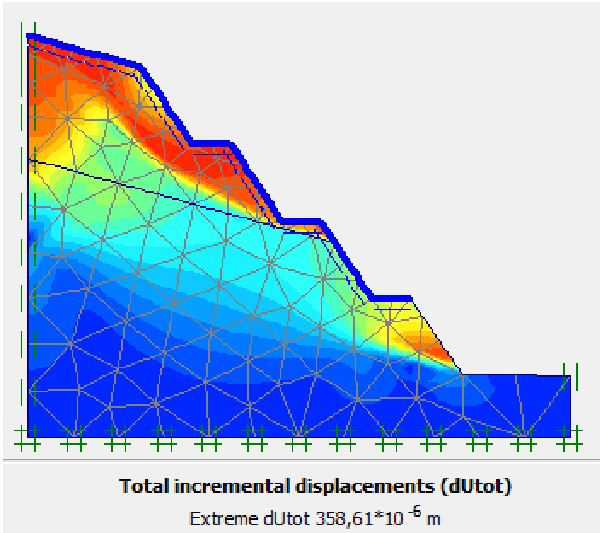
Figure 3.25. Result analysis of slope case 2H/3V by shotcrete (GeoStudio R2 2018)

ii. FEM results

Concerning the analysis with Plaxis 2D, the following construction stages have been applied:

- Phase 1: phi/c reduction calculation without shotcrete;
- Phase 2: Plastic analysis calculation including shotcrete;
- Phase 3: phi/c reduction calculation.

The output presents a slightly lower result as for GeoStudio one. The safety factor increases from **0.711** up to 1**.330 > 1.25** (Figure 3.26). These 2 convergent results for both LEM and FEM allow to conclude without any doubt, that shotcrete has improved the stability of the slope and can be therefore used.



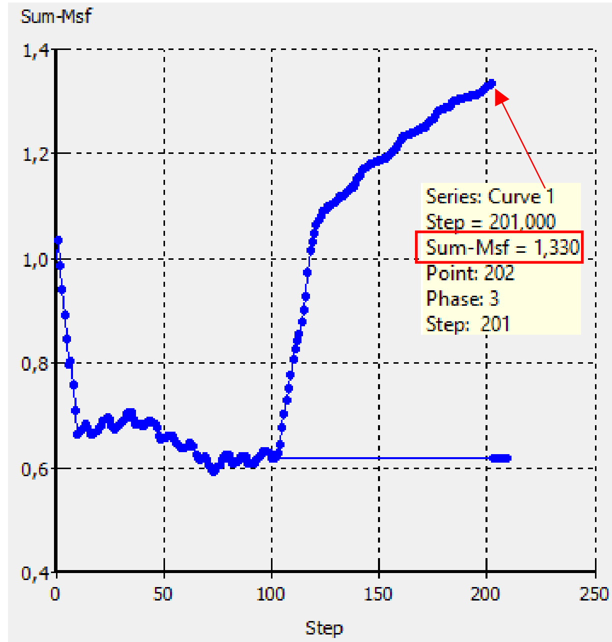


Figure 3.26. Result analysis of slope case 2H/3V reinforced by shotcrete (Plaxis 2D 8.6)

This increase in FoS is due to the fact that shooting concrete onto the slope surface create a sort of mattress that binds the surficial particles of pozzolan avoiding their detachment from the slope as earlier presented in Figure 3.3.a. above. The great permeability and porosity of pozzolan have played also important role in promoting the effect of this technique to stabilize the slope case 2H/3V.

The shotcrete technique have not just improved the FoS, but also the displacement especially the horizontal component passing from **0.616 m** to **0.0157 m** as we can see from the horizontal velocities arrows for slope without (Fig. 3.27.a) and with reinforcement (Fig. 3.27.b) in Plaxis 2D.

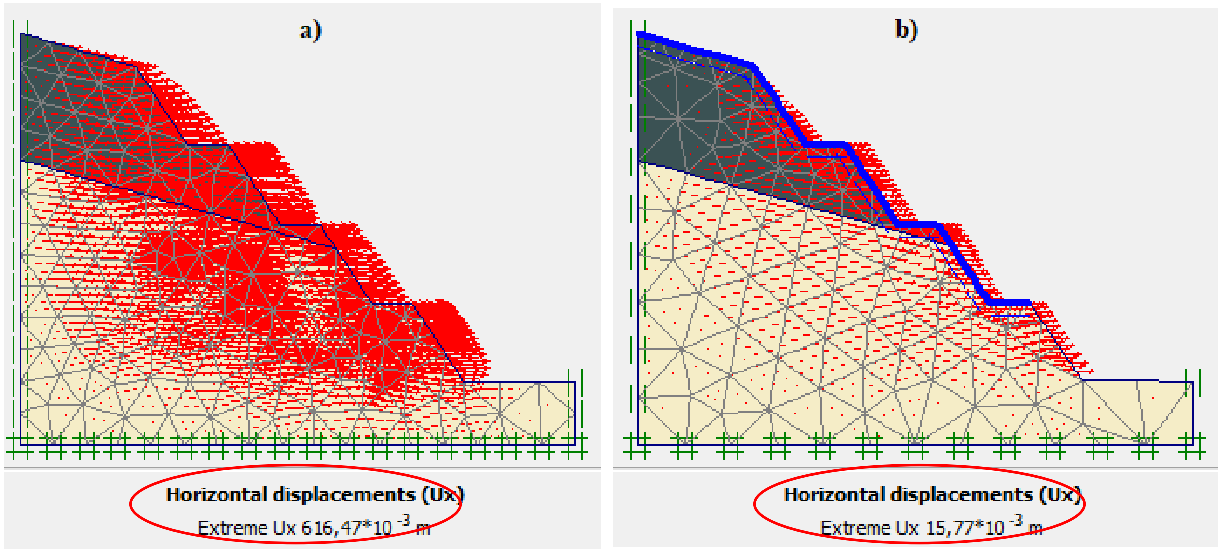


Figure 3.27. Horizontal displacement without (a) and with (b) shotcrete stabilization (Plaxis 2D 8.6)

3.4.2.3. Stabilization using a Soil Nailing

Soil nailing improves slope stability by increasing the normal force on the shear plane and reducing the driving force along the slip surface. It was chosen because it is applied directly onto the slope, and the nails can therefore intercept the sliding surface. Before analysing the 2H/3V slope case using this technique, it will be necessary to compute important parameters for the design by a preliminary design procedure.

a. Preliminary design

Among the parameters used for preliminary design, there are ones assumed, while others are the data of case study. These parameters are the following:

D_{DH} = 100 mm; H = 5 m; $\beta = 0°$; $\alpha = 0°$; $FoS_G = 1.35$; $S_v = 2$ m; $S_h = 2$ m

Using the table 2.4. in chapter 2 for a rotary drilled construction method in silty sand soil type (the sieve analysis shows that Foumbot pozzolan is a silty sand soil), we get q_u = 150 kPa. Therefore, the value of normalized bond resistance is obtained as:

$$\mu = \frac{150*0.1}{1.35*2*2*12.7} = 0.218$$

Using the chart of figure presented in appendix and the design procedure state in the methodology section 2.6.2.1, we get:

- a normalized soil length of L/H = 0.7;
- a preliminary nail length of L = 0.7*5 = 3.5m
- and a normalized maximum design nail force of t_{max} = 0.216.

We choose a nail length of L = 4m and assume the nail tensile capacity of about T_{max} = 150 kN.

In order to obtain the axial and flexural stiffness for modelling in Plaxis, we use a nail diameter d = 32 mm, an elastic modulus of nail E_n = 200 GPa and grout elastic modulus E_g = 22 GPa. Therefore, we get:

- Total area of grouted soil nail**:** A **=** 0.25 * π * 0.1² = 0.007854 m²
- Area of reinforcement bar A_n **=** 0.25* π *0.032² = 0.0008042 m²
- Area of grout cover A_g = 0.007854 - 0.0008042 = 0.00705 m²
- Equivalent elastic modulus of grouted soil nail E_{eq} = 40.22 GPa
- EA = 1.5794 *10^5 kN/m
- EI = 97.715 kN/m

All the design value used to model and analyse the 2H/3V slope case reinforced with soil nailing are given in Table 3.7.

Table 3.7. Parameters used for numerical modelling of soil nails

Parameters	Values
Length of nails, **L**	4 m
Inclination of nails, **i**	15°
Bond diameter, $\mathbf{D_{DH}}$	0.1m
Pull-out resistance, $\mathbf{Q_u}$	100 kPa
Resistance reduction factor	1.5
Nail spacing, $\mathbf{S_v}$	2 m
Nail tensile capacity, **T**	150 kN
Axial stiffness, **EA**	1.5794 * 10^5 kN/m
Flexural stiffness, **EI**	97.715 kNm²/m

b. Analysis results

Since soil nail technique is a top down construction, an excavation of 1m was done before modelling the nail inside the slope, increasing the berm width up to 3.5m.

i. LEM results

From the Figure 3.28, it is observed that the FoS has been raised from **0.899 to 1.474** which is now greater than 1.25 indicating that the slope is now stable. Therefore, the solution to the unstable pozzolan slope case 2H/3V can be solved with the used of soil nails as it has shown a clear proof from the factor of safety and the critical failure surface in the pozzolan part.

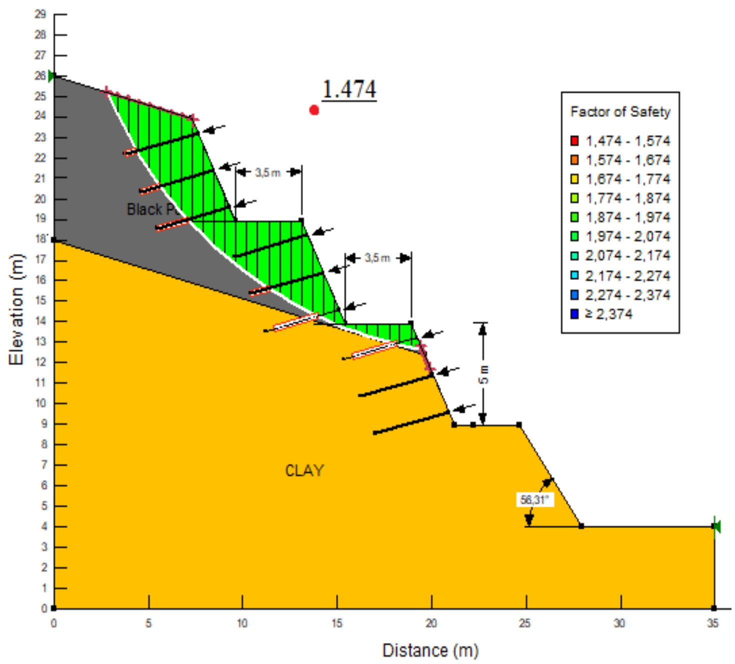


Figure 3.28. Result analysis of slope case 2H/3V stabilized by nails (GeoStudio R2 2018)

Though not presented in this figure, the factor of safety increases for slip surfaces that are shallower. Consequently, the design of this method is acceptable for both shallow and deep potential modes of failure inside the pozzolan deposit.

ii. FEM results

The analysis with Plaxis code is done proceeding the following construction steps:

- Phase 1: phi/c reduction calculation without nails;

- Phase 2: plastic analysis calculation including nails;
- Phase 3: phi/c reduction with nails.

The output sub program gives the slip surface shown in Figure 3.29.a) and the curve sub program the safety factor (Msf) vs step curve presented in Figure 3.29.b)

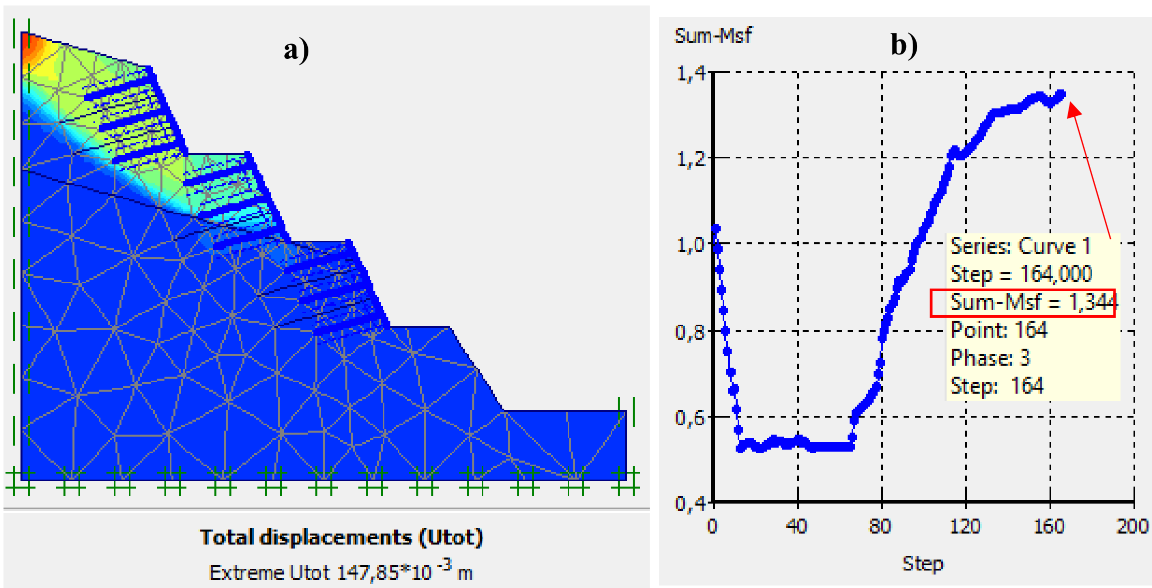


Figure 3.29. Result analysis of slope case 2H/3V stabilized by nails (Plaxis 2D 8.6)

The graph of Msf vs step number of this analysis shows a safety factor **1.344** which is greater than 1.25 improving by this the stability of the pozzolanic part of the 2H/3V slope. This result, joined with the one of GeoStudio confirms that Soil nailing is a good improvement technique to stabilize the unstable pozzolan cut slope.

As for the previous remedial method, the influence of soil nailing to the horizontal displacement was also evaluated. Figure 3.30. presents the horizontal velocities arrows for slope without (Fig. 3.30.a) and with nail reinforcement (Fig. 3.30.b) showing a considerable reduction of horizontal displacement due to the reinforcement passing from **0.616 m** to **0.0741 m**.

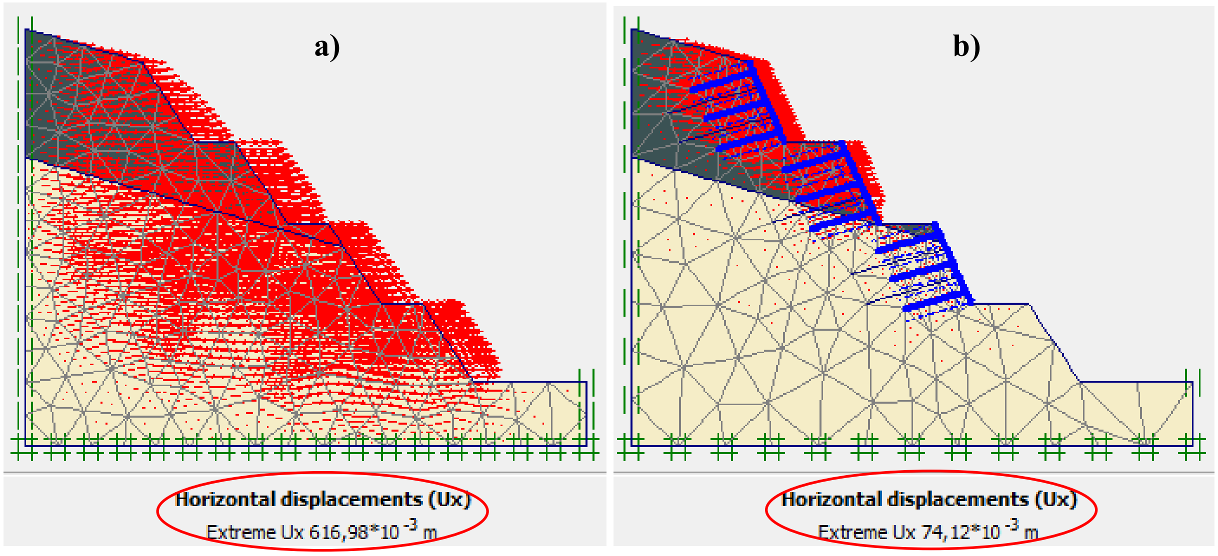


Figure 3.30. Horizontal displacement without (a) and with (b) soil nails stabilization (Plaxis 2D 8.6)

3.4.2.4. Summary of analysis reinforced slope

The Table 3.8. presents the safety factor obtain for all the 3 remedial method using both LEM and FEM analysis. Since the analysis showed a superficial instability, the use of stabilization measures directly applied to the pozzolan slope surface become necessary to remedy the problem. Geocell protection + excavation has increased the factor of safety up to 1.350 while Soil nailing has increased up to 1.474 and a shotcrete up to 1.467 making the pozzolan cut slope stable.

Table 3.8. Summary of safety factor without and with reinforcement

Analysis method	FoS of Unreinforced slope	Factor of safety of Reinforced slope		
		Geocell	Soil nailing	Shotcrete
GeoStudio	0.899	1.350	1.474	1.467
Plaxis	0.711	1.310	1.344	1.330

Even the total and horizontal displacements of pozzolan grain downward given by Plaxis code were also improved by these remedial techniques. The summary of these results is presented in Table 3.9.

Table 3.9. Summary of displacement values without and with reinforcement

Displacements	**Unreinforced**	**Soil nailing**	**Shotcrete**	**Geocell**
Horizontal (**U_x**)	0.616 m	0.0741 m	0.0157 m	0.0365 m
Total (**U_{tot}**)	0.919 m	0.147 m	0.0234 m	0.0570m

For both horizontal and total displacement, shotcrete is the method that has reduced the best the displacement of the slope.

Also, according to Table 3.10, these obtained displacements have not relevant effect on the stability since all the horizontal displacements are lower than 0.1cm.

Table 3.10. Limit values for permanent displacement (D'Elia, 1998)

Displacement	**Effect on slope stability**
<10 cm	Not relevant
10-100 cm	Relevant cracking associated with reduction in soil shear resistance causing failure during or after seismic shaking
>100 cm	Destructive movement

Since these 3 remedial techniques have stabilized the pozzolan cut slope, it would be essential to make a choice of the optimal one in terms of time construction, resistance, durability, cost and influence on environment.

3.5. Comparative study and choice of the optimal solution

3.5.1. Comparative study

The comparative study between soil nailing, shotcrete and Geocell slope protection technique was performed using 3 criteria namely the technical criteria, financial criteria and environmental criteria.

3.5.1.1. Technical point of view

The comparison on the technical point of view is assessed on the basis of resistance and durability of the technique, the construction procedure and execution time.

a. Resistance (FoS)

To help in the assessment of the most suited remedial option for the unstable pozzolan slope, the improvement factor of each technique in both LEM and FEM is established. It was calculated using the following expression:

$$\mathbf{IF} = \frac{\mathbf{F_0S_r}}{\mathbf{F_0S_{unr}}}$$

Where **FoS_r** is the safety factor of reinforced slope and **FoS_{unr}** without reinforcement.

We get:

- For geocell slope protection technique IF_{LEM} = 1.501, IF_{FEM} = 1.842
- For Soil nailing technique IF_{LEM} = 1.639, IF_{FEM} = 1.890
- For Shotcrete technique IF_{LEM} = 1.632, IF_{FEM} = 1.876

In order to better visualized the difference between these values, we plot the graph presented in Figure 3.31.

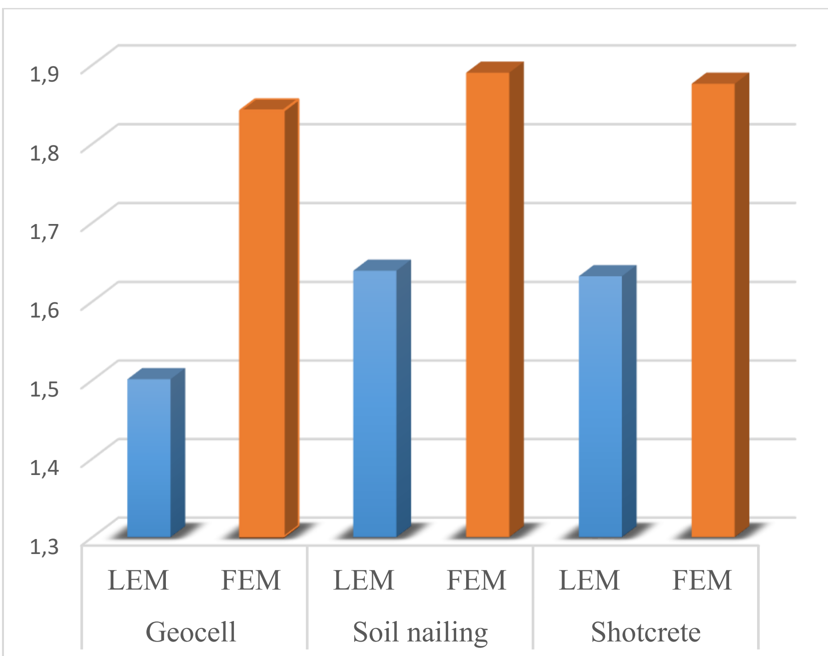


Figure 3.31. Safety factor improvement of each technique in LEM and FEM

From this Figure 3.31, we notice that for both analysis method, soil nailing technique has a greater improvement factor than shotcrete and geocell. This means that soil nailing technique has improved the stability of pozzolan slope better than others.

b. Durability

The durability is an important parameter to ensure the long-term life service of a given structure. Soil nails used in this study is designed for permanent purposes and for a life span of 100 years. The durability of a steel soil nailed system is governed primarily by the resistance to corrosion under different soil aggressivity. One of the factors influencing the aggressivity of soil is its PH value which is considered aggressive for a PH value less than 5.5. According to IRS CAM (1958), pozzolan of Foumbot city has a PH ranging from 6.2 to 7 (greater than 5.5) which is therefore no aggressive for the nail and will not reduce the durability of this technique.

Looking back from 30 to 40 years ago of some projects in U.S. where concrete was sprayed onto soil slope, it has been seen that shotcrete remains always stable (American Shotcrete Association, 2015). Furthermore, since in some cement manufactures, pozzolan powder is included to increase the internal cohesion and compactness of the concrete at hardened state, pozzolan is not a foreign, harmful, or incompatible material for concrete. Shotcrete can therefore maintain stable pozzolan slopes throughout the expected service time.

Geocell used in this study is the Neoloy Geocell which are the only qualified geocell today having a high strength and stiffness and with a performance guarantee of 75 years.

Therefore, in terms of durability, we choose the soil nailing technique as the most prevalent.

c. Construction procedure

The construction procedure of shotcrete onto the slope is defined by the use of pumping machine. When the concrete has been correctly mix being dry or wet, it is pumped onto the slope by means of a nozzleman or mechanically without any preliminary excavation.

Similarly to shotcrete, soil nailing requires a huge use of specific machine for the drilling and nail insertion with a specific angle and length. As a difference, it is a top down procedure requiring an excavation before the insertion of each row of nail. Also, this technique is not well known and applied in Cameroon, therefore, it will require a foreign expertise for implementation.

Specially in this project, the implementation of geocell onto the 2H/3V slope case required a great excavation work. Apart from this, the installation of Geocell on the slope is done in a very simple way, using mainly manpower.

Geocell slope protection was then chosen for its simplicity due to the fact that its installation procedure does not depends on the use complicated machines.

d. Effect of construction procedure on pozzolan

Even if all the above method has presented a considerable increase in FoS, it will be important to take into account their construction effect on the pozzolan soil. Pozzolan of Foumbot city is a well graded silty sand soil as earlier presented. As any others granular soils, it can liquefy when subjected to vibrations due to insertion of nail, even more when it is in saturated conditions. Therefore, considering the fact that we have no information on the water table, the feasibility of the use of soil nail in pozzolan cut slope can be therefore reduced.

On the other hand, the procedure of applying shotcrete tends to maximise the adherence between pozzolan particles because due to its pneumatic application, pozzolan is compacted against the surface to fill the voids between particles and prevent loose material from falling. This procedure limits the perturbation of pozzolan grains on slope.

The installation of geocell on the slope is not an activity that can disturb the pozzolan deposit and create a lot of debris material on pavements. It is therefore favourable to this material.

e. Execution time

The installation of geocell slope protection involves a huge amount of manpower, it can be therefore executed very slowly. In contrast, the execution time of shotcrete and soil nails methods depends highly on performance of the used machine. But since in soil nail technique a time is requires for excavation which does not exist in shotcrete technique, we can say that Shotcrete prevails on others stabilization methods.

3.5.1.2. Financial point of view

a. Reinforced slope area calculation

The price of each method was estimated for a slope length of 500 meters. Since the retained methods are especially surface covering, we will use a 3D model of slope case 2H/3V presented in

Figure 3.32. in order to well explain the calculations of slope surfaces in which each method is applied.

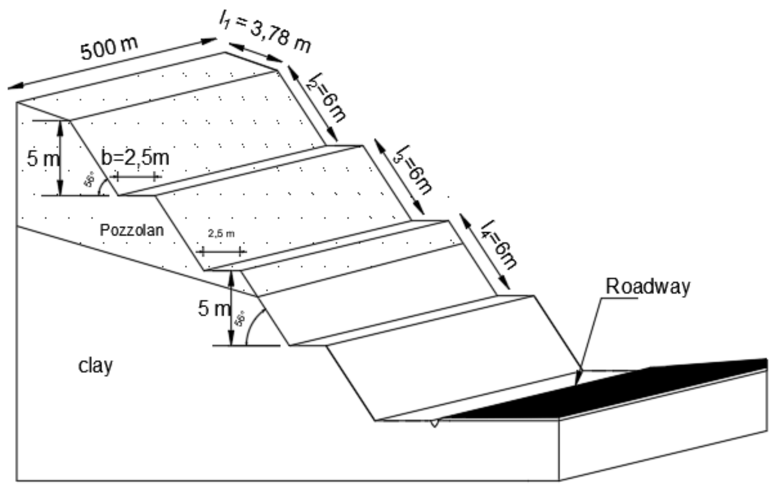


Figure 3.32. 3D model of slope case 2H/3V for a length of 500m (AutoCAD, 2018)

Shotcrete was applied covering all berms width and height except to one near the pavement. The considered slope surface is obtained as: $S_1 = 500*(l_2+l_3+l_4+3*b+l_1) = 500*(3*6 + 3*2.5 + 3.78) = 14640m^2$.

Concerning soil nail stabilization technique, it was just applied on the 1st 2 berms (the highest berms). So, we obtain: $S_2 = 500*(l_3+l_2+l_4) = 500*3*6 = 9000m^2$.

b. Shotcrete cost

The shotcrete used to stabilize the pozzolan banks is a sprayed concrete class SC7 C37/45. Steel bars of d=6mm with spacing of 15cm were used to enhance the tensile capacity of concrete.

We obtain for a unit surface of 1m^2, 14 bars of one-meter length each (L = 14 m), and the volume of bars inside 1m^2 is $V = \frac{\pi d^2 L}{4} = 3.958\ 10^{-4}\ m^3$

The correspondent mass of steel = V. ρ = $3.958\ 10^{-4} * 7850 = 3.107$ kg of φ6 inside 1m^2 of shotcrete.

The price presented here are the one based on the public work contract of the project. The Table 3.11. presents the cost of shotcrete onto slope surface of $S_1 = 14640m^2$.

Table 3.11. Cost estimation of shotcrete

Designation	Unit	Quantity	Unit price (fcfa)	Total cost
concrete C37/45	m^3	2928	320000	936 960 000
Steel mesh of diameter d = 6 mm	kg	45486.48	1450	65 955 400
Total price				**1 002 915 400**

c. Soil Nailing cost

Since soil nail technique is not frequently executed in Cameroon, the cost estimation was based on the cost data for soil nail walls on U.S. transportation projects given by **FHWA-IF-99-026**. For permanent walls, this cost ranges from 161300 fcfa to 215025 fcfa per m^2 of slope.

In order to present the cost of each construction phases for this case study, we are based on the public work contract of the project to estimate the cost of excavation, facing and geotextile for drainage. Concerning the others steps which are equipment mobilization, self-drilling hollow bars (thread bars, nuts, plates, couplers) and their installation, we are based on the MAXDRILL ROCK TOOL CO., LTD and on the cost detail of soil nails construction in U.S. Therefore, we obtain the cost in Table 3.12. for slope surface of S_2=9000m^2:

Table 3.12. Cost estimation of soil nailing

Designation	Unit	Quantity	Unit price (fcfa)	Total cost (fcfa)
Material mobilization (Equipment mobilized including drills and compressors)	----	1	24 187 950	24 187 950
Excavation	m^3	3750	7500	28 150 000
Self-drilling hollow bars R32n (thread bars, nuts, plates, couplers)	1 Piece	2250	43984	98 964 000
Geotextile (for drainage)	m^2	9000	6500	58 500 000
Nails Installation	Per nail	2250	44600	100 350 000
Mortar of grouting	m^3	32.7	150000	4 905 000

Facing: Welded mesh 150mm x 150mm spacing, diameter 6mm and 12cm of concrete	m^3	9000	54905	494 145 000
Total				**811 201 950**

d. Cost of geocell slope protection

The cost estimation is based on the public work contract of the project. The black surface presented in Figure 3.33 shows the excavated surface of 3 berms in order to reduce slope angle Area = 84.58 m² calculated in AutoCAD 2018. The total excavated volume is therefore: V = 84.58 * 500 = 42290 m^3. Table 3.13. presents the total cost of Geocell slope protection of the study.

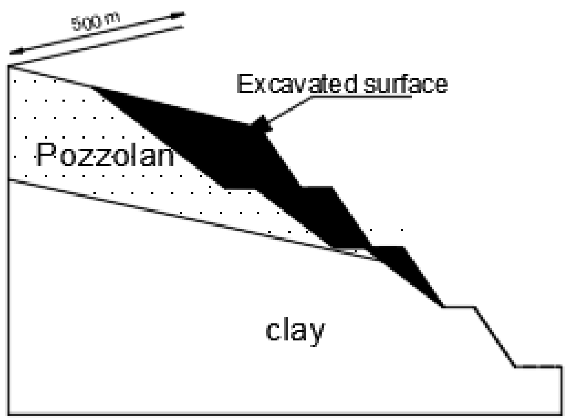


Figure 3.33. Sketch of excavated surface

Table 3.13. Cost estimation of Geocell slope protection

Designation	Unit	Quantity	Unit price (fcfa)	Total cost
Excavation	m^3	42290	7500	317 175 000
Geocell material, h = 200mm	m^2	18740	10.000	187 400 000
Steel bars to fix geocell into soil d = 10mm	1 bar (12 m)	390	4340	1 692 600
Infill soil	m^3	3750	4200	15 750 000
Total				**522 017 600**

e. Summary and comparison

After estimation of the cost of soil nail and shotcrete onto the 2H/3V slope case, we obtain a graph of Figure 3.34 showing by far the less expensive compared to others techniques. By using Geocell instead of Soil nailing and shotcrete, we reduce the cost of construction respectively from **35.65%** and **47.94%**. Therefore, based on the financial criteria, Geocell slope protection prevails on others.

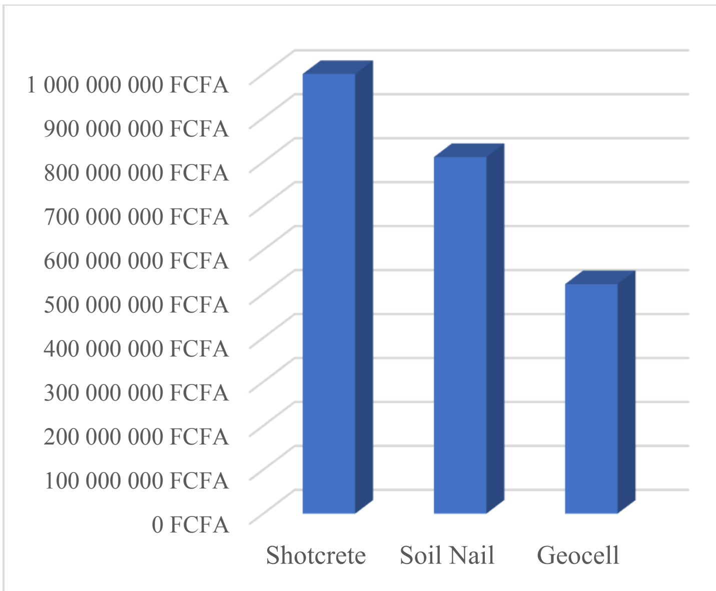


Figure 3.34. Summary of Cost estimation of each remedial method

3.5.1.3. Environmental point of view

In a sustainable development approach, it is very important to take into account the environmental impacts of our technologies. To do so, our approach will be based on the evaluation of the carbon footprint of each method (amount of CO_2 released into nature).

The Table 3.14. presents the amount of CO_2 of each technique evaluated kg of CO_2 for each construction step. These values are given by data sheet of carbon footprint given by ADEME v5 (2001 -2007). The quantities expressed in kg and m^3 come from the surface established in the financial criteria.

Table 3.14. Amount of CO_2 emitted by each remedial solution

Techniques	Material	Unit	Quantity	Total	Total
Shotcrete technique	Steel wire mesh (d= 6mm)	1.3 kg CO_2/kg	45486.5 kg	59132 kg CO_2	**622602 kg CO_2**
	Shotcrete	320.7 kg CO_2/ m^3	1757m^3	563470 kg CO_2	
Soil nail technique	Grout	937.6 kg CO_2/m^3	49.03 m^3	45968.36kg CO_2	**528692.36 kg CO_2**
	Facing	320.7 kg CO_2/ m^3	1080 m^3	346356 kg CO_2	
	Self-drilling hollow bars R32	1.2 kg CO_2/kg	113640 kg	136368 kg CO_2	
Geocell slope protection technique	Excavation	0.00246 kg CO_2/kg	53708300 kg	136124.87 kg CO_2	**143520 kg CO_2**
	Geocell	1.6 kg CO_2/kg	7121.2	11394 kg CO_2	

From Table 3.14 above, we can conclude that: the use of Geocell slope protection instead of Shotcrete reduces the amount of CO_2 of 77%. Similarly, a reduction of 73% in the amount of CO_2 is obtained using the Geocell slope protection compared to the nail insertion technique. Therefore, based on the environmental criteria, Geocell slope protection largely prevails on the others technique.

3.5.2. Choice of the optimal solution

In order to make a choice between these two methods, a table of criteria against the technique was drawn up, noting as (3) the most prevalent criterion, (2) the medium prevalent and (1) the least prevalent. In doing so, it is assumed that all criteria are equal, which is not always true. We obtain the results presented in Table 3.15.

Table 3.15. Degree of efficiency of improvement solutions proposed

Comparison Criteria		Remedial technique		
		Soil Nailing	**Shotcrete**	**Geocell slope protection**
Technique	Resistance (FoS)	3	2	1
	Durability	3	1	2
	Construction procedure	1	2	3
	Construction time	1	3	2
Cost		2	1	3
Effect on environment		2	1	3
Total		12	10	14

Therefore, from this Table 3.14 we can see that the optimal solution to remedy to the detachment of pozzolan grain onto the 2H/3V cut slope along the Bangangté-Foumbot-Bamendjing-Galim roadway is through Geocell slope protection.

The performance of this technique, as well as its safety factor, can be improved by the presence of native plants on the geocell and fill soil model as presented in Figure 3.35. This may depend on the type of roots and their configuration. The evaluation of the effect of plants on the 2H/3V slope case was not performed due to the lack of data on the depth of the roots and the cohesion they bring to the surficial soil.

Figure 3.35. Inclusion of plant root in the geocell slope protection technique (www.ecoraster.com)

However, another possible solution exists to evaluate the effect of selected native plants on the stability of pozzolan slopes. This can be done by means of an experimental study. It will require a lot of time and heavy laboratory equipment but present good results. The review (Mohammed, (2017); Gray and Sotir, (1995); Tardío and Mickovski, (2015); Prasad et al. (2012)) shows that although in the short term there is no great visible effect, it is very effective in the long term and is cheaper to stabilise non-cohesive soils with progressive failure mechanisms such as the pozzolan of this case study.

Conclusion

The main objective of chapter was the implementation of methodology, display the data collected and process the data to obtain necessary information. After the presentation of the site in its physical, economic and social aspects, the presentation of the site that followed allowed to better understand the instability problem and specially to recognize the type of landslide which led to establish possible causes of the instability. After the collection of geotechnical and geometrical data necessary for the numerical modelling, the numerical analyses via LEM and FEM confirmed the existence among the 3 slope cases of the unstable one (or the most unstable one) which is the 2H/3V case with a safety factor of 0.899 in LEM and 0.711 in FEM. Concerning reinforced slope results, all methods have proven to be effective in solving this problem with an increase in FoS of 33.4 % for Geocell, 38.71% for Shotcrete and 39% for Soil nails considering LEM and 46.54% for Shotcrete, 47.09% for Soil nailing and 45.47% for Geocell considering FEM. The comparative study that followed for the choice of the optimal method showed that: from a technical point of view, the shotcrete presents the less consuming execution time due to machine use; Geocell is the simplest in terms of construction procedure even though it is less effective in terms of resistance because it has shown the lowest safety factor improvement (IF = 1.501). The economic aspect showed that the Geocell appears to be more economical by 47.94% from Shotcrete cost and 35.65% from Soil nails cost. Finally, the environmental aspect showed that the use of Geocell slope reduces the amount of CO_2 produced of 77% and 73% compared to the Shotcrete and soil nail techniques respectively. This comparison led to the choice of Geocell slope protection as the optimal method for stabilising pozzolan slopes. Since its factor of safety is the lowest, its stabilizing effect can be increased adding the effect of roots vegetation on the protection system.

GENERAL CONCLUSION

The main objective of this thesis entitled "Stability of cut slope in a pozzolanic degraded rock mass along the Bangangté-Foumbot-Bamendjing-Galim Roadway", was to propose a technical and economic solution to stabilize the unstable bank of pozzolan after analysis of its stability.

To achieve this objective, a documentary research was first conducted to highlight the physical, chemical and hydraulic properties of pozzolanic soils as well as their mechanical behaviour determined by the Mohr Coulomb model which is similar to any other granular soil behaviour. Subsequently, its behaviour on a slope was clearly established, highlighting the factors influencing stability and the most common failure modes important to perform a good stability analysis. Secondly, based on the data collected from the "*Projet de construction de la route de desenclavement du Bassin Agricole de l'Ouest lot 2*", a numerical modelling and analysis of 3 cases of pozzolan slope was carried out and stability simulation of the unstable slope case under different remedial measures was also performed. The modelling was done with the SLOPE/W component of the GeoStudio 2018 R2 software based on LEM and with Plaxis 2D based on FEM allowing to obtain the factor of safety by a strength reduction method and also the deformation of the slope under different remedial technique. This is followed by a comparative study based on technical, economic and environmental criteria in order to select the optimal stabilization technique.

The results obtained enabled the following conclusions:

- The numerical analyses via LEM and FEM confirmed the existence of an unstable case among the 3 slope cases of pozzolan which is the 2H/3V case with a safety factor of 0.899 in LEM and 0.711 in FEM;
- Limit equilibrium method seems to overestimate from about 20% the factor of safety as compared to the Finite element method;
- All the 3 proposed remedial measures applied directly to the slope surface have proven to stabilize effectively the pozzolan cut slope. Shotcrete have increased the factor of safety of 38.71% in LEM and 46.54% in FEM; Soil nailing 39% in LEM and 47.09% in FEM and Geocell 33.4% in LEM and 45.47% in FEM. Also, the horizontal displacement of the slope evaluated in FEM was also improved by these remedial measures;

- The comparative study from a technical point of view has showed that shotcrete is the one requiring the smallest time execution due to the use of performant machine while Geocell is the simplest in terms construction procedure even though it is less effective in terms of resistance (safety factor);
- The economic aspect showed that the Geocell appears to be the less expensive from 47.94% less than Soil nailing and 35.65% less than Shotcrete;
- The environmental aspect has revealed that the use of Geocell slope protection reduces the amount of CO_2 produced of 77% and 73% compared to the Shotcrete and soil nail techniques respectively;
- Because of the several benefits of Geocell over soil nailing and Shotcrete, it has been chosen as the optimal method of stabilising pozzolan.

Limitation of the study

- Due to default of measurement, soil elastic parameters have been estimated based on the literature review which are not the exact value;
- Water table was not specified, leading to unrealistic conditions
- The results of the reinforced slopes have not been calibrated to verify their applicability on the site
- Seismic analysis was not carried out, although it is well known that granular materials are prone to liquefy under vibration, making the deposit unstable.

Perspectives

- Carry out in-situ tests to obtain the water table in each slope case. Make also a research of climatic data of the site, precisely the intensity of rainfall in the town of Foumbot in order to evaluate their effect on the stability of the slope.
- Compare the results obtained by the model with the in-situ behaviour of the slope.
- Since pozzolan is partially saturated in its natural state, it will be important to carry out a stability analysis of a pozzolan slope in a partially saturated condition taking into account the effect of suction on stability.
- Also, it will be important to research the effect of roots of Foumbot native plants in stabilising the cut slop

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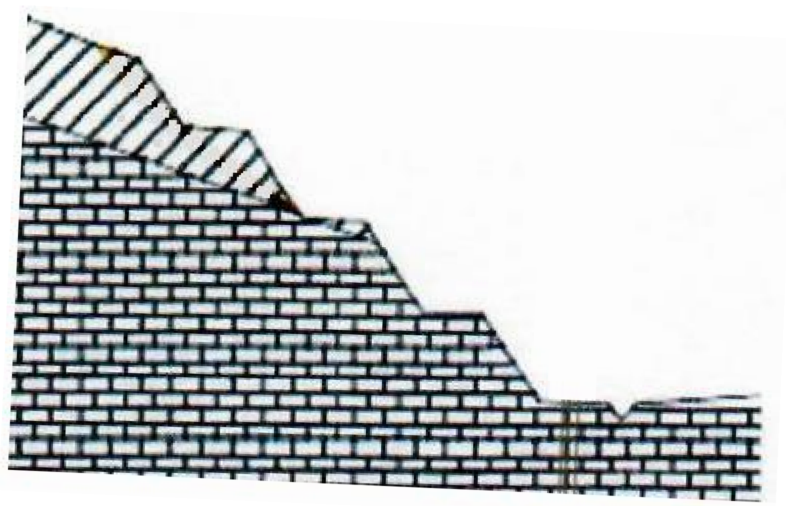
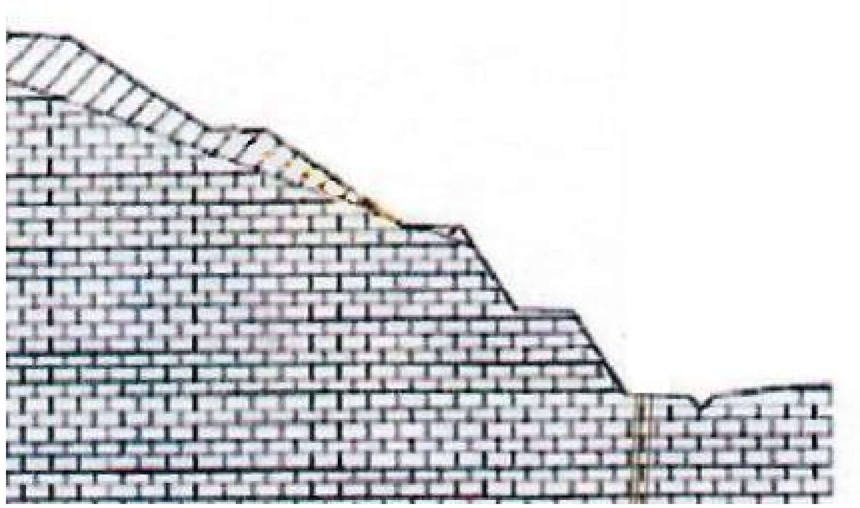
APPENDIX

1) Slope case 2H/3V from project document



2) Slope case2 3H/2V from project document



3) Direct shear test result sheet of pozzolan soil at 49+600 from project document

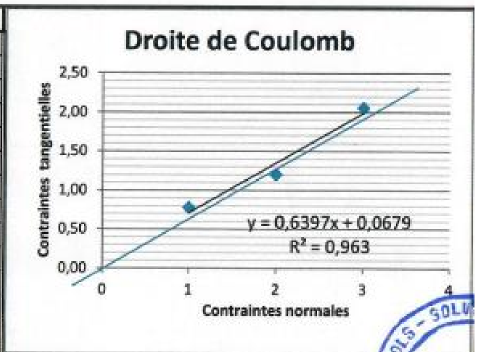
ETUDES CONCEPTION - PLANS - DEVIS - CONTROLE - MAITRISE D'OEUVRE - FORMATION - LABORATOIRE DE GENIE CIVIL - REALISATION DES PROJETS
BATIMENT - TRAVAUX PUBLICS
Ingénieurs - Conseils

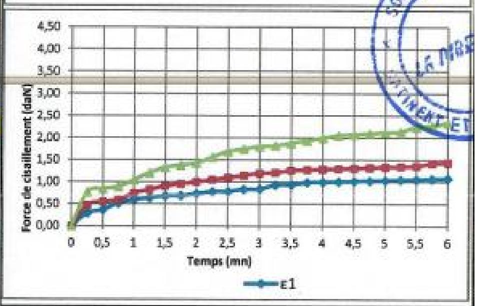
CISAILLEMENT RECTILIGNE (UU) A LA BOITE DE CASAGRANDE

Projet: GALIM-BAMENDJING-FOUMBOT-BANGANGTE
Nature: POUZZOLANE NOIRATRE
PK Prélèvement PK 49+600

Sondage: N°2
Profondeur : 3.00 - 3.80 m
Date: 26 Janvier 2017

Temps de lecture	Temps	contrainte σ_1= 1bar			contrainte σ_2= 2 bars			contrainte σ_3= 3bars		
(secondes)	(mn)	Lecture	Sc	ζ_1	Lecture	Sc	ζ_2	Lecture	Sc	ζ_3
0	0	0	0,00	0,00	0	0,00	0,00	0	0	0,00
15"	0,25	8	28,04	0,29	13	28,04	0,47	21	28,039	0,76
30"	0,5	10	27,80	0,37	15	27,80	0,55	23	27,803	0,84
45"	0,75	14	27,57	0,52	16	27,57	0,59	24	27,567	0,89
1'00"	1	16	27,33	0,60	20	27,33	0,75	28	27,332	1,04
1'15"	1,25	17	27,10	0,64	22	27,10	0,83	32	27,096	1,20
1'30"	1,5	18	26,86	0,68	24	26,86	0,91	35	26,861	1,33
1'45"	1,75	18	26,62	0,69	25	26,62	0,96	36	26,625	1,38
2'00"	2	19	26,39	0,73	26	26,39	1,00	37	26,389	1,43
2'15"	2,25	20	26,15	0,78	27	26,15	1,05	40	26,154	1,56
2'30"	2,5	20	25,92	0,79	28	25,92	1,10	43	25,918	1,69
2'45"	2,75	21	25,68	0,83	29	25,68	1,15	44	25,683	1,75
3'00"	3	21	25,45	0,84	30	25,45	1,20	45	25,447	1,80
3'15"	3,25	23	25,21	0,93	30	25,21	1,21	45	25,211	1,82
3'30"	3,5	23	24,98	0,94	31	24,98	1,27	46	24,976	1,88
3'45"	3,75	24	24,74	0,99	31	24,74	1,28	47	24,74	1,94
4'00"	4	24	24,50	1,00	31	24,50	1,29	48	24,504	2,00
4'15"	4,25	24	24,27	1,01	31	24,27	1,30	49	24,269	2,06
4'30"	4,5	24	24,03	1,02	31	24,03	1,32	49	24,033	2,08
4'45"	4,75	24	23,80	1,03	31	23,80	1,33	49	23,798	2,10
5'00"	5	24	23,56	1,04	31	23,56	1,34	49	23,562	2,12
5'15"	5,25	24	23,33	1,05	31	23,33	1,36	49	23,326	2,14
5'30"	5,5	24	23,09	1,06	31	23,09	1,37	51	23,091	2,25
5'45"	5,75	24	22,86	1,07	32	22,86	1,43	51	22,855	2,28
6'00"	6	24,00	22,62	1,08	32	22,62	1,44	52	22,619	2,34





CARACTERISTIQUES DU MATERIAU

Teneur en eau naturelle : 14.0 %
Densité humide : 15.51 KN/m^3
Teneur en eau après cisaillement : 14.9%
Densité sèche : 12.72 KN/m^3
WL= 0.0 % ; IP= 0.0 %
%AG à 80μ= 3,9
Cu = 0,000 bar
Cu = 0,0 Kpa
Ψu = 35.0°

4) Proctor test results sheet of pozzolan soil from project document

PROVENANCE : PK 53+861 CD ... | NATURE DU MATERIAU : Pouzzolane ...

Date de prélèvement	N° Echantillon	N° de l'essai	Classe granulaire	Etuvage	Pw	Ps	W
... -1- ...	1C3						

1. MOULAGE

MASSE ECHANTILLON	Mh		g	6000	— n —	6000	— n —
TENEUR EN EAU VISEE	Wv		%				
EAU DE MOULAGE	Me	=Mh*(Wv-W)/(1+W)	g				

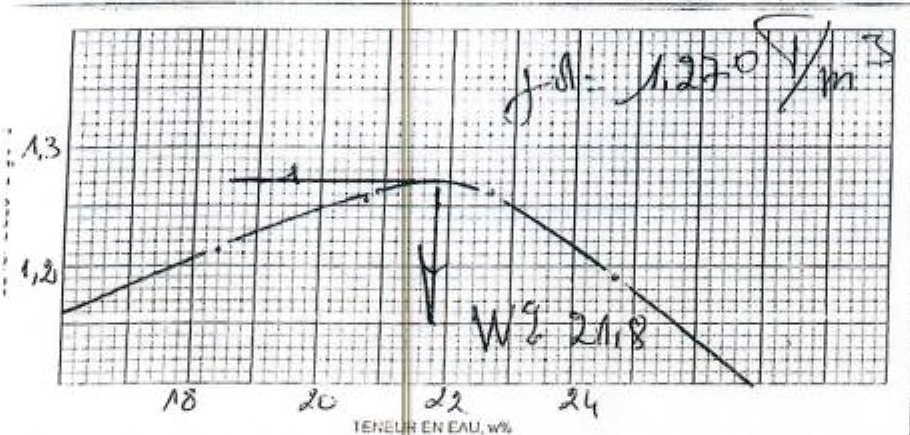
N° DE LA TARE			g	X	K	A44	P14	T1	A20	K8	A9
MASSE TOTALE HUMIDE	a		g	240,1	261,5	233,8	244,9	285,4	273,0	271,6	248,5
MASSE TOTALE SECHE	b		g	208,5	226,5	199,7	210,8	240,4	229,9	224,8	206,9
MASSE DE LA TARE	c		g	35,9	37,0	37,4	43,5	43,2	37,6	36,2	38,1
MASSE DE L'EAU	d	=a-b	g	31,6	35,0	34,1	34,1	45,0	43,1	46,8	41,6
MASSE DU MAT. SEC	e	=b-c	g	172,6	189,5	162,3	167,3	197,2	192,3	187,6	168,8
TENEUR EN EAU, w	f	=d/e	%	17,3	18,5	21,0	20,4	22,8	22,4	24,8	24,6
TENEUR EN EAU MOYENNE			%	18,1		20,7		22,6		24,7	

2. DENSITE

INTENSITE DE COMPACTAGE : 55 coups sur 5 couches

N° DU MOULE			p	15	— n —	15	— n —
MASSE DU MOULE	h		g	3795	— n —	3795	— n —
MASSE TOTALE HUMIDE	g		g	6831	6987	7053	6915
MASSE NETTE HUMIDE	i	=g-h	g	3036	3192	3258	3120
VOLUME DU MOULE	j		cm3	2105	— n —	2105	— n —
DENSITE HUMIDE	k	=i/j	g/cm3	1,442	1,516	1,547	1,482
DENSITE SECHE	l	=k/(1+w)	g/cm3	1,218	1,256	1,262	1,188

3. GRAPHIQUE



4. RESULTATS DE L'ESSAI

OPM

DENSITE SECHE (g/cm3) $\gamma_{d\ OPM}$ =

TENEUR EN EAU (%) W_{OPM} =

OPM corrigé

% MASSE >20mm $m\%$ =

DENSITE SECHE (g/cm3) $\gamma'_{d\ OPM}$ =

TENEUR EN EAU (%) W'_{OPM} =

DEGRE DE SATURATION S'_r =

5) Determination of normalized soil length L/H and the normalized max tensile force t_{max} using the Design chart of soil nailing giving by FHWA